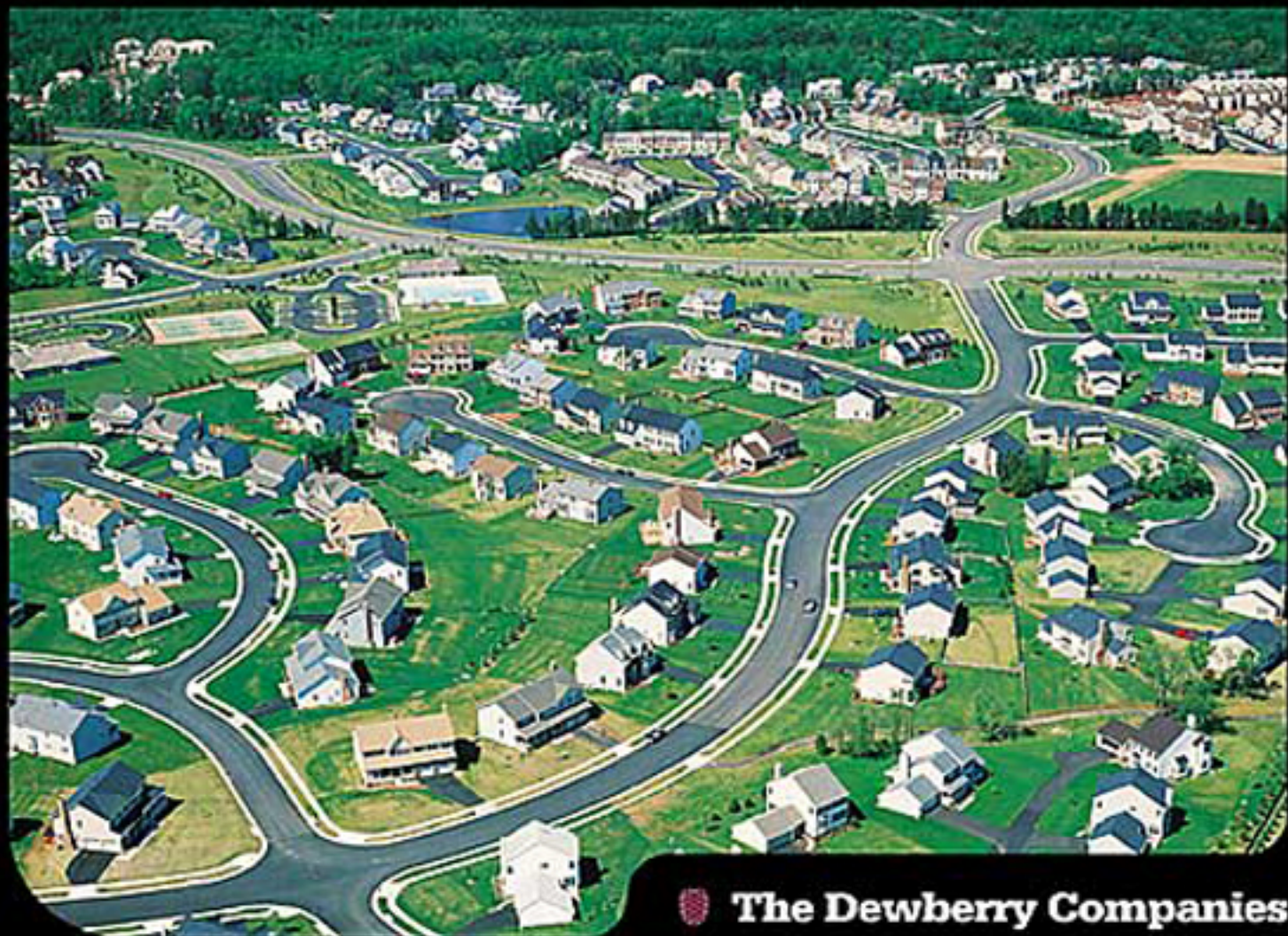


SECOND EDITION

# LAND DEVELOPMENT HANDBOOK

PLANNING, ENGINEERING, AND SURVEYING



**The Dewberry Companies**

# Table of Contents

Cover	1
PART I: OVERVIEW	4
1 OVERVIEW OF THE LANDDEVELOPMENT PROCESS	8
PART II: FEASIBILITY AND SITE ANALYSIS	24
2 COMPREHENSIVE PLANNING AND ZONING	38
3 SITE PLAN ORDINANCES, SUBDIVISION REGULATIONS, AND BUILDING CODES	66
4 EXACTIONS, INFRASTRUCTURE ENHANCEMENTS, AND FEES	72
5 REAL PROPERTY LAW	78
6 ENGINEERING FEASIBILITY	98
7 ENVIRONMENTAL REGULATIONS	138
8 ENVIRONMENTAL SITE ASSESSMENTS	146
9 HISTORIC AND ARCHAEOLOGIC ASSESSMENT	154
10 MARKET ANALYSIS AND ECONOMIC FEASIBILITY	160
11 THE REZONING PROCESS	174
P A R T III: CONCEPTUAL DESIGN	194
12 DEVELOPMENT PATTERNSAND PRINCIPLES	198
PART IV: SCHEMATIC DESIGN	228
13 BOUNDARY SURVEYS FORLAND DEVELOPMENT	232
14 CONTROL SURVEYS	250
15 TOPOGRAPHIC SURVEYS	276
16 PRELIMINARY ENGINEERING	300
17 ENVIRONMENTAL AND NATURAL RESOURCES	306
18 HISTORIC PRESERVATION AND ARCHEOLOGY	324
19 ENVIRONMENTAL CONSIDERATIONS	338
PART V: FINAL DESIGN	358
20 SUBURBAN STREET DESIGN	360
21 STORM DRAIN DESIGN	412
22 DESIGN OF STORMWATER MANAGEMENT FACILITIES	480
23 FLOODPLAIN STUDIES	540
24 GRADING AND EARTHWORK	550
25 WASTEWATER COLLECTION	600
26 WATER DISTRIBUTION	654
27 WASTEWATER TREATMENT	710
28 WATER SUPPLY AND TREATMENT	750
29 EROSION AND SEDIMENT CONTROL	776
30 CONTRACT DOCUMENTS AND SPECIFICATIONS	832

31 CONSTRUCTION COST ESTIMATING-----	836
PART VI: PLAN SUBMISSION AND PERMITTING-----	860
32 SUBDIVISION SUBMITTALS -----	862
33 PLAN SUBMITTAL, REVIEW, AND APPROVAL PROCESS-----	872
34 ENVIRONMENTAL PERMITS -----	886
PART VII: CONSTRUCTION-----	896
35 CONSTRUCTION STAKEOUTSURVEYS-----	898
36 BUILDING PERMITS AND CERTIFICATES OF OCCUPANCY -----	932
PART VIII: TECHNICAL APPENDICES-----	934
37 GEOSPATIAL DATA -----	936
38 TYPES OF DRAWINGS USED IN LAND DEVELOPMENT-----	966
39 SOILS -----	990
40 CASE STUDY: UNION MILLS -----	1054
41 TECHNICAL APPENDIX -----	1098

# PART I

# OVERVIEW



# PART I

# OVERVIEW



# CHAPTER 1

## OVERVIEW OF THE LAND DEVELOPMENT PROCESS

Sidney O. Dewberry, P.E., L.S. / Dennis Couture, A.S.L.A., R.L.A.

### LAND DEVELOPMENT

Urban design experts plan the cities and communities of today and tomorrow. The many specialists involved are highly educated and experienced in several fields. They handle a host of tasks, encompassing feasibility studies, zoning applications, environmental regulations, designs, permits, and the hundreds of steps necessary to design and construct a project.

### Just What Is Land Development?

The conversion of land from one use to another is the generally accepted definition of land development. As used in this book, it is confined to land conversion associated with the modern communities that are being constructed, or reconstructed, for people to live, work, worship, shop, play, and with other supporting land uses. This age-old process began when ancient societies organized themselves into tribes, settling on and claiming land, forming villages and primitive towns, for the mutual protection and livelihood of all. The great civilizations of Egypt, Greece, and Rome can be traced to humble beginnings in tribal communities. Their growth in size and complexity is typical of urban development and not unlike what we are experiencing today in the United States. With their complex roadways, aqueducts, commercial markets, and residential areas, the ancients battled problems of transportation, waste disposal, drainage, water supply, population densities, and a host of others. One can be certain that complaining neighbors existed then as now. Yet today, the process for finding solutions and developing scenarios that serve the greater good is systematic and, to a large degree, uniform in principle, and beginning to be uniform in practice. The systematic

approach to land use planning, analysis, and engineering is known as *land development design*. Land development is the entire process, from concept through design and construction.

### The Conversion of Land

Since the early 1950s, the conversion of land to a different use has generally meant a more intensive use. The definition formerly applied almost exclusively to residential, commercial, retail, industrial, and employment center land uses. It did not take long, however, before city planners and residents alike echoed Daniel Boone's call for elbow room and clamored to have areas preserved for recreational, educational, social, and cultural activities as well as for the roadways and services to support them. And while the typical land development definition was broadened to include such activities as conversion of rural land to agriculture use, construction of major transportation and utility systems, or even central city redevelopment, the meaning for the purposes of this Handbook is the conversion of land from one use to another, usually of greater intensity, and is typically applied to residential, commercial/retail, industrial, and employment centers with supporting uses and supporting infrastructure.

Land development design is the systematic process of collecting data, studying and understanding the data, extrapolating the data, and creating on paper the plans for reshaping the land to yield a land development project that is politically, economically, and environmentally acceptable to the client and the public. Persuasion, salesmanship, and negotiation are part of the process. For the purposes of this Handbook, architectural design is included only insofar as it provides preliminary or schematic elements sufficient to



define size, bulk, shape, and densities. Appearance, heights, setbacks, and aesthetics factor into the final product, and for this reason architects are frequently members of land development design teams. However, the detailed design of buildings is a subject for another time and another author.

### The Land Professional

With the advent of land use regulations, significant environmental constraints imposed by federal, state, and local governments, and the heavy involvement of citizens impacted by new developments, dedicated land use professionals must be prepared to draw on a wealth of resources to design a project that is appealing to both the end user and the surrounding community yet be cost effective to the client. These professionals must be adept at balancing the objectives of the client with the expectations of the citizenry and public approval agencies. They must be prepared to handle a diversity of projects, from the relatively simple conversion of vacant land to residential sites to the development of in-fill sites and complex, large-scale mixed-use projects.

The skills, innovative thinking, and creativity needed to make a land development project successful are gained through years of experience and encompass the entire spectrum of activities from land acquisition, rezoning, planning, engineering, and surveying to construction. The talents of many specialists, including environmentalists, architects, land surveyors, civil engineers, landscape architects, archaeologists, historians, geotechnical engineers, arborists, land use attorneys, and noise abatement technicians, just to name a few, contribute to a project's success.

### This Handbook

Undeniably, the land development design process, however systematic it is, varies considerably throughout the United States due to the diversity of state and local regulations controlling land use and land subdivision. Yet within the process there are many elements common throughout the jurisdictions. Even if it were intentionally written for a particular microregion, no book could present the specific design process due to the dynamics of the regulations. This book is a presentation of a typical design process, but in no way should it be construed as *the* design process.

It is our hope that this Handbook will aid you in anticipating the multiple issues and requirements you will no doubt encounter as you progress through the various stages of project development or as you seek to broaden your professional understanding of the complexities of land development. For the developer, it will be an invaluable tool to understand the services he or she will be acquiring from various design specialists and will prepare him or her for the regulation maze ahead. For those entering the land design profession, whether in the public sector or as a consultant, this Handbook will aid you in the development of the skills needed. For the practitioner it will prove an in-

valuable reference tool. But before we proceed with where we are today, a bit of background is in order.

### HISTORY OF SUBURBAN GROWTH<sup>1</sup>

At the heart of all suburban growth is land development—the conversion of rural or vacant land to some sort of residential use. The process involves property owners, speculators, banks, private lenders, builders, and buyers. As land values at the center of the metropolis rise, individual parcels either produce the higher yields to hold their place, or in the course of a few years, more profitable businesses move in on the site. By the same token, if much the same yield can be earned at a peripheral site of lower value, there is little incentive to remain in and around the central business district. The pattern of urban land investments affects the value of outlying farmlands, which either increase yields by more profitable crop, or, as is the usually the case, they give way to more lucrative subdivision and real-estate developments (Jackson, 1985).

These words describing the fundamental interrelationship of real estate, suburbanization, and land development were realized and practiced throughout the 19th century much as they are today. From ancient Mesopotamia to colonial America, one can find residential suburbs in every culture, yet the systematic evolution of suburbanization—the “process involving the systematic growth of fringe areas at a pace more rapid than that of core cities, as a lifestyle involving a daily commute to jobs in the center” (Jackson, 1985, p. 13)—did not really materialize until the very early 19th century in the United States.

Two significant factors influenced the suburbanization of America. First is the advancement of modern transportation. Second is the rapid population increase in cities during the first half of the 19th century (see Table 1.1).

### The Move to Suburban Life

In the 1700s, homes, shops, public buildings, hotels, places of worship, and commerce were interspersed throughout urban areas. This pattern reflects a lifestyle whose primary modes of transportation were the horse and foot. Shopkeepers usually lived above their stores, and those who didn't generally lived within one mile.

Before the 19th century, in most cities throughout the world, the distinction between the city and the country, that is, the suburbs, was both social and economic. The aristocracy lived at the prestigious addresses located in the core of the city near the places of culture and commerce. The denizens of the outer fringes farther away from available services were perceived to be inferior to those closer in.

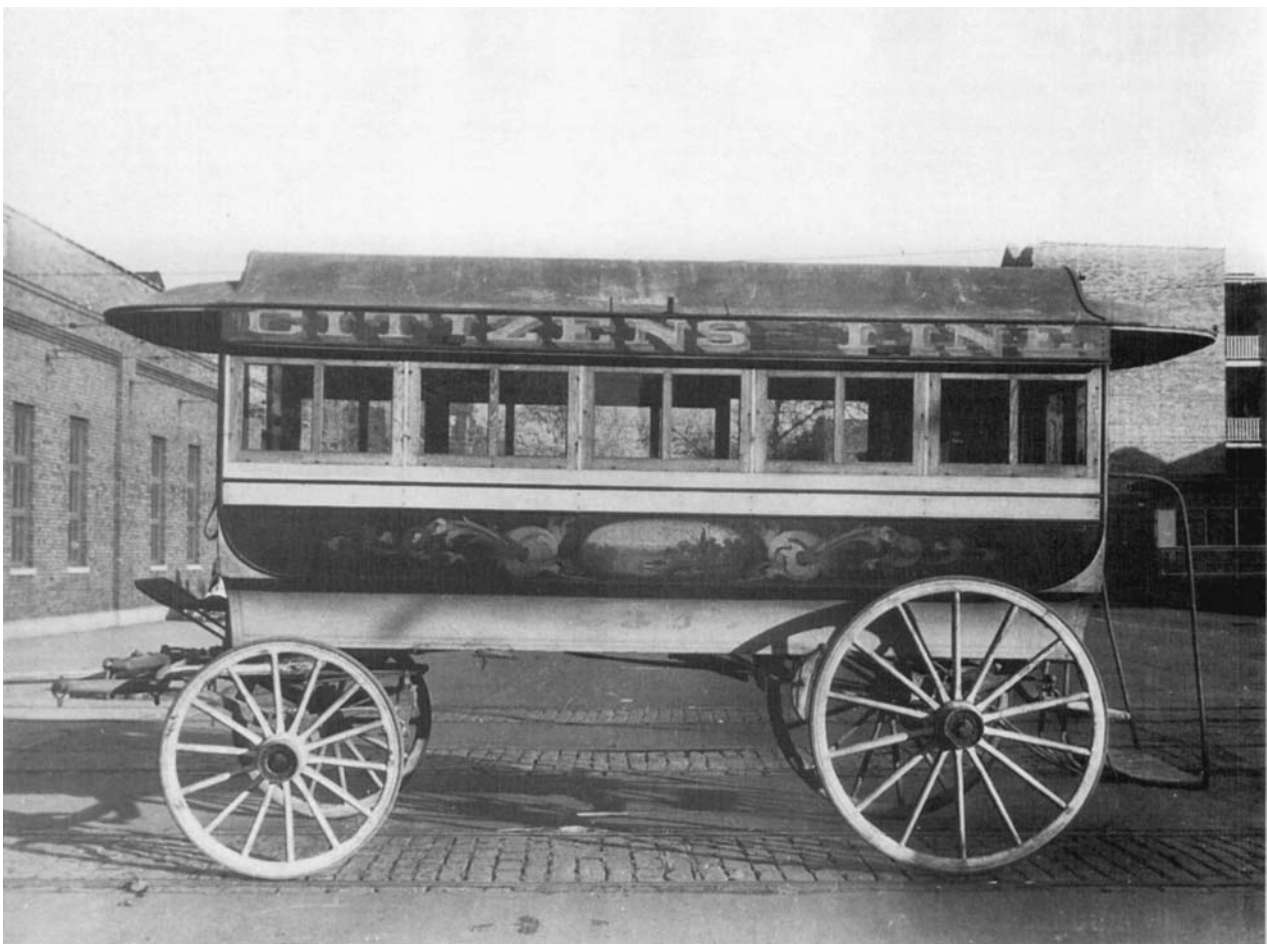
<sup>1</sup> A comprehensive treatise on suburban growth in the United States is Kenneth T. Jackson's *Crabgrass Frontier: The Suburbanization of the United States* (Jackson, 1985). The following has been adapted from Jackson's book.

**TABLE 1.1 Population of Principal Cities 1800–1850 (U.S. Census Bureau)**

LOCATION	1800	1810	1820	1830	1840	1850
Boston, Mass.	24,937	33,250	43,298	61,392	93,383	136,881
New York, N.Y.	60,489	96,373	123,706	202,589	342,710	515,547
Philadelphia, Pa.	69,403	91,874	112,772	161,410	220,443	340,045
Baltimore, Md.	26,114	35,583	62,738	80,625	102,313	169,054
Washington, D.C.	3,210	8,208	13,217	18,827	23,354	40,001
Cincinnati, Oh.	750	2,540	9,642	24,831	46,338	115,436
New Orleans, La.	.....	17,242	27,176	46,310	102,193	116,375

That soon changed with an expanded population and transportation network. Congestion and the need for better, more affordable living accommodations forced people to the periphery of the city, where land was more available and less expensive. In the 1820s, the distinction between city

and country was fading. The evolution of new technologies in transportation changed the character of the city from a walking city to a commuter city. As the outer fringe areas became accessible there was a shift in residential status. The suburbs were no longer perceived as the residence for the



**FIGURE 1.1** An antique omnibus. (Chicago Historical Society)

lower ranks but rather the destination for upper-class families who wanted to move away from the inner city's unattractive squalor.<sup>2</sup>

### The Role of Modern Transportation

The new modes of mass transit began with the omnibus, which was later replaced by the horsecar and eventually superseded by the electric streetcar. Steam ferries were used in cities divided by rivers, and commuter railroads were used for longer-distance commutes. Omnibus mass transit was established in 1829 in New York; Philadelphia's system followed two years later. Boston had one by 1835 and Baltimore in 1844. The omnibus is the urban counterpart of the stagecoach, and although the ride was often rough and unpleasant, it gave rise to a new category of traveler, the commuter.

During this same period, steam railroads came into use. Because steam locomotives required substantially long stopping and starting distances, the suburban developments that sprang up along the tracks had to be several miles apart. High fares associated with commuter trains further limited the migration to outlying suburbs along these rail lines to those who could pay the fares.

The horsecar, a combination of the steam railroad and the omnibus, became popular during the 1850s. Smoother and faster than the omnibus, the horsecar could move at 6–8 mph. It served to push the fringe even further outward from the central city.

With transportation improvements opening new areas for growth, many would-be homeowners and real estate specialists became active in the city-building process, lobbying city governments to extend the infrastructure, pressuring streetcar companies to build tracks to suburban area, and setting the property lines. With municipalities involved in paying for infrastructure, development and the public process became inextricably linked.

By the turn of the century, the electric streetcar had replaced the horsecar. The average speed of travel increased to 10–15 mph. Despite the improvement, the pattern of suburbanization was still linear due to the rails and power lines needed for operation. Although the motor car was invented before the electric streetcar, trolley lines were established in many areas within 10 years of its invention; acceptance of the motor car took much longer due to high vehicular costs, regulations, and lack of adequate roads.

### Enter the Automobile

With the advent of Ford's assembly line in 1914, the price of the Model T dropped from \$950 in 1910 to \$290 in 1924. Automobile registrations increased from 1 million in 1913 to 26 million by 1927. Because people no longer had to wait or walk to rail transportation, suburban develop-

ments were freed from the limitations of the rail lines. Coincident with the boom in automobile ownership, between 1922 and 1929 new homes were begun at the rate of 883,000 per year, a rate more than double that of any previous seven-year period. The automobile increased the amount of developable land by providing access to the vacant land between the rail corridors.

With more land available and traveling convenience significantly improved, densities of suburban developments decreased and lot sizes increased. Areas accessible only to cars were relatively less expensive. New construction techniques such as the balloon frame house revolutionized production and served to lower the cost of a new home significantly. Add to that such other innovations such as power lines and sanitary sewer services and one can understand the burgeoning demand. Builders rushed to take advantage of this new market opportunity.

### Success—Suburban Style

The preference for rural life over the city was characteristic of the middle class of the latter part of the 19th century. The suburban single-family dwelling was viewed by many families as a sign of success and the reward for hard work. The new attitude towards suburban living and the availability of relatively inexpensive housing had an impact on the architectural style of houses as well. Instead of city-like row houses with no yard or the rural farm type with vegetable-herb gardens, manicured lawns and picturesque gardens became the norm.

Early suburban developments reflected the gridiron street patterns of most cities. The system was simple, maximized the number of lots, and was easy to survey. However, new progressive philosophies evolved in the late 19th century. These new ideas were based on preserving natural beauty by providing open space within the development and using curvilinear street patterns. Grandeur was accomplished by wide streets and minimum setback distances of the house from the street. Zoning restrictions were not an issue at the time; therefore, legal covenants in the property deeds set the minimum distances.

### The Beginnings of Land Development Design

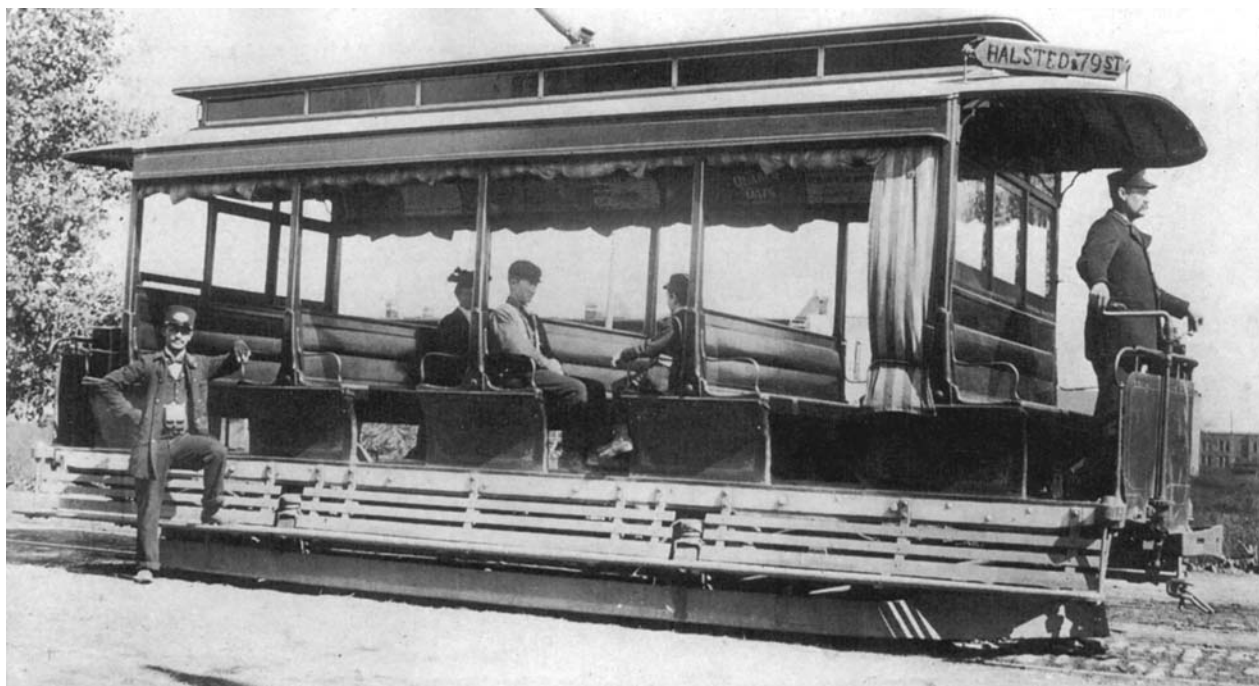
One of the first planned picturesque communities to capitalize on this new trend was the brainchild of Llewellyn S. Haskell and Alexander Jackson Davis. Haskell owned 400 acres in the eastern foothills of New Jersey and employed Davis to prepare the site plan for a development called Llewellyn Park. Davis's layout included two heretofore unheard of features: curvilinear streets and 59 acres of natural open space. Both features took full advantage of the natural landscape. Additional open space was provided by the average lot size of 3 acres.

To enhance even further a pleasant environment, Haskell and Davis stipulated in the covenants that no factory, shop, slaughterhouse, or other place of industry would be permitted in the planned community. Property owners were able to landscape their lawns freely but were encouraged to

<sup>2</sup> Today, in the early part of the 21st century, the outer fringes of a metropolitan city is 20 miles or more. In the early 19th century the outer fringes were only one to two miles from the central business district.



**FIGURE 1.2** Horsecar of 19th century. (Chicago Historical Society)



**FIGURE 1.3** Typical electric streetcar. (Chicago Historical Society)

harmonize their property with the character of the land. Fences were prohibited, however, because they interfered with the natural scenery.

Frederick Law Olmstead was another advocate of maintaining the natural character of land. He employed this technique in his first development, Riverside, a 1600-acre site located outside of Chicago. Together with his partner Calvert Vaux, they molded the development into their conception of a well-planned proper residential district. Riverside included curvilinear streets, generous 100 × 225-ft lots, and such amenities as a lake and a total of 700 acres for parks and recreation of which one was a 160-acre park along the Des Plaines River. Houses were set back 30 ft from the street and homeowners were required to maintain immaculate gardens.

Haskell, Davis, and Olmstead, like their successors at the turn of the 20th century, were not unlike the land developers of recent times. Rarely did a single individual (or firm) buy land, generate the site plan, construct the infrastructure and houses, and then finance sales to the ultimate owners. Typically an engineer or surveyor was hired to design the site. The finished lots were then sold, usually at auction, to buyers who would build the houses for sale or for their own occupancy or hold the finished lots as speculation/investment.

### The End of the First Boom and the Beginning of Federal Financing Assistance

The stock market crash of October 1929 ended the housing boom of the 1920s. Between 1928 and 1933 construction of residential property decreased by 95%. The virtual collapse in the housing industry prompted action by Congress to stabilize the housing industry and the economy. The Emergency Farm Mortgage Act of 1933 was intended to reduce rural foreclosures. The Home Owners Loan Corporation (HOLC) provided low-interest loans for owners to recover homes lost through forced sale. The loans were long-term and self-amortizing with uniform payments spread over the life of the debt. In the 1920s the typical length of a mortgage was 5 to 10 years. The HOLC program increased the repayment period to 20 years.

Another significant action by Congress was the National Housing Act of 1934, which created the Federal Housing Administration (FHA). An FHA secured loan required only about a 10% downpayment, only a third of what saving and load associations required. The repayment period for the guaranteed mortgages increased to 25 to 30 years with loans fully amortized. Additionally, the FHA set minimum construction standards for houses that were mortgaged under the program. The effect on the construction industry was evident by the 332,000 housing starts in 1937. By 1941, housing starts were up to 619,000.

### The Impact of Federal Mortgage Guarantees

World War II forced another slowdown in residential construction. Housing starts declined to 139,000 in 1944. After

**HOMES**  
IN  
**Lake View**

**THE NEW RAILROAD!**

**JUST NORTH OF THE CITY LIMITS!**

WELLINGTON STREET. 10 SOLD, ONLY 9 LEFT. PRICES, \$600 TO \$650. GRAVELED STREET, 60 FEET WIDE.

Proposed Early Extension of Street Cars.

Go and see the **HOUSES** being built.

**OUTSIDE FIRE LIMITS!**  
You Can Build Wooden Houses!  
**NO CITY TAXES!**

**GO AND SEE THEM.**  
**TAKE THE LINCOLN AVE. STREET CARS**  
Only 3 Blocks from present terminus of Street Cars. One of the most pleasant rides in the city, passing in front of Lincoln Park & along a finely shaded avenue, and away from the streets leading to the cemeteries.

**S. E. GROSS & CO.,**  
Subdividers & Owners of 10 NEW SUBDIVISIONS in and around the City.  
136 SOUTH CLARK ST. Corner of Madison St.  
PARTICULAR NOTICE—Our Agent will be at our Branch Office on the Lots for information week days and Sunday.  
See Other Side For Prices and Further Description.

**FIGURE 1.4** Advertisement for a streetcar subdivision. (Chicago Historical Society)

the war, marriages and birth rates increased dramatically, creating a high demand for affordable housing. The Servicemen's Readjustment Act of 1944 created a Veterans Administration mortgage program similar to FHA. The assurance of federal mortgage guarantees resulted in housing starts to skyrocket to 1,015,000 in 1946. By 1949 housing starts had reached an all-time high of 1,430,000.

To meet this demand for housing, designers developed innovative mass production techniques. The Levitt family did for the housing industry what Henry Ford did for the automobile industry three decades earlier. The development of Levittown on Long Island transformed 4000 acres of potato farm into 17,400 dwellings. This massive undertaking, the biggest private housing project in the United States, began in 1946.

### Back to the Future—Where Do We Go from Here?

The land development industry is the direct result of the ongoing need for housing and services as communities expand their borders and the population pursues the American Dream. Suburbanization in the 19th and early part of the 20th century evolved as a result of a combination of factors such as increased population, the need for affordable

housing, and the innate American desire to own one's own castle rather than rent. Science and technology in the form of new types of transportation, building materials, and innovative construction methods lowered the cost of housing. Together with innovative financing programs, these advancements helped more and more middle class Americans achieve that dream.

The challenge for those of us in land development is to understand the factors contributing to the demand for growth and expansion and be able to remain flexible enough to respond to the changing needs of the market. Part of that understanding includes knowing where the industry has been and how it evolved into the practices and procedures of today. The other part is understanding the nature of the land development industry and how to maintain the standards of quality, flexibility, and value that we have attained. Design professionals must meet the challenges of today while not losing sight of yesterday's lessons and today's high standards.

It used to be that a home builder depended largely on a surveyor to lay out the lots, and the local city or town to extend utilities and streets into the subdivision. That is no longer the case.

Following World War II and the Korean War, there has been a continuous and large demand for housing, caused by retiring veterans, rapid migration to our cities, smaller families and changing life-styles. This demand has required a revolutionary approach to financing of all facilities connected with housing. With government leadership and subsidies, huge mortgages with minimum down payments have been made available to Americans of all income levels. The large amount of capital generated by this process has revolutionized home construction and ownership in America in the past 30 years. Other major public works programs, such as the interstate highway system, mass transit, pollution control, and clean water acts, are all small in comparison to the billions which go into housing every year. While the social changes of these public works programs are vast and historical, home building and related construction has been the largest single industry in the United States for at least three decades (Dewberry, 1979).

## THE LAND DEVELOPMENT PROCESS

### Overview of the Land Design Process

This book is, in its entirety, an overview of the land design process as it applies to engineering, planning, and surveying. The engineer, planner, and surveyor are an intimate part of the development team. One or more of them is usually the first on the site and the last to leave after completion. They help guide and direct the process from start to finish. As an aid in understanding the overall land design process in as few words as possible and without having to peruse this lengthy book, consider a hypothetical project.

Imagine there is a developer or owner who wishes to construct and sell residential houses in a certain price range in a certain region. He may initially contact a real estate

broker, describe his proposed project and ask the broker to find him a suitable piece of land. More likely, a real estate broker will contact the developer and offer a certain piece of land that might support the style, amenities, and overall goals of the developer's particular project. The developer is usually experienced, either having been in business several years or having been employed by another developer where he gained the relevant experience. The first thing the developer will do, if he is not already familiar with the neighborhood, is visit the site and familiarize himself with the piece of land. On first inspection, does it appeal to the eye and appear worth pursuing? Is the proposed sales price comparable to the existing neighborhood? Utilities available? What is the zoning? Is the asking price in the ballpark? These are the first in a host of other very preliminary questions. If the preliminary inspection proves promising, the next step is to employ an engineer to do a feasibility study.

This feasibility study can be a one- or two-hour visit with the engineer or a 45- to 60-day process. If the engineer is familiar with the general area and is able to answer sufficient questions, the developer may be able to proceed with negotiations to purchase the property within a relatively short period, since a detailed analysis of the site can be performed in several days or weeks.

The experienced developer will usually perform a brief analysis of the property that addresses essential items and then proceed immediately to get the property under contract, with the final purchase contingent on a number of items. Speed is often critical to get the property under contract to prevent a competitor developer from gaining control.

During this study period, the developer is very busy doing a complete analysis for the property. This analysis could include a market survey, financing options, sales potential, *pro forma* financial models of the proposed project, and, not least, a detailed engineering feasibility study. There is usually a limited number of days to do this feasibility study (from 15 to 45 days is quite common), during which time the purchaser can release his option to purchase the property for any number of reasons.

During this engineering feasibility study, detailed information must be developed. What is the zoning? Can the developer reasonably expect to be able to get the desired rezoning? How many lots will the project yield? What is the cost of providing the needed infrastructure to the site, both on-site and off-site? What is the history of support or opposition to similar projects by citizens/neighbors in this area?

After the developer has satisfied himself on all of the questions, he may then proceed with the contract. If the feasibility study or other analysis indicates the potential for problems, renegotiations may be necessary. Oftentimes during a feasibility study period, items are discovered that will cause the developer to drop the contract or renegotiate the terms based on the new information.

The next step in the process usually involves hiring a surveyor to do a boundary and topographic survey. The boundary survey is required to transfer title, while the topography survey is required to accomplish design.

The planner, whether it be the surveyor, engineer, architect, or landscape architect, will then proceed to do a schematic design of the proposed project working closely with the developer and all members of the team.

If rezoning is involved, the developer must mobilize his entire team and develop a winning strategy. Chapter 11 in this book touches on this subject in detail.

Once rezoning has been obtained, the next step is preparation of a detailed preliminary plan. The schematic developed earlier or during the rezoning process is usually the basis for this plan, with more details of such things as sewer and water connection required to be shown. Other details to be addressed include. How is storm runoff to be handled? What about erosion and sedimentation control, access, turning lanes, right turn lanes; building set backs, lot sizes, and treatment of soils? All of these questions and more are related to the preparation of the final plans are addressed in the detailed preliminary plans.

The preliminary plans must then be processed through all the affected agencies and approval sought. Most of the time revisions are required before approval is granted. These requested revisions are sometimes mandatory but more often are negotiable. It is here that the skilled consultant can make a big difference by successfully managing revisions before the project reaches final design. Sometimes it is necessary to present this preliminary plan, once again, to the affected citizens even though they may have been involved in the rezoning. This aids in gaining community support and once again should be completed prior to embarking on final design.

Once all required approvals of the preliminary plan have been obtained, preparation of detailed construction plans can proceed. There may be several separate different plans, or all may be included on one comprehensive set of drawings. Often water and/or sewer systems must be engineered separately and reviewed and approved by that particular agency or authority. This same procedure will likely occur during the design of the streets and drainage. Several chapters in this book are devoted to detailed preparation of these final plans.

Concurrent to the engineer preparing these final plans, the architect may be preparing detailed architectural plans for review and approval, on a parallel path, by the agencies. Once the final plans have been approved, the developer must post a bond guaranteeing the work will be completed in accordance with the approved plans (see Chapter 37). The engineer usually assists the developer in preparing cost estimates for budget and bonding purposes. Frequently the developer will engage the engineer to oversee selection of a contractor or contractors to perform the construction. This may be through a closed-bid process or negotiated basis.

The next step is construction. The surveyor is involved in giving lines and grades to the contractor to make sure that the completed project complies with the intent of the final construction drawings. Sometimes the engineer will provide on-site inspection to ensure that specifications are fully honored, although in most jurisdictions the developer must pay a permit fee that covers the cost of inspection. The inspection or inspections are then performed by the jurisdictions' own inspectors.

In residential development, model homes are used to show potential buyers the types of floorplans and upgrades available within the community. In some cases, floor plans and rendered drawings are used as the only sales tool. Developers and lenders do not like construction to get very far ahead of sales.

The final step, after all construction of the infrastructure has been completed and the streets have all been paved, is to conduct an as-built survey and set final property corners. Final inspections and the engineer's certification are the last official actions in the process.

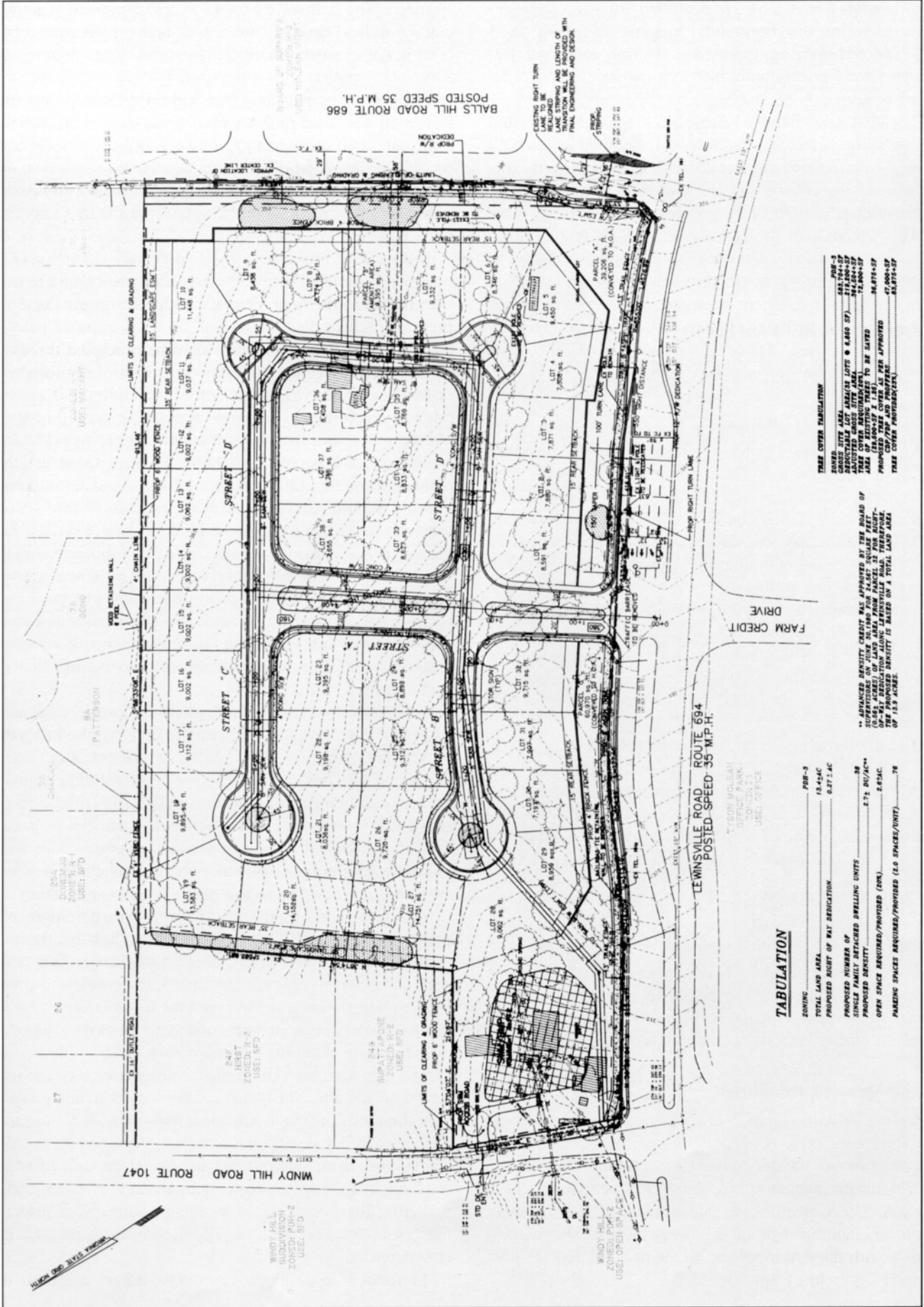
This overview of the land design process is of course simplified for ease of communicating what the entire process is. It is meant to be an overview. The details of the specific technical aspects can be found in the appropriate chapters.

### Communication Skills

No discussion of the land development process would be complete without addressing the critical need for solid communication skills.

Overlooked and unemphasized in land development are good communication skills. The process requires members of the team to be in constant communication with each other, with approval agencies, and with citizens. A project undergoes many changes between its inception and completion. These changes occur very rapidly and for many reasons. It is imperative to communicate the changes and updates to the proper people at the proper time. To do this effectively, one must ultimately know not only what has transpired technically but who should know what facts and what actions should follow. The ability to communicate effectively through letter writing, report writing, and technical writing is a very important credential for the land development professional.

Equally important is the ability to present your ideas clearly and precisely. Good public speaking skills before small and large groups are important in many professions but are particularly important in land development, where presentations to public approval agencies often make or break a project. This includes skills in adapting material for a technical audience, nontechnical group, or a mixture of the two. Accuracy should be unquestioned and enthusiasm a key ingredient. As always, respect for the time available is essential. Where appropriate, the use of humor can go a long way in easing tensions and building the relationship necessary to ensure the project receives a fair hearing. The



BALLS HILL ROAD ROUTE 686  
POSTED SPEED 35 M.P.H.

LEWYNVILLE ROAD ROUTE 694  
POSTED SPEED 35 M.P.H.

**TABULATION**

ZONING	PER-3
TOTAL LAND AREA	13.42AC
PROPOSED RIGHT OF WAY DEDICATION	0.271AC
PROPOSED NUMBER OF UNFINISHED BRILLING UNITS	38
PROPOSED DENSITY	2.72 DU/AC**
OPEN SPACE REQUIRED/PROVIDED (20%)	2.63AC
PARKING SPACES REQUIRED/PROVIDED (LO SPACES/UNIT)	76

**TREE COVER TABULATION**

ROSES	806-5
GROSS SITE AREA	583,704-SF
ADULTED GROSS SITE AREA	34,500-SF
ADULTED GROSS SITE AREA (CONVERTED TO H.O.A.)	73,900-SF
TREE COVER REQUIREMENTS TO BE MAINTAINED	36,876-SF
ADULTED GROSS SITE AREA (CONVERTED TO H.O.A.)	47,000-SF
TREE COVER PROTECTED	82,876-SF

\*\* ADVANCED DENSITY CREDIT WAS APPROVED BY THE BOARD OF ZONING ADJUSTMENTS (0.645 ACRES) OF LAND AREA FROM PARCEL 33 FOR RIGHT OF WAY DEDICATION ALONG LEWYNVILLE ROAD. THEREFORE, THE APPROVED DENSITY IS BASED ON A TOTAL LAND AREA OF 13.3 ACRES.

**FIGURE 1.5** Example: detailed preliminary plan.



ability to communicate effectively whatever the media is the mark of a leader and an essential quality for today's land development professional.

### Public Involvement

In today's land development practice, a working knowledge of the public process is essential for success. Public agencies are really part of the design team. Even though the relationship is mostly regulatory, in many jurisdictions they have the power to shape projects, deny applications and grant approvals. The smart designer will establish an early rapport with all agencies from which he or she will require later approval. The best way to establish this rapport is to understand their regulations thoroughly and submit plans that comply with these regulations, plans that are clear, easy to read, and complete. Resist the urge to avoid compliance to reduce costs or save time. This will only result in delays, which will result in turn in lost time and dollars. If you disagree with the regulation and have a good valid reason, or if the regulation simply does not apply under the development conditions you face, waivers or variances can often be obtained by confronting the issue squarely and working with the approval agencies to obtain a solution.

Involvement by public agencies is pervasive and must be understood thoroughly by the land designers, as there are many agencies involved, often with conflicting agendas. Approvals must be obtained from all involved agencies before the project can proceed. Compliance with applicable rules and regulations is often required by regulations, ordinances, or local, state, or federal law. A brief description of representative agencies and selected rules is included throughout the book but particular Chapters 2, 3, and 5 of the Planning Section and Chapter 12 of the Engineering Section.

The nature of public agency involvement varies greatly from jurisdiction to jurisdiction and agency to agency. Federal regulations, however, are reasonably consistent. Each state has its own set of rules that are dissimilar to those of other states. Even within a state, each county, town, and city can be different. Areas that are more urban and suburban in nature generally have a more detailed set of regulations. Sometimes regional authorities have jurisdiction for such services as sewer or water. It is imperative that the land designer thoroughly understand the rules of all the agencies having jurisdiction over a project.

The length of time involved in actual planning, engineering, and surveying is short compared to the length of time required to gain all the necessary approvals. Many politicians have been elected on platforms pledging to cut and streamline regulations, not only for private projects but also for public projects such as highways and utilities. Regardless of these good intentions, the regulatory process continues to expand as new regulations and agencies that enforce them continue to be created.

The costs for land design and final approvals have changed dramatically since the days of Levittown. Just as contemporary humor labels the federal tax code a welfare

program for tax lawyers and accountants, the regulatory environment surrounding land development has created a large cadre of experts, both in and outside the agencies. Forty years ago, for a typical suburban house, the cost of engineering, planning, and surveying, including fees paid to agencies, was about the cost of a refrigerator. Today it is probably greater than the total cost of all of kitchen and other electronic appliances combined. Another amazing phenomenon is that the fees paid to various agencies, just for review and approval of plans, frequently exceed the fees charged by consultants preparing the plans! This does not include special assessments for sewer and water hookups, school assessments, transportation, and so on.

Citizens have become much more involved in the process of approvals for a project. Citizens are very skillful in opposing projects they do not want. They often have experts of their own, skilled in the complexity of land designs and versed in the myriad of regulations a developer must handle in order to gain final approval. These regulations are open to differing interpretation by different experts, and it is very common for petitions to be made to the courts for final resolutions.

It is important to establish early relations with citizens who many be interested in the development of a particular piece of land. Early participation by these citizens can usually lead to modifications in a plan that will be acceptable to both the citizens and developer. It is not always what you do but how you do it that is all important to affected neighbors. Many things can be offered citizens or neighbors that will make the project more amenable to them. Discussions of this are covered throughout the book, but more particular in Chapter 3 of the Planning Section.

### BASIS FOR DESIGN

"Quality design" is a much used and many times misused phrase. In today's world, quality design is much more than the ability to "determine precisely how best to develop our program on the site selected, in specific forms and materials." ECKBO 1969, 11. Quality design is a result of the design team's ability to produce a plan that not only conforms to the client's established development program, goals, and objectives, recognized site constraints, laws, ordinances, regulations/policies, accepted design standards, and market considerations but has also withstood the test of private and public special interest groups' scrutiny.

At the risk of oversimplification, the prerequisites for undertaking quality site design are predicated on a designer's level of familiarity and dexterity in dealing with a core of base information that is required throughout the process. The information base may be loosely categorized under the following major topics: *Development Program*, *Site*, *Planning and Regulatory Controls*, and *Design Team*.

### Development Program

The development program is initially a determination on the part of the client/owner as to what type of development

is expected for a given parcel of land. For the most part, such preliminary determinations are based on a cursory review of zoning, planning, and market considerations as applied to a specific property. This development program concept is presented to the project design team for discussion and refinement. The challenge of project design is formulating a response that simultaneously seeks to balance the highest and best land use with the character of the site and its environs, client and consumer expectations, economic and marketing factors, and public/private approval requirements.

Land uses and their associated building types need to be consistent with current construction practices and consumer and user requirements. Market conditions, development costs, and numerous alternatives in development technologies afford the designer the opportunity to develop distinctly different designs for any given property. Design should be predicated upon a thorough understanding and appreciation of the success associated with previous land development designs. Such awareness strengthens the posture of the land development designer. The intent is not to mimic that which has proven to be successful, but rather to gain an understanding as to the reasons for success and expand on those attributes. The fundamental requirement of land design rests in a working knowledge of the physical/functional characteristics and locational constraints associated with specific building product types. While certain base considerations, such as adequate vehicular access, represent a common requirement to all land use types, the appropriate design response varies substantially as one proceeds from detailing low density single family residences to the more complex urban mixed use development.

### **The Site**

The site, that piece of real estate on which a development program will be implemented, affords a special set of resources and opportunities for project design. Each site is unique and requires an understanding of and appreciation for the specific characteristics to elicit a tailored design response. Consideration should be inclusive of both surface and subsurface characteristics, as well as the dynamics associated with the natural and cultural processes which it makes unique. Attributes of a site normally considered relevant to land development activity include those that bear on the land's ability to absorb specific development program elements. These include both on-site and off-site considerations. Site considerations brought to the design table range from site configuration to adjacent land uses and are discussed in more detail throughout this book.

### **Regulatory Controls**

Knowledge of the public regulatory controls, design standards, and technical requirements associated with land development, as well as preexisting legal requirements or development conditions associated with a given property, is necessary to successful project design. Site design can not

commence without a thorough knowledge of the appropriate regulatory ground rules. Issues relating to local comprehensive plans, zoning, and subdivision ordinances and other regulatory controls are first analyzed during the feasibility and programming stage of the design process and remain an influence until final site plan approval.

Other regulations and legal requirements have varying degrees of impact on site design. Restrictive covenants that have been placed on a given property may dictate a site design with standards more stringent than those of state or local jurisdictions. Existing planned developments may mandate that the designer work within previously established design guidelines. Further, special development conditions may have been agreed to or imposed upon the land at a previous time in the development review and approval process. Such conditions may direct or influence design on a particular site in a manner atypical of other sites with similar uses.

Finally, statutes and policies of state and federal government provide regulation of such items as wetlands, coastal zones, hazardous waste, air and water quality, noise and handicapped accessibility, to cite a few. These considerations, coupled with varying building code requirements relative to building access and spacing, can have a significant influence by creating site design constraints and opportunities.

It is imperative that the designer bring as much information to the design effort as possible. All appropriate rules and regulations that may affect the design aspects of a specific project must be understood at the outset of the design process. While manipulation of select site characteristics and development program components may allow a degree of interpretation and flexibility, statutory and/or legal requirements are not as forgiving. The ultimate test of a successful design effort is whether it can be approved by the appropriate public agencies and constructed.

### **Design Team**

Who are these experts following in the footsteps of Haskell, Davis, and Olmstead? Oftentimes the key player for land design is the civil engineer and/or land surveyor. This is especially true for smaller projects and in less urbanized areas. Originally, the engineer and/or surveyor played an overall role. However, today, the story is a bit different. The acceleration of American urbanization, with an increasingly complex and regulated planning and design process wherein citizens are deeply involved, has resulted in many players being involved in the process and on the design team.

In a typical project, the land development team might consist of any of the following:

1. Client
2. Financial institution

3. Land surveyor
4. Civil engineer
5. Landscape architect
6. Architect
7. Project designer
8. Attorney—land use and other
9. Urban planner
10. Transportation planner
11. Environmental specialists
12. Market analyst/researcher
13. Real estate brokers
14. Real estate specialist
15. Economist
16. Public approved agencies
17. Citizens
18. Sociologists, recreational specialists, cultural and education specialist, other engineers, architects, etc.
19. Geotechnical engineer/geologist
20. Structural engineer
21. Archeologist

Land development has become a very complex industry, and the design team requires a comprehensive understanding of it. Persons involved in this undertaking have had to become specialists, but to participate in the overall process, the specialists should have a broad knowledge of the popular terms and principles used in the industry. It is mandatory that certain of the essential professionals have an overall knowledge of all facets of the process. These would certainly include the planners, engineers, and surveyors, as well as the land use attorney and, of course, the client.

Others who play various roles, depending on the project, include the financial institution providing the financing, real estate specialists, geotechnical engineers/geologists, structural engineers, archaeologists, sociologists, recreational specialists, cultural and education specialists, and, of course, architects. We will confine our discussion here to a few, but others are elaborated on throughout this book.

**Client.** While the land development client may be the end developer or builder, it is not unusual for the designer to operate under the general direction of a representative of a major corporate, institutional, or financial interest. As a key to establishing a successful working relationship, a designer must understand his client's familiarity and past experience in land development and project design. In addition, the client's knowledge of the development activity within the subject jurisdiction and his expectations of the design team

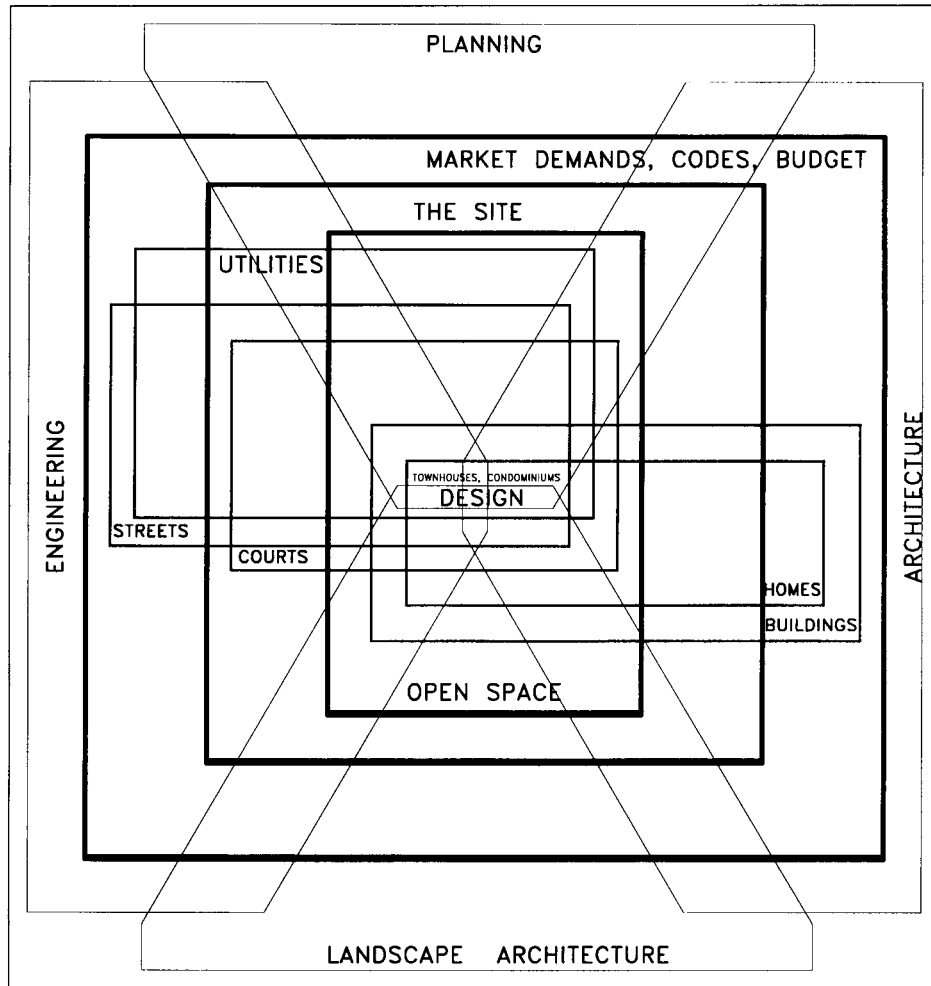
are important. The client's degree of familiarity with project design and the land development process may have significant bearing on the latitude extended to their design consultants. It is prudent to confirm what services are expected from the design consultant at the beginning of the process to minimize the opportunity of false expectations. Frequently, several individuals within an organization may represent the client interests. If the client is to be represented by others, it is important to establish an appreciation of each member's role and responsibilities, particularly in terms of ascertaining who will ultimately make the decisions that affect the project design.

**Project Designer.** Generally the principal site designer/planner has been schooled in the physical design aspects of land planning, with education and/or training in either landscape architecture, architecture, urban design, civil engineering, or a related discipline. While the project designer is responsible for the actual preparation of a design response, it is generally done in close cooperation with other members of the client design team. The core design team may be composed of professionals from a single firm offering multiple discipline services or representatives of different firms providing specialty design or engineering consultation.

In addition to having knowledge of the physical design aspects, the designer must also understand and be responsive to the client's likes and dislikes in accomplishing the established development program. The client is key, however, even when the client program is more "seat-of-the-pants" than a reflection of sound planning, zoning, and design principles. It is, in part, the designer's role to respond tactfully to the initial proposed development program with a plan that conforms to planning and zoning regulations, is marketable design, and meets the client's needs.

**Attorney/Legal Counsel.** The complexity of regulatory controls and public administrative procedures has prompted an increase in the participation of legal counsel in the land development process. While they may be primarily responsible for the preparation and review of documents associated with legal or procedural requirements, they may have a potentially significant role in project design. First, by virtue of their prior exposure to a variety of design efforts, they may have knowledge of comparable projects that have been successful in securing public approval. Secondly, depending on their personal relationship with the client, they may be positioned to influence decision-making during the project design process.

**Engineers.** Civil engineers have a very important technical role in the project design and site planning process. In the early stages of design, civil engineers provide valuable information in terms of the location, routing, and sizing of various site infrastructure, including street improvements, stormwater management facilities, sanitary sewerage, water systems, and other utilities. The engineer's involvement in the early design stages is important because technical de-



**FIGURE 1.6** Overlapping disciplines of the design team, etc. (Urban Land Institute 1979)

isions will be carried through to final site plan design and ultimately certified by a registered professional engineer. Given their typical responsibility in the preparation of the final site plans and construction documents, their continuing participation in the design process is essential.

**Surveyors.** Prior to about 1950, the surveyor was the key player in laying out a standard subdivision. The suburbs were still largely rural, and development, which pushed out from the fringes of the urban core, was usually a matter of laying out lots, cutting in the streets, installing roadside ditches, and some cross-draining of culverts. As development intensified and new regulations for better streets, clean water, and sewer systems were promulgated, surveyors began to become civil engineers or to add civil engineers to their staff. The basic services of the land surveyor are still very important to the overall success of the project. It is the surveyor who provides the boundary survey and topographic information that the designers must have in order to begin. Not only is the exact computation for street and lot alignment the job of the surveyor, but so is the final horizontal and vertical alignment of all the infrastructure and buildings to ensure that the specifications of the de-

signer and civil engineer are met. It has often been said that the surveyor is the first and the last on the job as he sets the final permanent monumentation and prepares the “as-builts” when the project is completed.

**Urban Planners.** Given the increased complexity of local zoning and planning documents, the urban planner’s role in the design process has increased notably. Based on knowledge of the local zoning/planning requirements, the urban planner is in a position to guide the design team by interpreting the impact of governmental regulatory requirements on the design process. Often, the urban planner will team up with the project attorney to act as a front-end team to carry the design documents through the public review process. Due to his or her involvement in local planning and zoning initiatives, often the urban planner is able to cultivate a positive relationship with administrative staff and elected officials. This relationship is important to the design team as the urban planner gains access to staff and elected officials to promote project advocacy.

**Transportation Planners.** Concern about the impact of new development on the existing vehicular transportation networks has made transportation planners an integral

member of the design team in many communities. This has been prompted by requirements that proposed development activity be subjected to a rigorous assessment of projected traffic impacts and documentation of associated roadway improvements necessary to sustain a acceptable level of service. The nature and extent of required on- and off-site road improvements varies with the size and specifics of each project. Suffice it to say that there is financial and design coordination associated with such required improvements that may have significant consequences in developing the appropriate design response.

**Environmental Specialists.** Some level of environmental assessment is usually required as part of land development activity. Historians, archaeologists, botanists, acoustical specialists, arborists, geologists, hydrologists, and other environmental scientists have increasingly become participants in the design and development process. Their participation may occur throughout the design process, commencing at preliminary site investigation and extending through to final design with the formulation of measures to mitigate the impacts of development. The environmental scientist's role varies from project to project. However, given the increased emphasis on protecting the environment through preservation, conservation, buffering, and other mitigation measures, their role is important.

**Market Analyst/Researcher.** Market analysts often provide varying levels of feasibility studies that assist in the formulation of project-specific building programs. Their role in project design can range from actually determining and describing the details of the development program to providing guidance in locating uses on the site. The importance of this discipline varies with the proposed use or development program. A market study may be required to obtain development financing in addition to providing the client/designer with advice relating to the established development program.

**Public Review/Approval Agents/Citizens.** Representatives of the public interest have a significant role in project design. While jurisdictional regulations establish the ground rules, required public agency review and interpretation of public policy and performance criteria, as well as citizen concerns, have a significant influence on many aspects of project design.

## TRADITIONAL STEPS IN THE LAND DEVELOPMENT PROJECT DESIGN PROCESS

Land development design has traditionally been composed of several distinct steps or stages leading to the final plan. While structured to be orderly and sequential, like other problem-solving endeavors it seldom follows a straight-line path to solution. It requires sufficient overlap in the performance of select tasks to ensure the timely availability of relevant information. Based on a sequence of activities traditionally characteristic of the design professions of architecture and landscape architecture, the land development project design process and resultant products must be sensitive to both client decision-making and the general sub-

mission requirements associated with routine public review and approvals.

The level of detail and sequence required for submission documents for public review may vary somewhat from jurisdiction to jurisdiction, and different terminology may be used in identifying the documents. However, the traditional sequence of design resolution includes the following stages of evolution of a project design, all of which will be elaborated upon later in this chapter.

### Feasibility and Programming

This initial step in the process requires an understanding of the proposed development program and an overview of the site characteristics and surrounding area. The base objective of the feasibility and programming stage is to become familiar with existing site conditions, the jurisdiction's comprehensive plan recommendations, and local zoning ordinances and other regulatory and/or physical elements of the property that may influence the initial proposed development program.

### Site Analysis

A site analysis is conducted to provide the designer and the entire design team with a complete understanding of the opportunities and constraints associated with a property. The physical characteristics, including site configuration, topography, soils, hydrology, utility availability, and adjacent land uses, are evaluated in the context of the client's proposed development plan. The results of the site analysis also may offer opportunities to identify alternative development programs for review by the client.

### Conceptual Design

The objective of the conceptual design is to establish a preliminary framework depicting the distribution, organization, and arrangement of the development program in a manner that honors the development constraints and takes advantage of the opportunities identified in the site analysis stage. The resultant sketch plan or plans may include alternative strategies that illustrate possible arrangements of principal land uses and infrastructure requirements. This exploratory stage deals with general distribution of uses. The sketch plan or plans at this stage are generally "blob" diagrams that identify usable area, type of development, points of ingress and egress, site circulation patterns, and major infrastructure facility that may be required. This stage in the process is based on a "what-if" mindset and proceeds to explore alternatives to assist in formalizing the preferred project design.

### Schematic Design

This level of project design is a refinement of the selected conceptual studies that provides more precise scale and site detail of program components and supporting site improvements. The detail included in the schematic design is based, in part, on information obtained during the feasibility and site analysis stages and provides further assurance that the

development program and goals can be achieved. Included in the schematic design is a site layout, which details and dimensions the arrangement of program components. The site layout should confirm that the development plan is consistent with the goals and objectives established by the client and conform to all regulatory requirements.

An important element of schematic design is preliminary engineering. The purpose of the preliminary engineering analysis is to verify and document the technical aspects of the schematic design. The resultant of this study usually is in the form of a graphic such as a preliminary site plan or rezoning development plan. Checklists and/or reports are often prepared as well. These documents represent a final check of the development program just prior to proceeding with more detailed final engineering.

### Final Design

After the client and local governing agencies have reviewed and accepted the schematic design and preliminary engineering study, the civil engineers prepare the final design. The final design reflects the detail necessary for project review and approval by local governing agencies, which authorizes the final commencement of construction. The site plan developed during the final design represents the final documentation of the land development design process. The main components of final design are suburban street design, storm drainage design, design of stormwater management facilities, floodplain studies, grading and earthwork, wastewater collection, water distribution, wastewater treatment, water treatment, and erosion and sediment control. Contract documents, construction specifications, and cost estimating are also elements of the final design program.

### DESIGN: UNDERSTANDING OF BASIC REQUIREMENTS

The amount of reference material focusing on land development and specifically project design is extraordinary. Similar is the case with the volume of resource materials, which more singularly focus on specific development and building prototypes such as residential, commercial, industrial, office, recreation, mixed use, planned communities, waterfront, and golf course developments. Basic to the success of project design is the need for the designer to have an appreciation for the concepts and standards identified in that body of information. A design response premised on anything less must be recognized and valued as the technical solution it is. Historically, land development is steeped in technical solutions. They satisfy a multiplicity of functional and regulatory requirements inherent to site engineering and ultimately program constructability. However, they do not necessarily address the environmental, social, sensory, or visual dimensions, which are fundamental components of the built environment. Design solutions need to be based in a sensitivity to basic sociocultural, physical, economic, and political concerns while reflecting the importance of economic and marketing constraints. In private sector land development activity, a design must be capable

of being constructed and must provide a financial incentive to warrant its undertaking. No one profession possesses a monopoly on the diverse body of knowledge and resources that is required to achieve quality land development design. Land development is a process that is dependent on diverse disciplines and an extraordinary commitment to promote all aspects of the project with equal fervor.

### Design Elements

Every development program is composed of elements that define, shape, and establish the essence of that use. The constituent parts include both the physical—dimensional—building blocks that house principal activities and the ancillary or support elements, which are necessary to sustain the principal use. The former are the major space-taking elements that characterize a land use and its related building components. For example, the dwelling unit is the principal building module in residential development. The accessory uses include such considerations as connection to vehicular or pedestrian circulation systems and utility requirements necessary to maintain a certain quality of life, such as water, sewer, power, and light. Collectively, they constitute an operational whole. Project design must address all of these elements. There may be a number of ways to orchestrate a design that satisfies some of the basic requirements associated with a given land use or product type. However, the successful response seeks to reduce any conflict with program objectives and optimizes the relationship between all component parts. This approach applies to large-scale and mixed-use projects as well. The manner in which a site design response addresses these requirements should be a result of a conscious decision and not an insensitivity or ignoring of any component or relationship.

Project design requires an ability to understand the relative needs and physical attributes of the program components. More homogeneous uses at lesser densities or intensities are generally more easily dealt with than mixed-use programs at higher densities. As an example, there is significantly more flexibility in siting a single-family residence on a large lot than there is for more dense residential projects. The challenge of site design rests in both a knowledge of the requirements associated with a given land use or building type and an ability to make valid judgments and establish priorities as to which requirements should take precedent in formulating the design response.

### REFERENCES

- Dewberry, Sidney O. 1979. "The Firm's Direction," *DND Newsletter*, No. 1, p. 1.
- Jackson, Kenneth T. 1985. *Crabgrass Frontier: The Suburbanization of the United States*, Oxford University Press, New York.

### ADDITIONAL READING

- Booth, Norman K., *Basic Elements of Landscape Architectural Design*, Elsevier Science, New York, 1985.

- Chudacoff, Howard P., and Judith E. Smith, *The Evolution of American Urban Society*, 3d ed. Prentice Hall, Englewood Cliffs, N.J., 1988.
- De Chiara, Joseph, and Lee E. Koppelman, *Site Planning Standards*, McGraw-Hill, New York, 1978.
- Eckbo, Garrett, *The Landscape We See*, McGraw-Hill, New York, 1969.
- Engstrom, Robert, and Marc Putman, *Planning and Design of Townhomes and Condominiums*, Urban Land Institute, Washington, D.C., 1979.
- Gold, Seymour M., *Recreation Planning and Design*, McGraw-Hill, New York, 1980.
- Jensen, David R., *Zero Lot Line Housing*, Urban Land Institute, Washington, D.C., 1981.
- Lynch, Kevin, and Gary Hack, *Site Planning*, MIT Press, Cambridge, Mass., 1984.
- National Association of Home Builders (NAHB), *Cost Effective Site Planning: Single Family Development*, NAHB, Washington, D.C., 1982.
- National Association of Home Builders (NAHB), *Land Development 2*, NAHB, Washington, D.C., 1981.
- Simonds, John Ormsbee, *Landscape Architecture: A Manual of Site Planning and Design*, McGraw-Hill, New York, 1983.
- Tomioka, Seishiro, and Ellen Miller Tomioka, *Planned Unit Developments: Design and Regional Impact*, John Wiley & Sons, New York, 1984.
- Torre, L. Azeo, *Waterfront Development*, Van Nostrand Reinhold, New York, 1989.
- Untermann, Richard, and Robert Small, *Site Planning for Cluster Housing*, Van Nostrand Reinhold, New York, 1977.
- Urban Land Institute (ULI), *The Community Builders Handbook*, ULI, Washington, D.C., 1968.
- Urban Land Institute (ULI), *Industrial Development Handbook*, ULI, Washington, D.C., 1975.
- Urban Land Institute (ULI), *Land Use in Transition*, ULI, Washington, D.C., 1993.
- Urban Land Institute (ULI), *Office Development Handbook*, ULI, Washington, D.C., 1982.
- Urban Land Institute (ULI), *Project Infrastructure Development Handbook*, ULI, Washington, D.C., 1989.
- Urban Land Institute (ULI), *Recreational Development Handbook*, ULI, Washington, D.C., 1981.
- Urban Land Institute (ULI), *Residential Development Handbook*, ULI, Washington, D.C., 1990.
- Urban Land Institute (ULI), *Shopping Center Development Handbook*, ULI, Washington, D.C., 1977.
- Urban Land Institute (ULI), *Working with the Community: A Developer's Guide*, ULI, Washington, D.C., 1985.
- Warner, Sam Bass, *Streetcar Suburbs: The Process of Growth in Boston (1870–1900)*, Harvard University Press, Cambridge, Mass., 1978.

## PART II

# FEASIBILITY AND SITE ANALYSIS

*Dennis Couture, A.S.L.A., R.L.A.*

### INTRODUCTION

One of the most significant changes to the second edition of this book is its format. As stated previously, the first edition grouped the various steps in the land development process by the discipline for which they pertained, whereas this edition presents the material in the chronological order of the land development process. While site design is a continuous process, it can be broken down into at least five distinct steps (or stages), each one producing a specific deliverable documenting the migration through the design process.

**STEP 1:** *Feasibility/programming* provides for the initiation of the process with a general review of program and site considerations and assurance that appropriate base line and base map information is available for design.

**STEP 2:** *Site analysis* results in the identification of both the physical and regulatory site constraints and opportunities and establishes a usable site area in which to fit one or more of the client's/designer's development concepts.

**STEP 3:** *Conceptual design* is based on feasibility and site analysis information and presents the initial organization of the development program.

**STEP 4:** *Schematic design/preliminary site layout* is a refinement of the initial study sketches that adds scale and precise testing of specific uses, including building arrangements and dimensions.

**STEP 5:** *Final design* is the final step in land design. At this stage of the design process when the

preliminary plans have been reviewed and approved by the client and the local governing agencies, engineers finalize their design plans such that their level of detail is sufficient enough to construct all aspects of the project.

When a parcel of land is being developed, feasibility and site analysis seemingly go hand in hand. Typically, the developer will perform both simultaneously or in near concurrence to one another. While the book's format groups these steps together, we will address them separately in the following brief introduction to delineate their differences and indicate how each step serves a distinct purpose in land design.

### STEP 1: FEASIBILITY

The initial step in the development design process requires an understanding of the general development intent and characteristics of the site. During this phase, the design team reviews all available feasibility and impact studies that may have been previously undertaken for a given property. The designer and other members of the design team work with the client to fully explore and describe the proposed development program in conjunction with preliminarily identified opportunities and constraints.

The feasibility effort requires investigation and documentation along three main avenues. These include confirmation of micro and macro site characteristics, program components and development intent, and applicable planning and regulatory controls. Once this information base has been established for a specific site, the formulation of a design response can proceed.



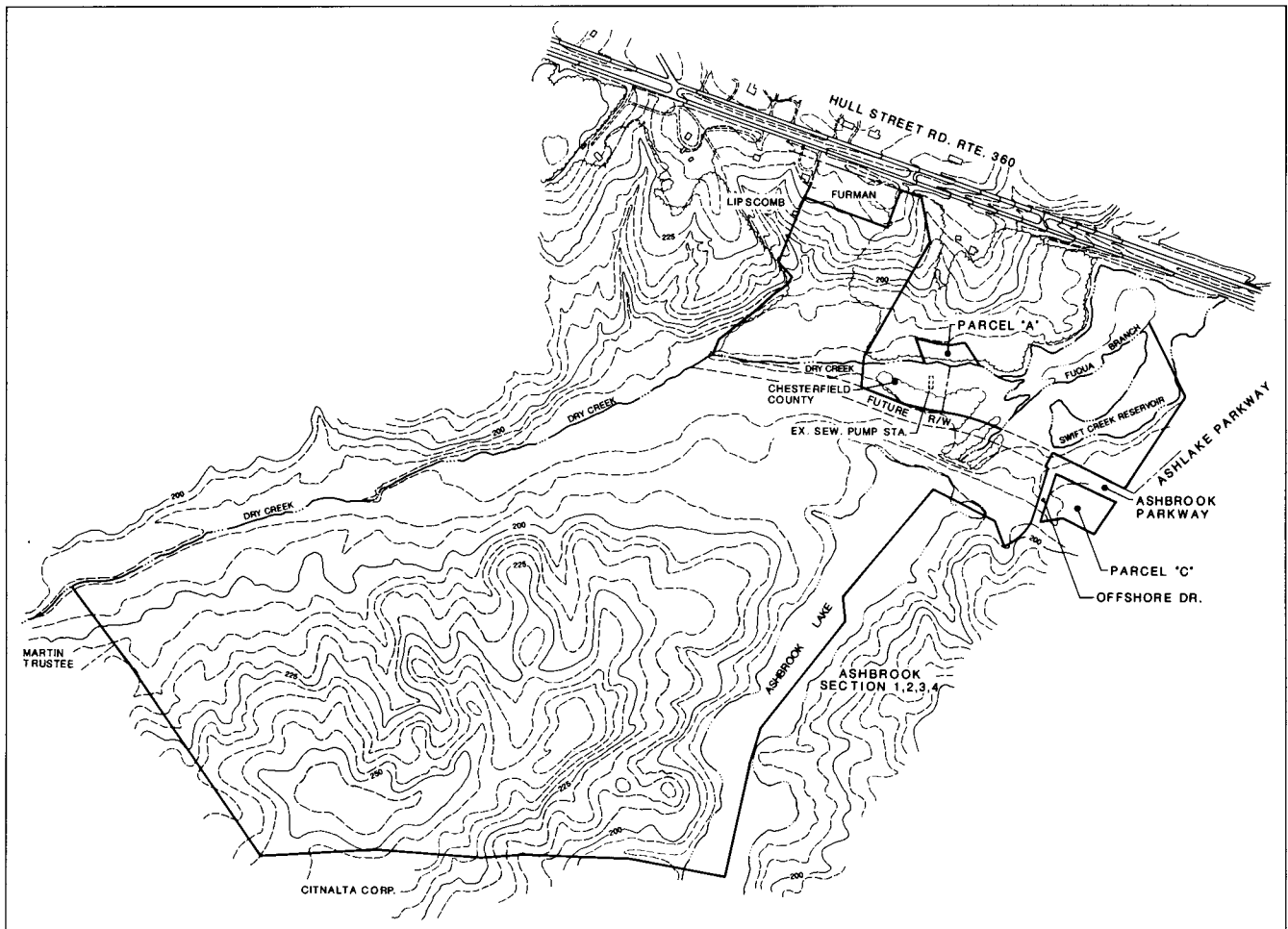
Usually the designer is charged with several important tasks that will assist in initial development discussions. These include:

- Preparation of the base map
- A contextual reference of the site and its surrounding area
- The assembly of secondary source information that will assist in determining site use potential
- The completion of an overall site analysis that focuses on the identification of development opportunities and constraints associated with the subject site

### Base Map

For site design, a base map (see Figure II.1) that details the property configuration and terrain is a fundamental requirement. Ideally, a property survey and topographic map will be available from the client. If not provided by the client, boundary information available through local property tax map sources and topographic mapping, which may have

been compiled by local jurisdictions or the U.S. Geologic Survey, should be obtained. The latter source is more suitable at conceptual design stages for larger land holdings as the scale of the map is 1 in.: 2000 ft with topographic intervals of 10 ft. Municipalities may have more usable base information at 1 in.: 200 ft with contour intervals of 5 ft. The designer should continually be aware of scale and accuracy limitations imposed by secured base information. Despite the variety of technologies available to enlarge base materials, it is preferable to work with information that was originally compiled at a scale close to that which the designer is using in his studies. The scale of a base map may vary depending on both the size of the subject property and recognition of ultimate plan submittal requirements necessary as part of a public review and approval process. While there are many reasons for selecting a particular scale for the base map, one should never lose sight of (1) the need for the base to satisfy public submission requirements; and (2) a scale that is consistent with the designer's need to present a level of accuracy in reviewing the various development considerations.



**FIGURE II.1** Example of a base map.

Property limits, from either a field run property survey, research and plotting of existing deeds and records, or available parcel tax maps, together with topographic information constitute the primary elements of the base map. This information should be augmented with secondary information including such items as road rights of ways/pavement widths, existing structures, vegetation, bodies of water, streams, adjacent property boundaries and surrounding parcels, and utility rights of ways and easements.

In addition to the information provided on the base map, another important aspect of project design is an awareness of existing and future conditions/uses of a given property. Current land use plans, zoning maps, road and utility plans, and public facility plans are important and should be analyzed during the feasibility and programming step. Information regarding adjacent property is also helpful in evaluating the client's proposed development program. A current quality air photograph can be of immeasurable assistance in verifying preliminary information and augmenting site research and field observations. If previously performed preliminary feasibility studies are available, they should be reviewed. If an extended period of time has lapsed since the report was prepared, it is prudent to corroborate the accuracy of the information. Refer to Chapters 2 and 4 for detailed discussion of comprehensive planning and regulatory functions and feasibility studies.

## STEP 2: SITE ANALYSIS

The purpose of the site analysis is to provide the designer with a full understanding of the opportunities and constraints associated with a property. A field visit is the best means of ensuring site familiarity. This should be done with a base map in hand on which appropriate annotations can be made in the field. The visit should ideally be completed after a review of previously available site data to provide the designer with a sense of what to expect on site as well as allow for field verification of previously compiled information. Ideally, the site analysis notes should be on a base map that is at the same scale as the ultimate conceptual design studies.

Typically, a site analysis is conducted following the establishment of the proposed development program and parameters that allow for realistic assessment of the property. While this is not always possible, it is advantageous to conduct more intensive site investigation after the initial range of desired uses for a given site has been established. The checklist of site characteristics that may pose potential consequence to later site design may include the following characteristics and associated considerations.

- Topography, slope, soils:
  - Is the topography fairly uniform throughout the site?
  - Are there areas where the existing topography is not conducive to the proposed program?
- Are there problem soils or soil/slope relationships that reflect potential instability, unusual construction practices or excessive costs, or development restrictions imposed by local government?
- Do soils indicate the potential for wetlands or other unique surface characteristics?
- What implications does the topography have on utility and drainage consideration? Confirm the location of major and minor ridge lines or drainage divides.
- Property configuration:
  - Are there constrictions or dimensions that may inhibit the utility or usability of the site for the intended program?
- Existing vegetation:
  - Confirm the location and character of existing vegetation.
  - Does the vegetation type vary within the site?
  - Does the quality and distribution of existing vegetation afford opportunities to enhance the intended use(s) by incorporating the existing vegetation in the design?
  - Do the prevalent species indicate probability of poor or wet soils?
  - Does the existing vegetation afford natural opportunities for screening or buffering of internal or external views within the site?
- Hydrology, drainage, water, wetland, floodplains:
  - Do the size, location, distribution, or quality of water-related site elements provide opportunities or constraints in regard to the intended use program?
  - Confirm general condition of site runoff. Are there indications of erosion or intermittent ponding?
  - Is stormwater detention/retention required on-site? If so, what type and size facility will be required and will its location serve as a site constraint or opportunity?
- Views, visual characteristics:
  - Document and qualify the existence of internal and external viewsheds.
  - Evaluate the potential for openness or enclosure associated with development opportunities.
  - Will the removal of existing vegetation extend viewsheds or detract from site visual quality?

- Will it be possible to screen undesirable views?
- Climate, site orientation, and exposure:
  - What are the prevailing wind directions/patterns?
  - Are there on-site conditions that prompt microclimate nuances particular to the subject property?
  - Are there on-site opportunities to promote passive solar techniques?
- Adjacent land uses:
  - Are the adjacent land uses compatible with the program considerations for the subject site?
  - Is there a need or opportunity to extend existing or planned community systems into the site, such as parks or pedestrian trails?
  - Are there visual or acoustical conflicts associated with surrounding uses/activities? traffic noise? airport flight patterns? conflicts that will require mitigation strategies focusing on the provision of additional setback and buffers, or unique building design or site layout concessions?
- Access, potential, circulation patterns:
  - Does the property have existing public street frontage?
  - Where are opportunities to access the community vehicular (and pedestrian) circulation systems?
  - Will potential connections meet applicable standards?
  - Does the site terrain suggest or mandate desirable alignments for internal circulation systems?
- Utility locations, existing easements:
  - Are existing utilities available in the vicinity to serve the development program? How and where would these systems access the site? Is the topography conducive to these routings?
  - Would some utilities require individual systems on-site (i.e., well and/or septic)?
  - Are areas of the site more suitable for on-site utilities?
  - Will additional easements be required to route utilities to the site?
  - What are the cost implications in getting the utilities to the site?
- Existing development encumbrances on-site:
  - Are there additional design or development criteria that will govern the site design response, such

as deed restrictions, covenants, and design guidelines?

- Other regulatory requirements:
  - The Fair Housing Accessibility Guidelines and the Americans with Disabilities Act Accessibility Guidelines have significant consequences on layouts. These and similar issues should be reviewed as part of the site analysis step.

Following the assessment of the site characteristics listed above, the results are generally presented in a series of exhibits and/or a single summary site analysis which documents the site's development opportunities and constraints. This information may be summarized in composite maps, diagrams, or other exhibits to depict development opportunities and constraints graphically. Figure II.2 provides examples of the various site analysis maps that could be developed to show various site characteristics. An example of a composite map is provided in Figure II.3.

In conclusion, the site analysis should provide an overview and initial delineation of those portions of the site that are *capable* or *not capable* of supporting various elements of the development program. This assessment should result in an ability to quantify areas of the site deemed "usable" for program development. Often this quantification is referred to as the net buildable area of the site. It represents that portion of the entire gross acreage of the site that the designer and other members of the design team have determined can reasonably be used in the proposed development program.

### Appreciation of Program

As part of a multidisciplinary team, the designer may be an initial participant in orchestrating the research and background information that leads to describing the development program. However, with increasing frequency the program is being spearheaded by the client alone or in concert with his market consultant. The designer is, however, in a unique position to assist the client team in refining the use associated with the development program based both on site characteristics and public planning, land use, and regulatory controls.

In order to ensure a firm understanding of the development program, the designer should elicit from his client as much information as possible at the inception of project design. It is insufficient to begin design activity mindful only of the generic or seat-of-the-pants agenda of land use types desired. Clients generally have strong preconceived ideas regarding the character of the project they wish developed. At a minimum they can relate their expectations relative to existing projects they believe most resemble their current proposal. Even the most unseasoned client has certain base development objectives that have prompted the initiation of a specific development effort. The designer should discuss these expectations with the client, including

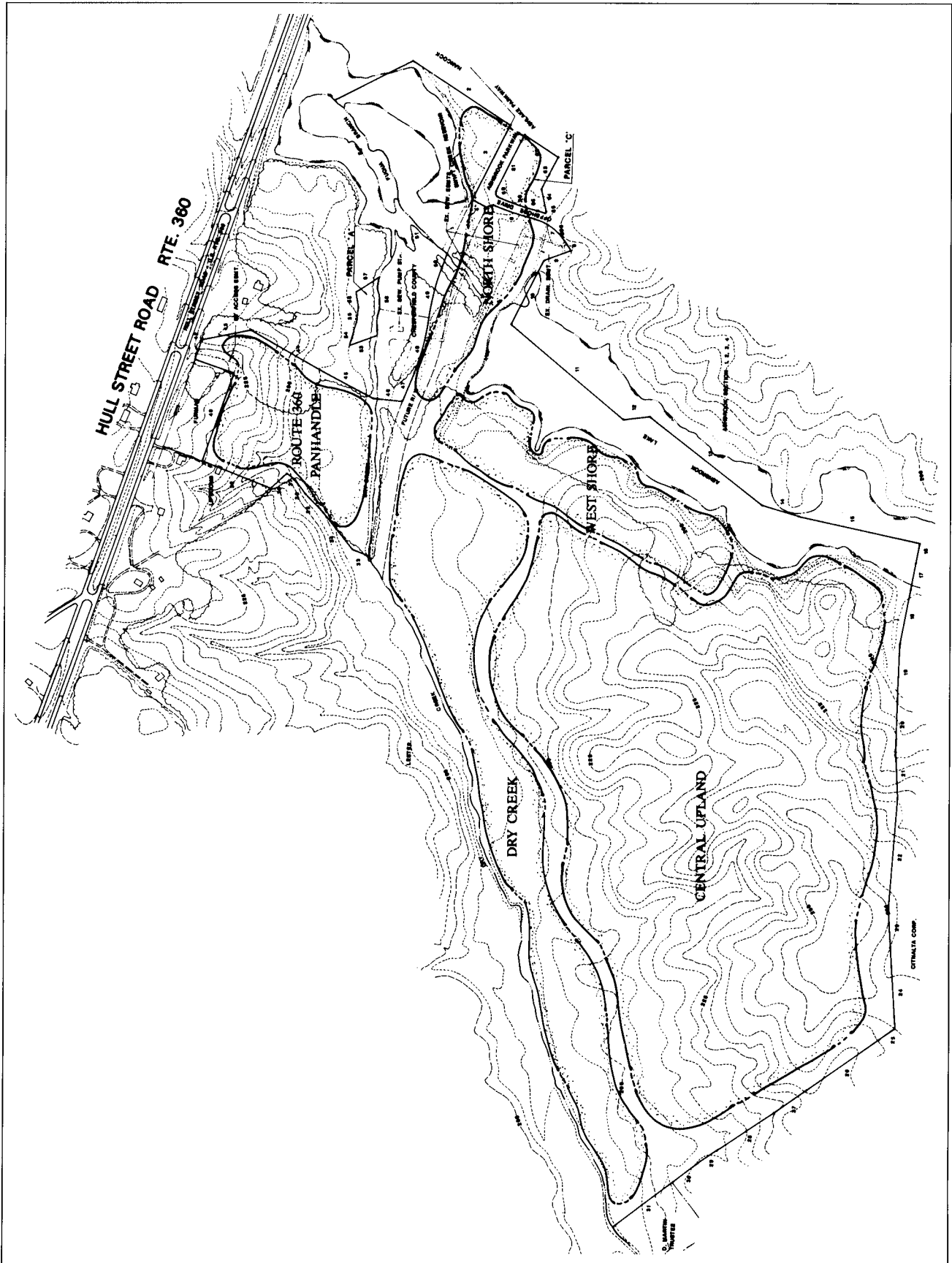


FIGURE 11.2 Example of series of site analysis (seven maps).

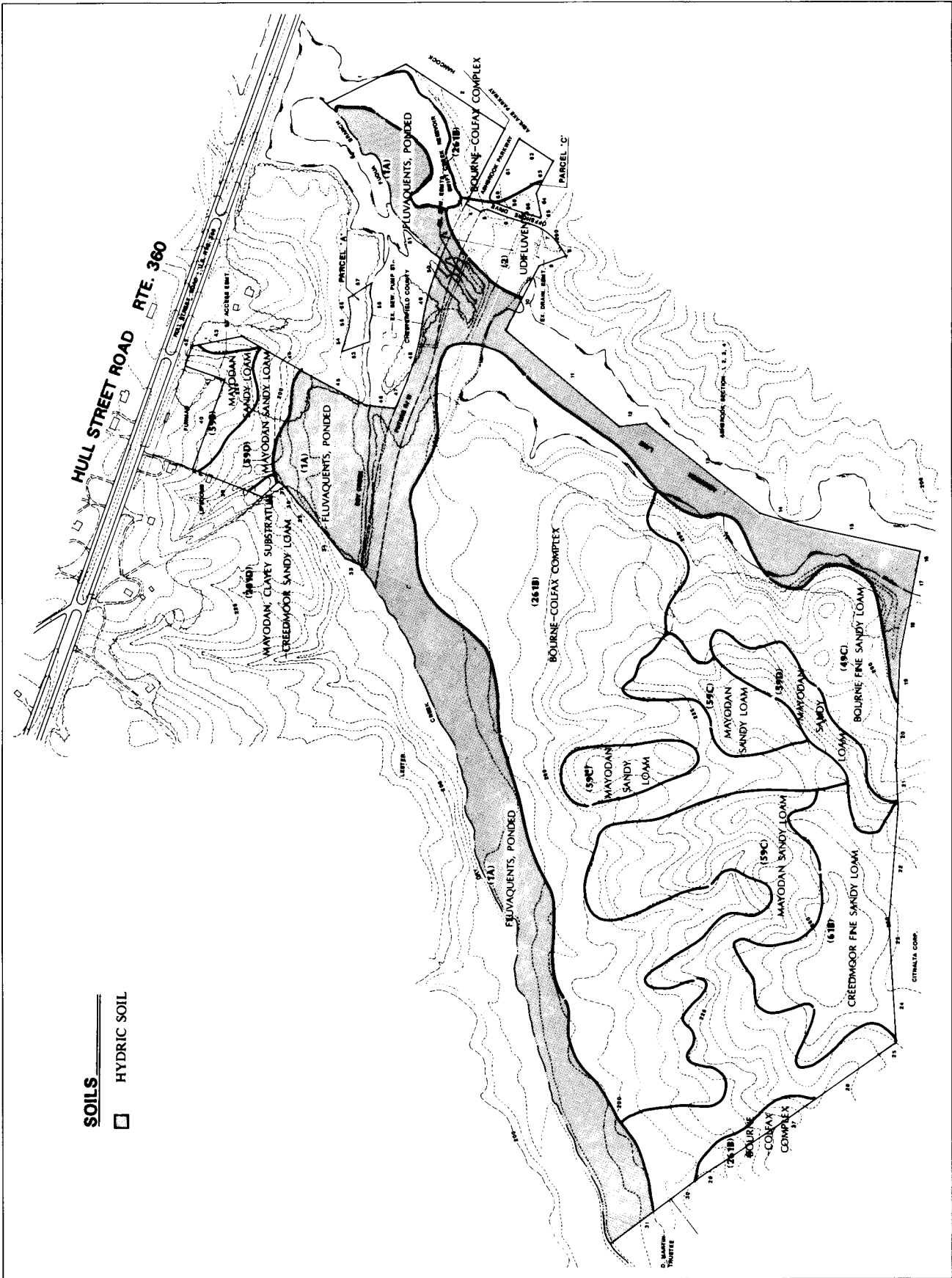


FIGURE II.2 (Continued) Example of series of site analysis (seven maps).

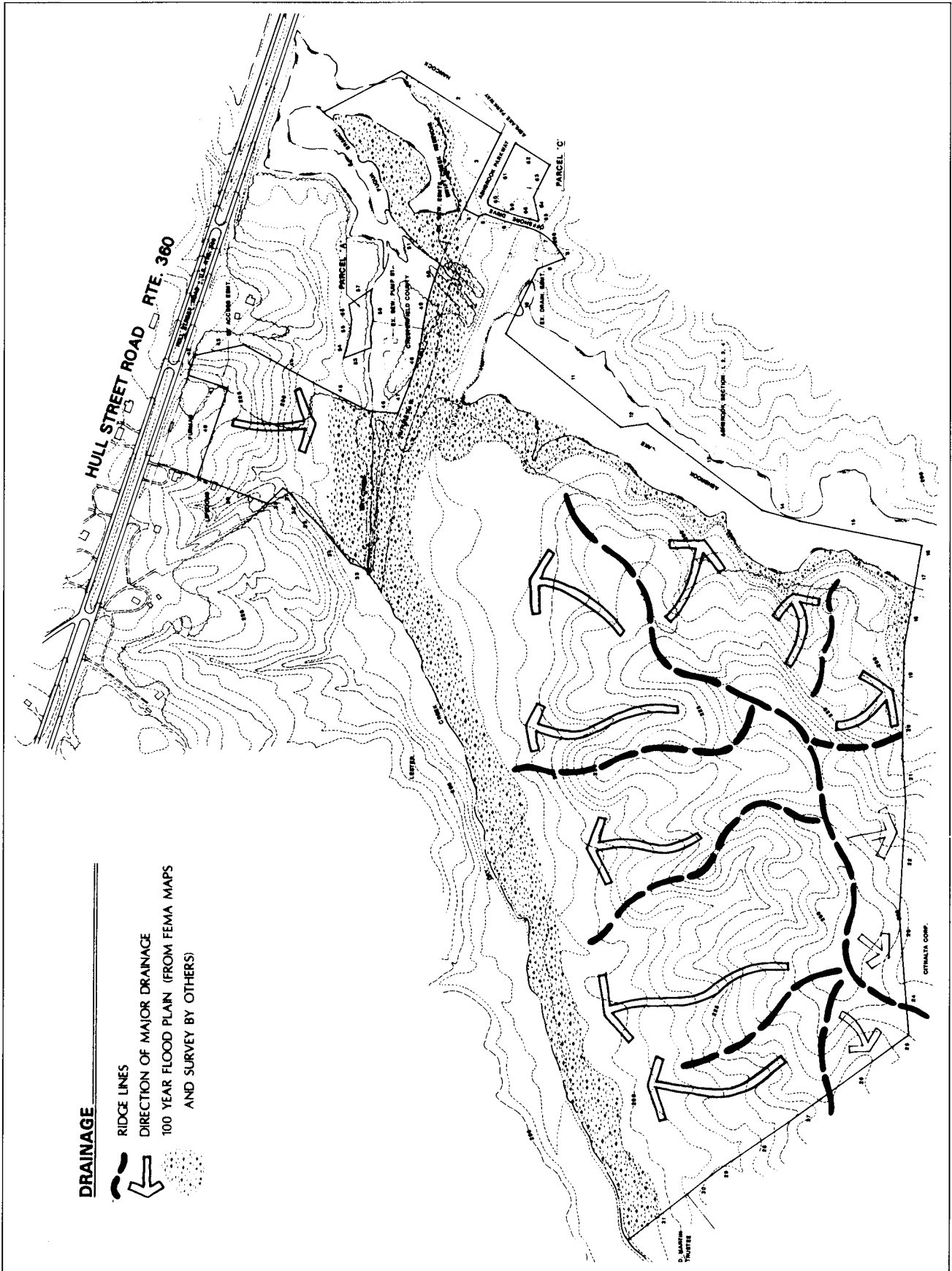


FIGURE II.2 (Continued) Example of series of site analysis (seven maps).



FIGURE 11.2 (Continued) Example of series of site analysis (seven maps).

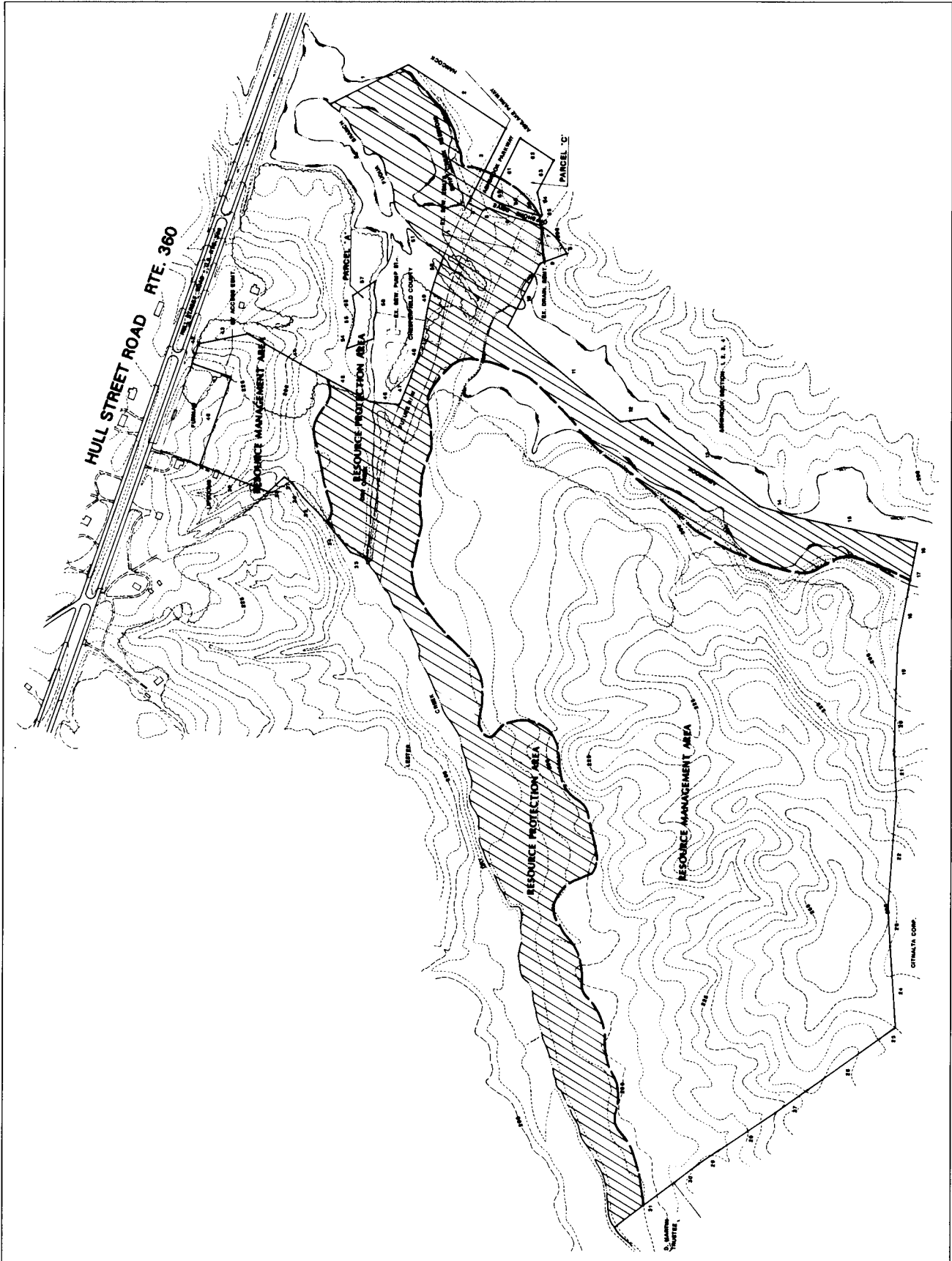
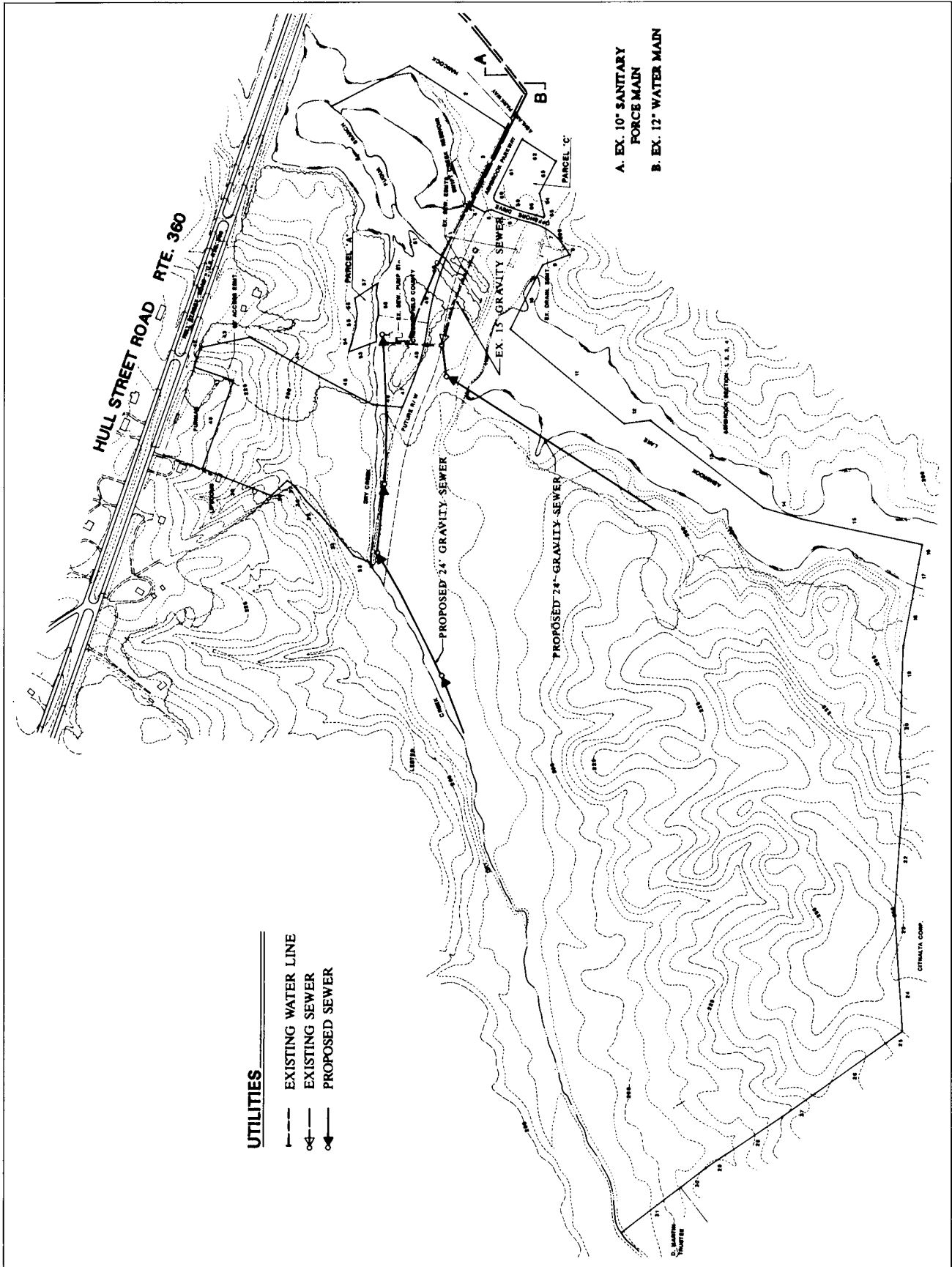


FIGURE II.2 (Continued) Example of series of site analysis (seven maps).





**FIGURE 11.2** (Continued) Example of series of site analysis (seven maps).

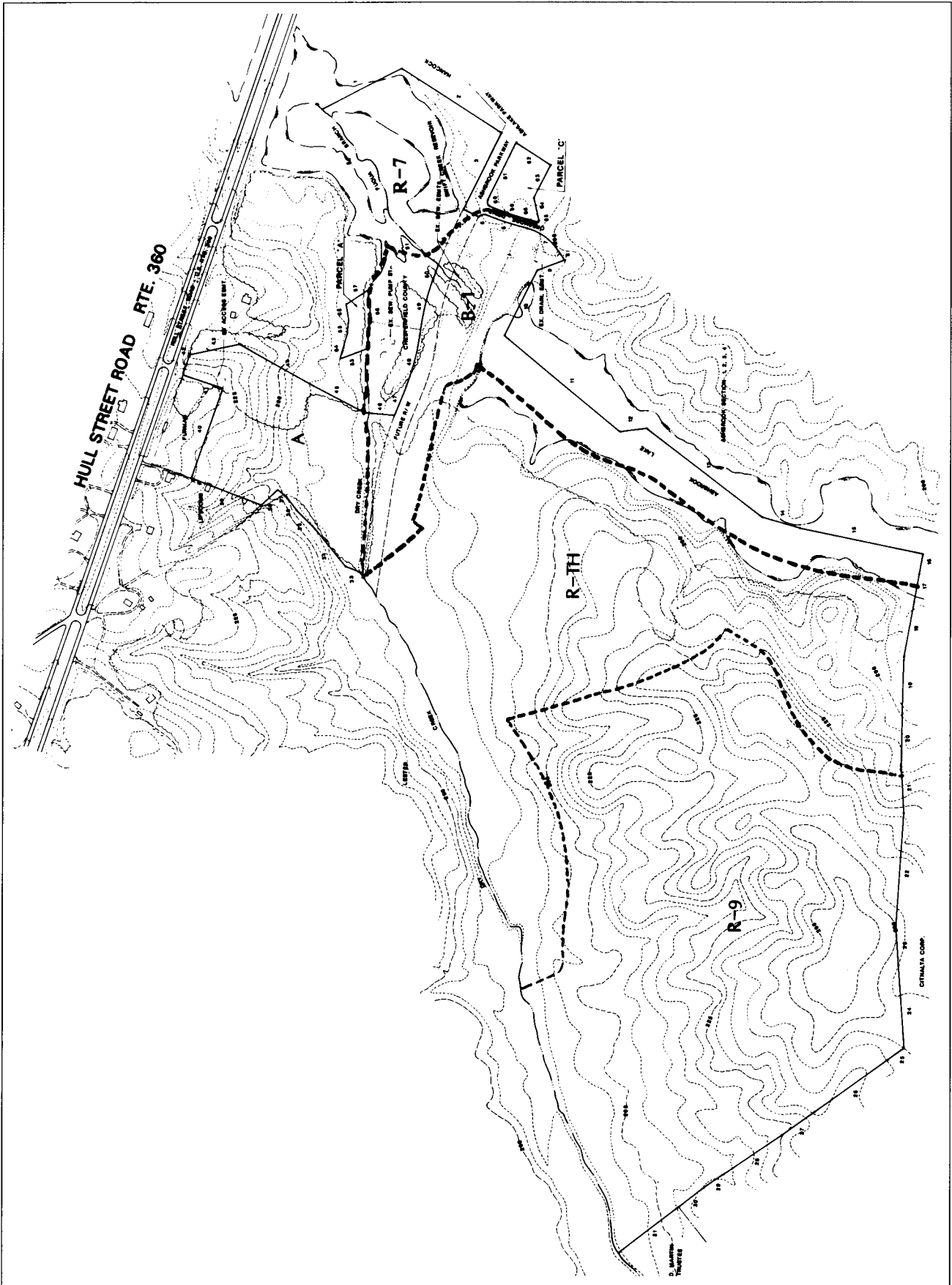


FIGURE 11.2 (Continued) Example of series of site analysis (seven maps).

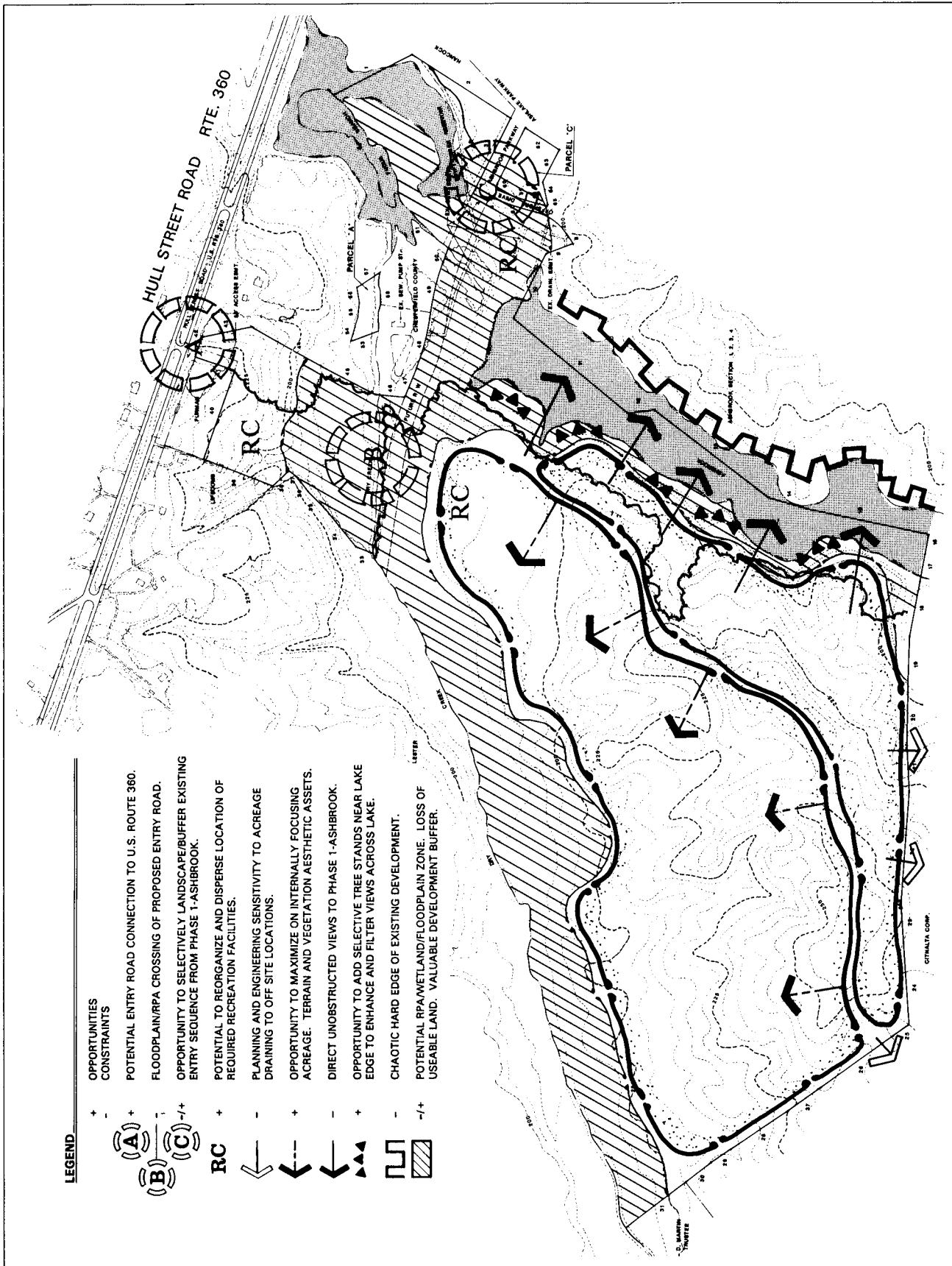


FIGURE I.3 Example of composite map showing opportunities and constraints analysis.

an inventory of objective criteria such as unit or building type, building dimensions, architectural finish, parking ratios, and amenity elements, and more subjective statements regarding the desired character or ambiance of the finished product. Although clients will often indicate the density or intensity of the development they desire, more often than not it is insufficient to initiate a site design. Most clients have completed at least a rudimentary assessment of the possible return on their investment for a project prior to the initiation of discussion with a design team. The design team should attempt to verify the anticipated yield, as well as preliminary budgetary allowances for such items as utility construction, amenity development, and landscape, to understand better the anticipated character and quality of development.

It is similarly important that program objectives be discussed in the context of existing planning and regulatory controls. Land use type and use intensity should be reviewed with a clear understanding of existing jurisdictional comprehensive plans and zoning. Based on this discussion, the development program or alternative programs should reflect a realistic proposed land use. Ultimately, the success

of a site design will be measured to the extent it optimizes client objectives against those of public sector expectations and standards. It is a benefit to no one to foster questionable expectations associated with unreasonable program formulation.

This step is a precursor to design. It is, in reality, an information-gathering stage, but information gathering with specific purpose and direction. Information is expensive to compile and time-consuming in its review. Therefore, it is important to undertake the site analysis effort with a clear focus on the judicious expenditure of time and resources. At this juncture in the design process, all available existing information, including base maps, aerial photographs, engineering information and controls, and planning reports, should be assembled.

The end product of this phase of project activity includes a clear understanding of the proposed development program and options, the identification of site constraints and opportunities, assurance that all regulatory requirements can be satisfied in conjunction with refinements to the proposed development program, and a positive finding that the physical and functional characteristics envisioned by the client can be realized.



# PART II.A ALLOWABLE USE OF SITE

## CHAPTER 2

# COMPREHENSIVE PLANNING AND ZONING

*Lawrence A. McDermott*

### INTRODUCTION

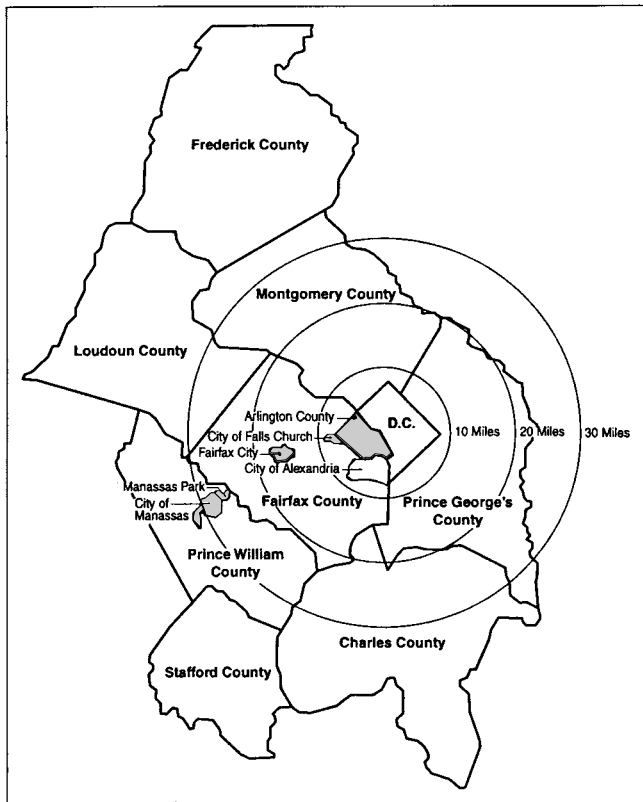
Prior to the land development consultant putting pencil to tracing paper, and hopefully before the land developer's signature appears at the bottom of a land purchase contract, both parties must understand the effect of local planning, zoning, and other regulations. Planning and zoning standards and controls adopted by local government affect everything from the uses allowed on a site to the drainage of stormwater. In some jurisdictions, even the amount of traffic generated by the development is subject to detailed review regulation.

Under our federal system of governance, where states control all powers not granted to the U.S. government under the Constitution, local land use regulations are the product of what is permitted by state planning enabling legislation. This legislation pertains to what kind of land use regulatory powers local governments can exercise. Given the differing state authorizing legislation, local governments operate in different ways. Much of this diversity results from the size of the jurisdiction, the structure of its government, and the legislative authority granted by the state. In addition, each jurisdiction has unique problems and priorities. Residents have varying levels of affluence and sophistication and different attitudes toward their community and government. These all have an impact on the way local governments regulate land development.

Even within a single region, the land developer and land development consultant may operate in several jurisdictions. Consequently, they will encounter a broad variety of plans, ordinances, regulations, policies, and procedures that can all affect project design, economics, and plan approval. Figure 2.1 depicts the Washington, D.C., metropolitan area.

Note that two states, Maryland and Virginia, surround the District of Columbia. Each state operates under a different legislative process and constitutional mandate. Within those two states, there are 13 adjacent or nearby counties and 4 major cities and a significant number of independent towns and villages of varying sizes. Due to the post-World War II growth in the region, several once-rural counties have become integral parts of the region. The merging of the Washington and Baltimore economic regions and the advent of commuter rail have dramatically expanded the market boundaries within which a Washington developer can reasonably operate. Metropolitan-based land development and building companies now reach as far west as the eastern panhandle of West Virginia, south to Richmond, north to the Pennsylvania border, and across the Chesapeake Bay Bridge to undertake new projects. Each of the jurisdictions of the region is unique in its economic base, its citizens' attitudes, and its government's priorities. Perhaps more to the point, each of these jurisdictions within the metropolitan area has its own unique set of planning and zoning regulations. The breadth and expansion of the Washington market is not unique; similar situations exist in most metropolitan areas.

Differences in government budgets and the size and sophistication of review staff and commissions also have broad implications for the development team. These differences affect not only the number of public officials and agencies the development team must come in contact with, but also the level of scrutiny given to the proposed rezoning documents or development plans. As the government's size and structure grows, so does the potential for conflict between its administrative and elected officials. Each official or department manager reviews rezoning and development



**FIGURE 2.1** Jurisdictions in the Washington, D.C., metropolitan area.

proposals with a different constituency or agenda in mind. Each has competing needs and generates competing conclusions. This bears directly on the time it takes to secure zoning approval.

As a jurisdiction matures, political or civic leaders often demand more exacting standards for development and construction. The government responds by tailoring zoning regulations and procedures to address new priorities. Sometimes these new rules and standards take effect even while a proposed zoning project is well underway. These new standards and/or regulations, coupled with increased review time and sometimes circuitous approval procedures, can increase the future land development costs dramatically. The land development team must frequently adjust design and construction budgets during the rezoning process as well as the life of the project.

In an attempt to minimize risk, it is not uncommon for land developers to specialize in several locations within a region, while avoiding projects in other jurisdictions. Decisions to avoid certain jurisdictions may be made because of market characteristics, unfavorable tax rates, or restrictive development regulations. However, it is increasingly rare for all but the smallest developer to operate within a single jurisdiction.

Given these differences, the land development consultant must become familiar with the land use planning and zoning documents and regulations in effect in each jurisdiction

within which the company operates. Absent this expertise, the consultant cannot offer the developer sound advice or produce plans or proposals that can win government approval. It is the consultant and the land use attorney to whom the developer looks to be an expert on local policies and the controls applicable to each new project. Even consultants with prior local government work experience must keep track of newly evolving priorities, policies, regulations, and standards. Furthermore, as government workload increases and operating funds decrease, development review staffs are forced to limit their efforts to technical review. They are unable to devote considerable time or energy to shepherd applicants through the jurisdiction's regulations or review procedures. This places a heavier burden on the land development consultant to develop independence and expertise in the local land development review and approval process.

Developing a clear understanding with local plans, policies, and requirements may seem a monumental task. It is impractical for one person to be sufficiently familiar with all local regulatory programs in the region. This would require extensive research into past practices and countless hours in government boardrooms. Acquiring an overall familiarity with the basic structure of government function and regulations makes it easier to assimilate this information. However, effective land development consulting firms designate team leaders with extensive local experience. Each of these people becomes responsible for knowing and monitoring development and regulatory activity within the jurisdiction where the firm maintains a presence. Absent a clear understanding of a community and its attitudes and politics, the land development process can become a frustrating exercise, with false starts and blind alleys that can prove costly to both consultant and developer.

This chapter addresses only the most typical planning and zoning regulations, ordinances, and procedures. As mentioned above, it is neither practical nor possible to address all variations that exist throughout the nation. However, there are basic similarities in the structure of each type of regulation. This text focuses on their differing impacts on land development and the manner in which regulations or procedures are implemented.

**COMPREHENSIVE PLANNING**

The comprehensive plan, sometimes called the master plan or general plan, serves as a formal statement of the community's goals and objectives. The plan establishes policies and procedures relating to the community's future growth, including new development of land and maturation of existing neighborhoods. The plan represents the collective input of public and private sector attitudes, needs, and desires. Its recommendations are based on extensive analysis of economic, social, demographic, and other forces evident in the community. The plan attempts to provide an adopted vision of the community for some distant point in time, perhaps 10 or 20 years. The comprehensive plan pro-

vides valuable guidance for those making important economic decisions, including local officials, land developers, existing and prospective residents, employees, and business operators. In most states, the comprehensive plan is a guide that is advisory in nature and not a legally binding regulation, such as a zoning ordinance, which indeed is law. However, in a minority of states, the comprehensive plan is directive in nature, requiring conformance, and rises to the level of law.

To government officials, the comprehensive plan serves several purposes. It defines a general pattern of projected land use for the community. It recommends policies that alternatively encourage desired development or discourages inappropriate use or intensity of development. It establishes and reinforces community standards for appearance, design, delivery of public services, and protection of the environment, and it serves as an important guide for allocating resources used for the provision and distribution of public facilities and services. The overall objective of the comprehensive plan is to establish and achieve goals that result in a high quality of life for both the residential and business communities.

Land development, infrastructure management, and the operation of community services involve extreme public, private, and personal investment. As a result of this investment, the court systems have placed increased importance on the comprehensive plan as either a guide for development or a binding source of legally mandated land use regulation. Therefore, most local officials rely heavily on the plan. Its recommendations form the basis for making reasoned and predictable decisions about land use and budgets, rather than those that may be challenged in the courts as arbitrary and capricious.

To the community's residents, the comprehensive plan represents a blueprint for the quality of life they expect to enjoy. It creates identity for the neighborhoods in which they live and defines the services they expect to receive. The decision to move to a community and purchase a home is perhaps the most important of personal investments made, and the comprehensive plan is viewed as a way to predict the soundness of that investment. It identifies the location of new neighborhoods, office and retail centers, new roads, and schools that may affect that investment. The plan offers more certainty and provides a way of anticipating intrusion or impacts that could reduce property value.

For the business sector, the comprehensive plan is an essential source of information on potential new markets. The future locations of new centers of employment or residential communities are particularly important to businesses operating support, supply, and service establishments. The plan provides information that can be used to determine the potential customer base available to the business community. Large employers in need of properties with room for expansion also rely on the comprehensive plan for guidance. Existing and future labor force and sales markets can be determined from the plan. At the same time,

businesses and employers look to the plan for an expression of long-term commitment to business. Like the residents, the land developer also views the comprehensive plan as a protection of property value. It suggests what decisions will be made that are beyond his or her control. More importantly, however, the developer uses the plan to identify new opportunities. The developer is in the business of converting raw or underutilized land to new uses. The land development consultant and the developer use the comprehensive plan to determine the suitability of purchasing specific land for new projects based on the planned land use and their particular development program.

Many land developers specialize in a specific land use and product; therefore, the plan is an important tool in identifying the area, and sometimes the specific parcels, that are best suited for the particular development program being pursued. The availability of public facilities and services is an important part of identifying the potential of a piece of property. Similarly, the compatibility and impact of adjacent use is an important factor in selecting property for development. Just as a homeowner is concerned with the character of the community, so too is the developer concerned about the character of the community. For example, a luxury office developer may not believe that an adjoining industrial or warehouse facility is a suitable neighbor. The plan helps define the market area of a site. The developer uses this information to determine whether there are sufficient employees, residences, or customers to make the project a successful venture. Later chapters discuss market analysis and project feasibility in detail.

As the developer's agent, the land development consultant is responsible for understanding the comprehensive plan and its various components. The consultant must also be familiar with the planning process itself. The information used in planning and the events that lead to the plan's adoption form an important foundation for project design. The consultant must understand not only the impact of the plan's recommendations to the land use, but how the jurisdiction will enforce its recommendations.

The plan provides other critical information without which the application of other regulatory tools could be meaningless. This information relates to general design requirements imposed on a project beyond simple use and scale. These include conditions for reducing environmental and service impacts of the development. The plan also aids the developer by projecting when public and private infrastructure and services will be available to the site.

### **Community Planning: A Participatory Process**

Since the plan is intended as a reflection of community attitudes and desires, the process usually involves several opportunities for citizen participation and input. The process that some municipalities follow to adopt or revise the comprehensive plan is often lengthy and controversial. Even when this is not the case, it is important for the land development team to become involved in that process.



Whether or not projects are underway or envisioned, it is important to participate in the process because the results will shape or constrain development opportunities for years to come. The information gathered and attitudes voiced are an invaluable resource for preparing future proposals.

Although in some cases a close monitoring of the planning process is sufficient, full participation in the process may be preferable. Coming from residential citizens of the municipality and members of its business community, the team's input is a critical and rightful component of community thinking. The policies and subsequent regulations and procedures that the plan will recommend must reasonably coincide with development practice and not serve as a hindrance to future success. Where communities are encouraged to accept the practical expertise offered by development professionals, the plan and its policies are more effective in achieving the community's goals.

Most medium- and large-sized municipalities employ a professional planning staff that coordinates the overall planning effort, while smaller communities often retain outside consultants for this purpose. The governing body relies heavily on the recommendations of its planning staff, although decisions about the plan and policies will ultimately be its own. A timetable for the plan and framework for the analysis will be prepared. Sometimes working with an ad hoc task force composed of the community's civic, business, and political leaders, the staff undertakes extensive data collection. The staff will compile the demographic and economic inventory statistics needed for the analysis, as well as review existing development patterns and activity.

The comprehensive planning process often provides for a period of open nominations, during which citizens can suggest new or revised development policies and specific parcel recommendations for inclusion in the plan. For the land development design team, this is perhaps the most important part of the planning effort. During this period, it may be possible for citizens to recommend, and the governing body to consider, new or revised land use designations for properties under the control of the developer. This could result in a downplanning, or reduction in the land use or development intensity recommended in the comprehensive plan. Conversely, this forum provides the land development design team with the opportunity to suggest a different and perhaps more intensive use of the property and strive to have the comprehensive plan recommendation revised accordingly. Depending on existing parcel zoning, future projects could be drastically altered if development intentions are not confirmed or defended by the land development design team during this nomination period.

As stated above, this nomination period also provides an opportunity for citizens as well as landowners to introduce new proposals for future development of the holdings of the developer. Circumstances unforeseen in prior plans, such as expansion of an airport, revitalization of a business center, decline of a neighborhood, or influx of new industry, may justify a change. Similarly, assembly of smaller parcels into a large development tract may also suggest an oppor-

tunity for a project of grander scale than anticipated in the plan. The planning staff and other involved groups consider these proposals in preparing the plan. The planning staff and other involved groups consider these proposals in preparing the plan. Supporting documentation and testimony presented by the development team form an important component of the analysis.

As the planning staff prepares preliminary drafts of the comprehensive plan, the governing body often schedules town meetings, informal public hearings, or similar forums to allow public input and comments. The purpose of these meetings is to gain a sense of the community's reaction to the plan. Frequently, the planning staff compiles testimony and comments presented at these meetings and responds with an analysis of issues and conflicts. As the process draws to a conclusion, formal, statutorily mandated public hearings will be held by either or both an appointed planning commission or similar board and the governing body of the municipality. Throughout these hearings, the information presented by all segments of the community provides an important record upon which to base subsequent land development decisions.

Depending on the size of the community, the entire process can take several months to several years. To spread the demanding workload, many municipalities update their comprehensive plans, or portions of it, on a cyclical timetable. Some state legislation mandate the time within which the jurisdiction must review and update the comprehensive plan, such as every five years. Opportunities for property owners to seek out-of-turn or off-schedule plan amendments may be limited. The land development consultant must be familiar with local practice to advise development clients properly.

### **Composition and Characteristics of the Comprehensive Plan**

Most comprehensive plans consist of six major parts:

1. Statements of the community's goals and objectives
2. Inventories of its existing characteristics, features, land uses, and facilities
3. Projections of trends expected within the life of the plan
4. Text describing policies to be applied in order to achieve the plan's goal
5. Maps and text depicting and discussing the community, showing current and future land use, the location of future public facilities, environmental resources, and other features
6. Text describing how the community intends to carry out the goals of the plan

The statement of goals and objectives is a reflection of attitudes towards the elements that interact to form the built environment. Typically, these elements include:

- Population
- Housing
- Employment
- Retail activity
- Industrial output
- Transportation
- Recreation and open space
- The environment and natural resources
- Public utilities, services, and facilities

Many communities add goals addressing:

- Economic development
- Social or human services
- Historic preservation
- Major institutional uses, such as airports, colleges, or government installations
- Other important indigenous issues or land uses, such as agriculture, mineral production, or maritime activities
- Intra-jurisdictional and intraregional relationships and responsibilities

### Establishment of Goals and Objectives

The goals statements themselves are usually broad. They are often criticized as being innocuous “motherhood and apple pie”-type assertions. For example, in 1978, Anne Arundel County, Maryland, adopted a new comprehensive plan with three goals described as “the foundation for this Plan.” One of the goals, the overall social goal, was to “provide the best possible opportunities for all of the people to earn a living, to have a comfortable home, and to enjoy physical and emotional health, personal security and safety, access to new skills and knowledge, and control over their own lives” (Anne Arundel County, 1978). Certainly the goal is important. However, it offers little in the way of advice for those making development decisions, either in the public or private sector.

In some jurisdictions, however, the goals are very focused. They reflect strong attitudes about shortcomings or conflicts in prior comprehensive planning efforts or the rate of growth being experienced in the community. The Fairfax County, Virginia, Board of Supervisors in 1987 appointed 20 civic and business leaders to a countywide Goals Advisory Commission. The action was a response to a divisive conflict over the county’s explosive growth and government efforts to control it. The panel spent 11 months in intense analysis and dialogue about county and industry past practices. In addition, 11 months of controversial public hearings were held by the commission, the county’s Planning Commission, and the Board of Supervisors. The resulting

18 goals formed the basis for a 3-year, multiphased review of the county’s comprehensive plan. In contrast to the Anne Arundel goal, a typical goal in the adopted Fairfax plan follows:

Transportation—Land use must be balanced with the supporting transportation infrastructure, including the regional network, and credibility must be established with the public and private sectors that the transportation program will be implemented. Fairfax County will encourage the development of accessible transportation systems designed, through advanced planning and technology, to move people and goods efficiently while minimizing environmental impact and community disruption. Regional and local efforts to achieve a balanced transportation system through the development of rapid rail, commuter rail, expanded bus service and the reduction of excessive reliance upon the automobile should be the keystone policy for future planning and facilities. Sidewalks and trails should be developed as alternative transportation facilities leading to mass transit, high density areas, public facilities, and employment areas. (Fairfax County, 1990)

Whatever the form of the goal statements, they set a standard against which the actions of both the land developer and local governing body are judged. In preparing the client’s plans, the land development consultant must consider how those plans address these overriding goals. The team representative will likely be called upon to justify the proposal and must be sure the plan is characterized as a solution rather than an exacerbation of a problem identified in the plan.

### Inventory and Trends

Inventory and trends analysis provides a starting point or snapshot of the community and its citizens’ mindset. The inventory is a valuable resource to the development team because it includes a demographic analysis of the population of the community. It shows statistics such as age distribution, household formation, income, labor force, and home and automobile ownership. Along with a large-scale map showing existing land use, the plan describes land use characteristics, including a breakdown of number and type of housing units, structural condition, and land and building area of nonresidential uses. The plan also provides economic data, such as retail sales and manufacturing output. Many plans contain information on real estate absorption and conversion rates, property sales, and leasing activity, while others also offer statistics on commuting patterns and transportation use. With the help of the land development consultant, the developer uses the inventory to determine the needs of residents and business, and their financial capacities and limitations.

The premise of comprehensive planning is that, by identifying growth patterns and adapting land use and economic controls, it is possible, if determined good public policy, to alter the patterns. The application of these controls may involve redistribution of public resources in accordance with reordered priorities. Strict controls and disincentives

may be considered to prevent or deter specified development actions. Conversely, policies may be adopted that provide incentives to land developers and users that succeed in altering their actions to achieve the goals of the plan.

Incentives may simply take the form of public support for a desired change in zoning. Policies contained in the plan may suggest density or other bonuses that provide financial incentive to the land developer in exchange for public benefit. The plan may suggest that public funds be applied to critically needed public works improvement projects in areas where growth is desired. It may recommend the timing of development to keep pace with those improvements.

### Land Use Maps

Accompanying the comprehensive plan text is the proposed land use map. Often color-coded or shaded, the generalized land use maps of the community represent a graphic depiction of the relationship of existing and future land uses and facilities. The map divides geographic areas into desired and projected uses and intensity. These areas usually represent the broad categories of land use, such as residential, commercial, industrial, and other employment centers. Subcategories of development intensity show the gradation of land use patterns. For instance, the map may show that an area of high-density residential is a desirable future use adjacent to an existing commercial center. The map defines the boundaries of the area and provides a range of relative densities. The map frequently shows the proposed location of significant facilities, such as transit centers, proposed freeway interchanges, regional shopping malls, and schools. It offers a snapshot of what the community would like to be at a distant but determined point in time. Figure 2.1 provides an example of a section of a comprehensive plan map.

The mapping of land use at large scale often generates considerable controversy. By its nature, a map provides geographic references and landmarks to which people can relate—the corner down the street, the area across from the shopping center. The generalized land use map, therefore, appears to fix geographic location for the information it presents. Consequently, many community residents look to the map as a tool to control land use and as a source of absolute information.

Often, however, land development does not conform to the finite categories shown on the map. The plan map may propose that an existing residential neighborhood is redeveloped to a higher density or a major facility is shown in a location that concerns some in the community. Citizens often lose sight of the fact that the plan serves as only a guide. So, too, the fact that the plan spans a 5- to 20-year period is often forgotten. Some properties may undergo several use changes awaiting conditions favorable to the ultimate land use. Reference to the plan's designations for broad sections of the community ignores the fact that the plan, in some instances, intends for individual parcels to develop differently.

To eliminate some of the ambiguity created by scale and time, many communities undertake a multitiered and multiphased planning effort. This effort narrows its vision, often at several intermediate steps, in order to examine smaller subsections of the community. At its extreme, the plan makes parcel by parcel recommendations on land use, density, compatibility, and the facilities that serve them. When this happens, the comprehensive plan begins to resemble the zoning ordinance more than a plan to guide future development. However, even at this scale, the plan still serves only as a recommendation for future development, albeit a strong one.

### Plan Implementation

Although the goals statements in the plan represent the community's vision of quality of life and future development, the plans for implementation are the key to that vision's realization. Despite its detail and breadth of coverage, the comprehensive plan is, in fact, only a guide in most jurisdictions. The governing body should strongly consider the plan's recommendations when making land use or fiscal decisions. However, it does not, in most jurisdictions, carry the force of legislative acts adopted by the jurisdiction, such as the zoning or subdivision ordinance. Despite this, the link between those ordinances and the plan is important, as most communities rely on the findings in the comprehensive plan when considering zoning and development applications. The zoning ordinance may mandate that the land developer consider the detailed provisions of the plan and address those issues in the development proposal. In a minority of states, sometimes referred to as plan conformance jurisdictions, the plan is a mandatory directive for land use regulations and, by law, zoning decisions must conform to the plan's mandates. For purposes of the following discussion, the majority rule that the plan is a guide will be studied.

The existing zoning of a parcel takes precedence over the recommendations of the comprehensive plan in almost all cases. This is true even where the use and intensity provisions of the plan are more restrictive than those in the zoning ordinance. For this reason, an important role of the comprehensive plan is to recommend the adoption of changes to ordinances that regulate land use and development. The governing body of a jurisdiction may amend zoning and subdivision ordinances without specific direction from the comprehensive plan. However, the courts often find that such actions are an abuse of authority when they are accomplished without the rational basis offered by comprehensive planning analysis.

To the land development consultant, the comprehensive plan is more than just a guide to a community's future physical growth. Rather, the plan represents a historical depiction of the community and a retelling of the planning process that the community undertook to develop its vision. It represents an invaluable resource in the data and direction it offers, yet the language in the text, whether it relates to an overall policy or a recommendation for the

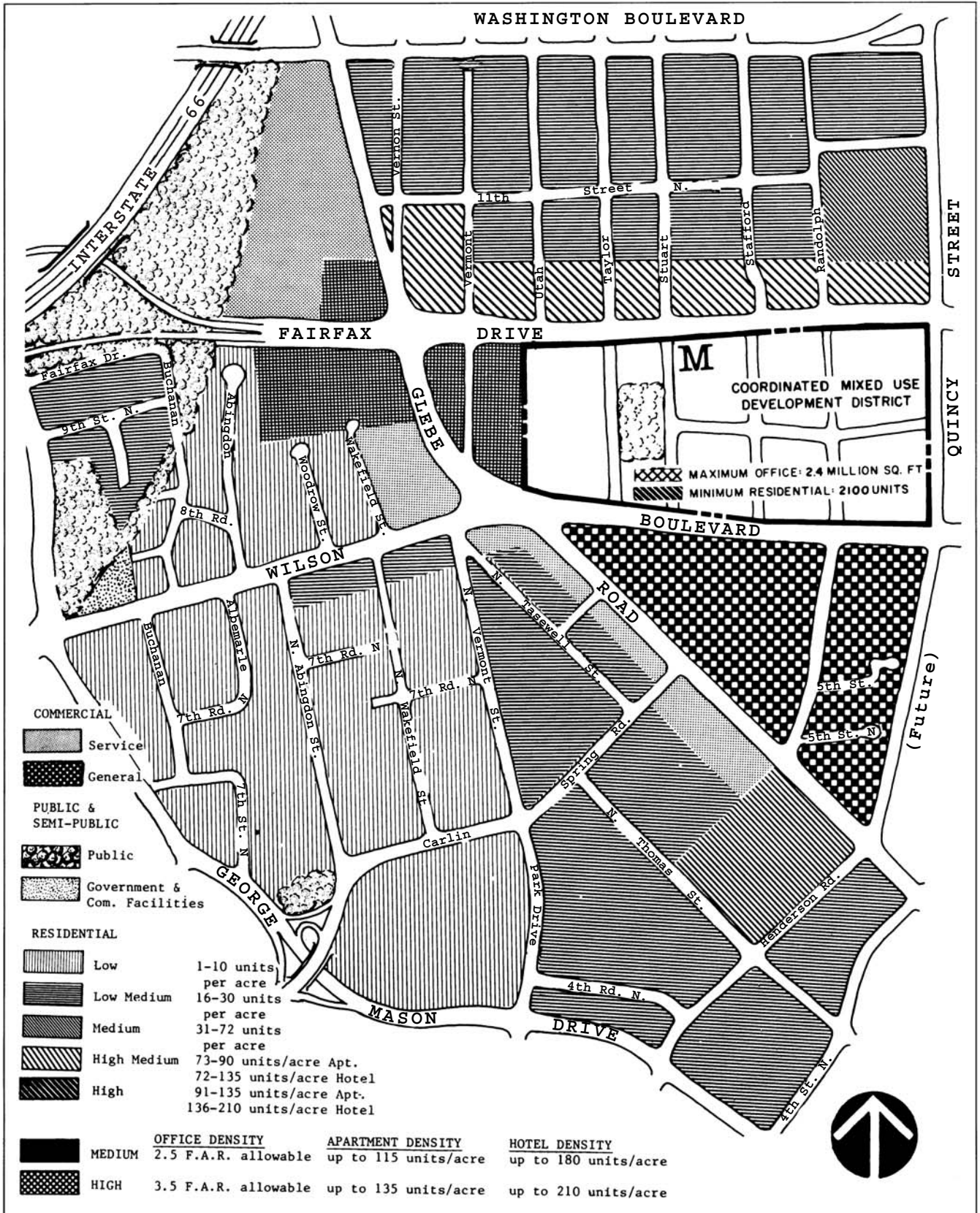


FIGURE 2.2 Example of a comprehensive plan map.

developer's parcel, is often vague. Frequently, the plan leads to varying interpretation by regulatory officials, citizens, or the developer. This adds to the considerable challenge facing the land development consultant and the developer in preparing the proposed development proposal.

**What to Look for and How to Use the Comprehensive Plan**

Depending on the size of the community and its particular planning and zoning authority, the comprehensive plan may consist of one or several documents and maps. Many large localities will segment the land area into several planning areas or sectors. Where this occurs, the text provides extensive discussion of both jurisdiction- and sector-wide policies, issues, and opportunities.

Some communities identify smaller areas that warrant special study, such as business districts, industrial areas, transit centers, areas of important institutional use, or other significant concentrations of population and employment. Such areas often have unique circumstances deserving closer examination due to their overall importance to the community. Finally, there may be distinct functional plans that address partial elements of the plan that are of special importance to the community, including transportation, sewer and water, the environment, urban design, and even trail systems. As is the case with the area plans, these also examine issues and opportunities and provide a set of policies considered in development review and budgeting. The plan specifies how the community intends to achieve its specific goals. These functional plan recommendations

overlay and work in conjunction with the underlying plan and parcel-specific findings.

All documents and maps that relate to jurisdiction-wide issues must be assembled by the land development team and analyzed for possible impact on a specific parcel planned for development. Table 2.1 demonstrates the variety of plans produced in a given municipality.

The consultant must obtain the most recent version of the plan and keep it current. Local governments periodically revise their comprehensive plans, both in their entirety and on a piecemeal basis. Minor small-area amendments may have been adopted in response to new information or circumstances or by another developer's or citizen's proposal. Because the governing body frequently acts on these amendments apart from a more extensive plan review process, interim amendments will usually not appear in printed versions of the plan. The planning office or town clerk is required to maintain an index and file of plan amendments.

Comprehensive plans with a multitiered structure most often provide a parcel-by-parcel analysis that offers detailed use, intensity, and design recommendations that apply to the development of each parcel. The identification and evaluation of these recommendations as they may affect a proposed development is one of the most important responsibilities of the land development consultant.

Jurisdiction-wide comprehensive plan maps and text usually limit their scope to broadly defined geographic areas and identify density or intensity ranges that apply within these areas. The more narrowly focused sector plan may attach a more specific range or absolute maximum density

**TABLE 2.1 Listing of Montgomery County General, Master, Sector, and Functional Plans**

The type and number of plans available from various public agencies which can aid in the early design stages of a project vary widely according to the municipality. For example, this list describes the types of plans available for Montgomery County, Maryland.

*General Plan:*

Identifies, in broad terms, those areas suitable for various types of land uses, e.g., residential, commercial, agricultural, open space, etc.

*Local Area Master Plan:*

Are available for each of the county's 27 planning areas. The local area master plans include a statement of planning policies, goals and objectives, and description of the planning area. Plans also include maps outlining recommended land uses, zoning, transportation facilities, recommended general locations for public facilities and possibly even include recommendations for scheduling financial capital improvements.

*Sector Plans:*

Are comprehensive plans for a portion of the Master Plan area. They describe the relationship of various land uses to transportation, services, and amenities. Plans may include maps or other graphics, text, and design guidelines.

*Functional Master Plan:*

Shows details of a specific system such as major highways, drainage, and stormwater management systems.

for a targeted area or parcel. Of particular importance are recommendations for project density or intensity or other conditions required to achieve full potential development. The plan may include techniques to reduce potential impacts on the neighborhood and community.

The land development consultant must understand how the community prepares its land use maps and the intent of their graphic representations. For example, a symbol proposing a regional shopping center site on the comprehensive plan's generalized land use map may not necessarily fix its location. The text may refer to specific circumstances or conditions that are favorable to another site in the general area of the symbol. Furthermore, specific land use maps, within the area or sector plans, may make explicit recommendations for specific parcels.

Land uses different from those shown on the maps are often permitted and encouraged, depending on circumstances and conditions identified in the text. For example, the map may show a broad geographic area identified as low-density residential, but text may explain that nonresidential uses, such as neighborhood shopping or service facilities, are appropriate. In such cases, the plan specifies conditions that the land developer must consider in developing such uses.

The land development consultant must review the discussion of available public facilities such as roads, water, and sewer. The existence of such facilities, system capacities, and timing of future improvements is important where these entities are needed to support the proposed development. In some communities, the adequacy and phasing of public facilities expansion controls development. For instance, Anne Arundel County, Maryland, adopts a sewer and water master plan as a functional element of its comprehensive plan. This master plan shows areas of existing service and planned expansion of service areas in 5-, 10-, and 20-year increments. Therefore, even when the comprehensive plan and actual parcel zoning agree on land use and density, the sewer and water master plan dictates lot size and project density. Consequently, the final project may be significantly less intense than envisioned, unless the developer can expand the system or defer the project to a consistent time frame with the adopted comprehensive plan. Policy language in the plan often prohibits the developer from constructing needed improvements to advance project development. Anne Arundel County uses this approach to discourage premature development in areas where growth is not anticipated or desired.

The land development consultant must identify the techniques and procedures that the local government uses to accomplish plan recommendations. These procedures may range from new or revised regulations or disincentives to discourage development to incentives that provide greater economic return to the developer in exchange for features that provide public benefit. The land development consultant must consider the fiscal impacts of these recommendations in determining project feasibility.

## ZONING

### Introduction

Despite the effort involved in adopting a comprehensive plan, legally, in most jurisdictions, it serves only as an advisor recommendation. In these majority jurisdictions, the plan does not have the force of law needed to protect the citizens' health, safety, or welfare and is not an exercise of a jurisdiction's police powers. That task falls to the community's zoning ordinance. The zoning ordinance is the legislative means by which a community sets detailed requirements to all aspects of the use of land. These requirements include layout and intensity of each parcel of ground within the government's jurisdiction.

Although zoning is the primary means by which the comprehensive plan is implemented, it is the comprehensive plan's recommendations that are the primary guide for making zoning decisions. While in majority jurisdictions a decision-making body may disregard the recommendations of the comprehensive plan, it does so at the risk of losing regularity and uniformity in its zoning practices. In its simplest form, zoning separates different or incompatible land uses and serves as a nuisance-prohibiting device. Its purpose is to reduce the likelihood of one use creating a nuisance or having an undesirable impact on occupants or future occupants of adjoining property. For instance, operation of a business such as a grocery store requires the installation of signs, lighting, parking, and trash disposal facilities and generates traffic from both customers and delivery trucks. If that grocery store operates next to a residence, the occupants' use and enjoyment of their property is diminished. The traffic, noise, activity, and lighting are likely to become a nuisance that infringes upon the residential character expected by the homeowner. As a residence, its location has become less desirable than that of others in the neighborhood. The business may reduce the value of the adjoining residence. However, it may also have the simultaneous effect of increasing the value of the land upon which the house is built by generating pressure to expand the commercial area and thus increase the value of the property. Zoning seeks to prevent such conflicts by providing greater separation between incompatible uses. Where greater distance cannot be achieved between incompatible use, the zoning ordinance requires other techniques to reduce impacts, such as a different architectural solution, smaller signs, a larger yard between the building and property lines, fencing, landscaping, or restricted hours of operation.

The zoning ordinance achieves separation by defining categories of land use that relate to agricultural, residential, commercial, office, and industrial uses. The ordinance is usually organized in a hierarchy that reflects intensity and operating impacts on the community. Typically, the zoning categories are further broken down into zones or zoning districts. These represent the differing types and intensity of uses possible within the broad category. The regulations

for each district specify what uses may or may not be constructed or undertaken on property within its boundaries. The most common zoning ordinances present an exhaustive list of land uses permitted in each zoning district. The zoning ordinance also regulates how much density and intensity of development are acceptable on property in each specific zone.

For instance, under the broad category of residential, an ordinance might specify districts for low-, medium-, and high-density development. Each zoning district would specify a maximum number of housing units allowed on each parcel of land. The text usually expresses this as a minimum lot size or number of dwelling units per acre. The ordinance may also list types of permitted residential development, such as single-family detached and single-family attached, garden or high-rise apartments. In addition, the ordinance sets standards for the size and arrangement of structures on each lot or parcel. These might include the maximum area of lot coverage permitted for buildings and the setback of those buildings from adjoining roadways and property lines. Zoning ordinances regulate maximum building height, distance between buildings, required off-street parking, and similar features. The community adopts these standards as a protection to adjoining property owners who rely

on the ordinance to anticipate potential conflict or impact upon their own property. These standards become an essential element in the design of a project. Figure 2.3 is exemplary of the detailed control language found in the typical zoning ordinance.

**The Zoning Map**

Zoning does more than just prevent conflicts between land uses. Through the adoption of a detailed map (an example is shown in Figure 2.4), zoning is also used to achieve the land use patterns endorsed in the comprehensive plan. The map that accompanies the zoning ordinance text shows the division of the entire jurisdiction into the zoning districts defined in the text. These zoning districts reflect the actual zoning of the land. Applications to rezone property are routinely viewed in terms of the specific recommendations embodied in the approved comprehensive plan. The zoning ordinance and the zoning map are both tools essential to the realization of the comprehensive plan's goals.

How communities create the zoning map varies. Many jurisdictions adopted the first zoning map simply as a graphic representation of land use patterns that existed at the time. Areas of vacant land were frequently zoned to a district similar to the surrounding area. In some commu-

	RESIDENTIAL DISTRICT REGULATIONS	3-403	3-403	FAIRFAX COUNTY ZONING ORDINANCE
<b>PART 4</b>	<b>3-400 R-4 RESIDENTIAL DISTRICT, FOUR DWELLING UNITS/ACRE</b>			
<b>3-401</b>	<b>Purpose and Intent</b> The R-4 District is established to provide for single family detached dwellings at a density not to exceed four (4) dwelling units per acre; to allow other selected uses which are compatible with the residential character of the district; and otherwise to implement the stated purpose and intent of this Ordinance.			B. Art and craft galleries C. Rooming houses D. Summer theatres
<b>3-402</b>	<b>Permitted Uses</b> 1. Accessory uses and home occupations as permitted by Article 10. 2. Dwellings, single family detached. 3. Public uses.			7. Group 8 - Temporary Uses. 8. Group 9 - Uses Requiring Special Regulation, limited to: A. Home professional offices B. Accessory dwelling units
<b>3-403</b>	<b>Special Permit Uses</b> For specific Group uses, regulations and standards, refer to Article 8. 1. Group 2 - Interment Uses. 2. Group 3 - Institutional Uses, limited to: A. Churches, chapels, temples, synagogues and other such places of worship B. Convents, monasteries, seminaries and nunneries C. Family day care homes; child care centers and nursery schools which have an enrollment of less than 100 students daily D. Group housekeeping units E. Medical care facilities, limited to nursing facilities which have a capacity of less than fifty (50) beds F. Private schools of general education which have an enrollment of less than 100 students daily G. Private schools of special education which have an enrollment of less than 100 students daily 3. Group 4 - Community Uses. 4. Group 5 - Commercial Recreation Uses, limited to: A. Commercial swimming pools, tennis courts and similar courts 5. Group 6 - Outdoor Recreation Uses, limited to: A. Golf courses, commercial 6. Group 7 - Older Structures, limited to: A. Antique shops		<b>3-404</b>	<b>Special Exception Uses</b> For specific Category uses, regulations and standards, refer to Article 9. 1. Category 1 - Light Public Utility Uses. 2. Category 3 - Quasi-Public Uses, limited to: A. Colleges, universities B. Conference centers and retreat houses, operated by a religious or nonprofit organization C. Cultural centers, museums and similar facilities D. Housing for the elderly E. Institutions providing housing and general care for the indigent, orphans and the like F. Medical care facilities, except nursing facilities which have a capacity of less than fifty (50) beds G. Private clubs and public benefit associations H. Quasi-public parks, playgrounds, athletic fields and related facilities I. Child care centers and nursery schools which have an enrollment of 100 or more students daily J. Private schools of general education which have an enrollment of 100 or more students daily K. Private schools of special education which have an enrollment of 100 or more students daily L. Alternate uses of public facilities M. Dormitories, fraternity/sorority houses, rooming/boarding houses, or other residence halls 3. Category 4 - Transportation Facilities, limited to: A. WMATA facilities
Reprint 9/88			Reprint 9/88	
	3-33			3-34

**FIGURE 2.3** Example of zoning district text.

RESIDENTIAL DISTRICT REGULATIONS	3-406	3-406	FAIRFAX COUNTY ZONING ORDINANCE
<p>4. Category 5 - Commercial and Industrial Uses of Special Impact, limited to:</p> <p>A. Commercial off-street parking in Metro Station areas as a temporary use</p> <p>B. Convenience centers</p> <p>C. Funeral chapels</p> <p>D. Marinas, docks and boating facilities, commercial</p> <p>E. Offices</p> <p>F. Plant nurseries</p>		<p>3-407</p>	<p>5. The minimum district size requirement presented in Par. 1 above may be waived by the Board in accordance with the provisions of Sect. 9-610.</p> <p><b>Bulk Regulations</b></p> <p>1. Maximum building height</p> <p>A. Single family dwellings: 35 feet</p> <p>B. All other structures: 60 feet</p> <p>2. Minimum yard requirements</p> <p>A. Single family dwellings</p> <p>(1) Conventional subdivision lot</p> <p>(a) Front yard: 30 feet</p> <p>(b) Side yard: 10 feet</p> <p>(c) Rear yard: 25 feet</p> <p>(2) Cluster subdivision lot</p> <p>(a) Front yard: 20 feet</p> <p>(b) Side yard: 8 feet</p> <p>(c) Rear yard: 25 feet</p> <p>B. All other structures</p> <p>(1) Front yard: Controlled by a 35° angle of bulk plane, but not less than 25 feet</p> <p>(2) Side yard: Controlled by a 30° angle of bulk plane, but not less than 10 feet</p> <p>(3) Rear Yard: Controlled by a 30° angle of bulk plane, but not less than 25 feet</p> <p>3. Maximum floor area ratio: 0.30 for uses other than residential</p> <p><b>Maximum Density</b></p> <p>Four (4) dwelling units per acre</p> <p><b>Open Space</b></p> <p>For subdivisions approved for cluster development, 15% of the gross area shall be open space</p> <p><b>Additional Regulations</b></p> <p>1. Refer to Article 2, General Regulations, for provisions which may qualify or supplement the regulations presented above.</p> <p>2. Refer to Article 11 for off-street parking, loading and private street requirements.</p> <p>3. Refer to Article 12 for regulations on signs.</p> <p>4. Refer to Article 13 for landscaping and screening requirements.</p> <p>5. Refer to Article 17 for uses and developments which require the submission of a site plan.</p>
<p>3-405 <b>Use Limitations</b></p> <p>1. No sale of goods or products shall be permitted, except as accessory and incidental to a permitted, special permit or special exception use.</p> <p>2. All uses shall comply with the performance standards set forth in Article 14.</p> <p>3. Cluster subdivisions may be permitted in accordance with the provisions of Sect. 9-615.</p>		<p>3-408</p>	
<p>3-406 <b>Lot Size Requirements</b></p> <p>1. Minimum district size for cluster subdivisions: 7 acres</p> <p>2. Average lot area</p> <p>A. Conventional subdivision lot: 8,800 sq. ft.</p> <p>B. Cluster subdivision lot: No requirement</p> <p>3. Minimum lot area</p> <p>A. Conventional subdivision lot: 8,400 sq. ft.</p> <p>B. Cluster subdivision lot: 6,000 sq. ft.</p> <p>4. Minimum lot width</p> <p>A. Conventional subdivision lot:</p> <p>(1) Interior lot - 70 feet</p> <p>(2) Corner lot - 95 feet</p> <p>B. Cluster subdivision lot:</p> <p>(1) Interior lot - No Requirement</p> <p>(2) Corner lot - 70 feet</p>		<p>3-409</p> <p>3-410</p>	
<p>Reprint 9/86</p> <p style="text-align: center;">3-35</p>		<p>Reprint 9/88</p> <p style="text-align: center;">3-36</p>	

FIGURE 2.3 (Continued) Example of zoning district text.

nities, the adoption of the comprehensive plan is followed by a comprehensive, jurisdiction-wide rezoning. In such cases, the zoning map closely mirrors the land use patterns recommended in the comprehensive plan. In this way, the community encourages development to follow the plan recommendations because a rezoning of the property is not required. The developer need only process project plans in accordance with other development controls, such as subdivision and building permit regulations. For jurisdictions that do not practice comprehensive rezoning, the zoning category assigned to vacant property often represents the least intense land use. This is often a low-density residential category. Any more intense use requires that the developer seek a rezoning to a higher classification. Chapter 11 addresses rezoning and the map amendment process in more detail.

### What to Look for and How to Use the Zoning Ordinance and Map

The zoning ordinance is usually a separately published title or chapter in the jurisdiction's code of laws. The land development consultant must be sure to use the most current version of the zoning ordinance. Many communities adopt text amendments on a continuing basis as the government

responds to a variety of problems and constituents' issues. Most jurisdictions publish periodic text amendment updates, with a page numbering system designed to ease insertion in a master document. These are sometimes available on a subscription basis. Others reprint the ordinance in its entirety only after the governing body adopts a substantial number of amendments. This requires that the land development consultant maintain a file of amendments until the reprint is available.

The zoning map is often available in several forms and scales. A generalized zoning map is usually a single-sheet depiction of the community, perhaps at a scale of 1" = 2000'. The map shows major roadways, landmarks, and natural features, with zoning district boundaries identified by various symbols. Each district is labeled with the name of the zone that corresponds directly to the ordinance text. Frequently, the district name is a shorthand indication of the use and intensity of the district. While the meaning of these labels is consistent within a single jurisdiction, they often vary between jurisdictions. For instance, a district labeled "R-5" might be a residential district with a permitted density of five dwelling units per acre, or it may mean a residential district permitting minimum lot sizes of 5000 ft<sup>2</sup>. Or it may simply be the fifth in a hierarchy residential



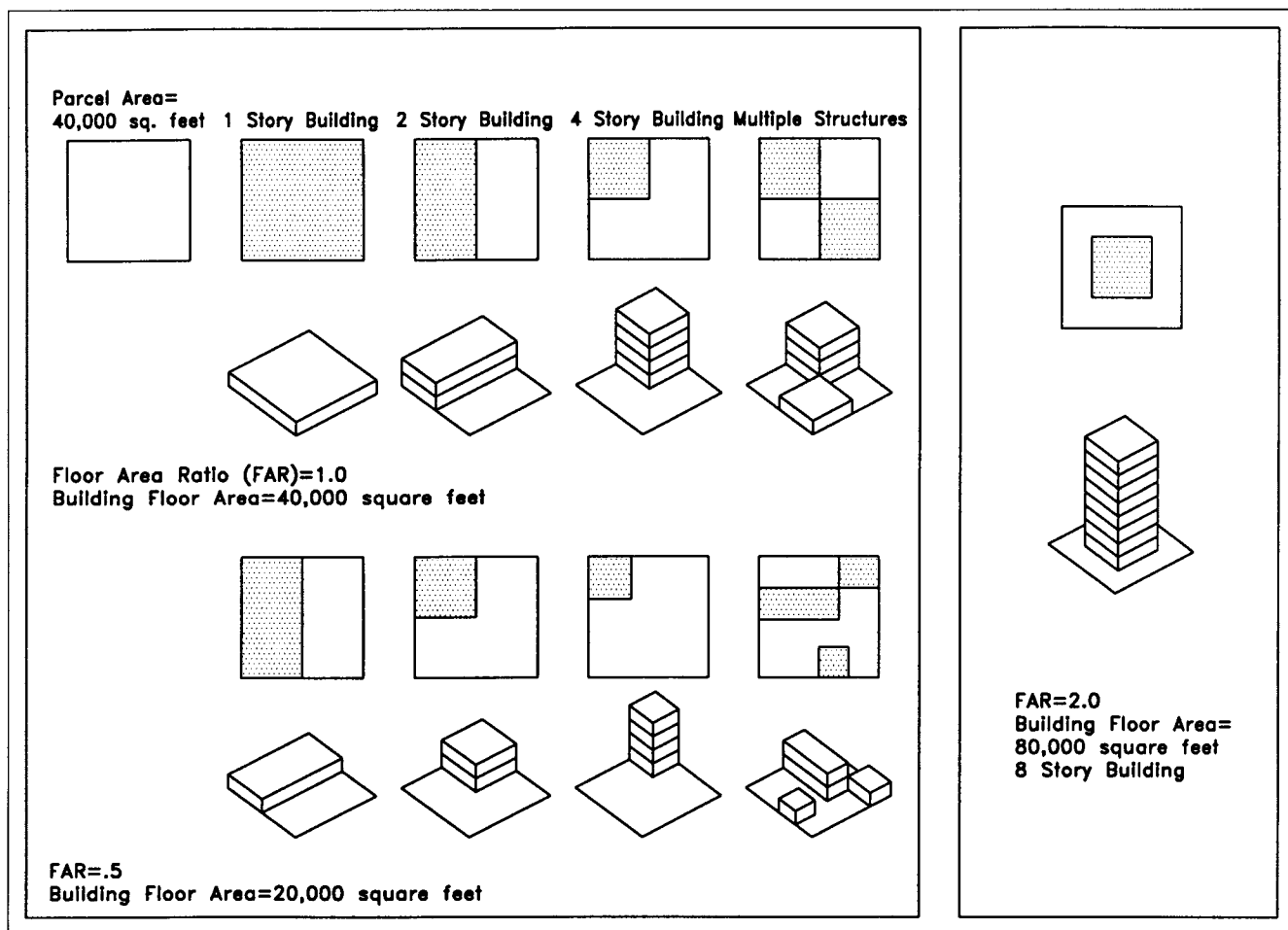


FIGURE 2.4 Graphic illustration floor area ratio (FAR).

district. Although this type of map is useful for understanding broad zoning and land use patterns in the community, it is less useful in verifying actual parcel zoning because the map in some jurisdictions omits property lines. In addition, zoning lines frequently bisect properties.

A large scale map, often at 1" = 200', may be maintained by the planning or zoning office, town clerk, or similar office of records. Maps at this scale will frequently include parcel boundaries or may reference a tax map designation for the property. In some jurisdictions, the official map shows additional information, such as previous and pending zoning special exception or special permit applications as well as subdivision activity.

Some jurisdictions use metes and bounds or other description to maintain zoning district boundary records. Compare these to actual parcel boundaries to verify property zoning.

Eliminate doubt! Have the appropriate government agency certify parcel zoning in writing. In some jurisdictions, a certification of zoning may become a legally binding decision unless appealed.

Check the jurisdiction's practice about the zoning of street and railroad rights of way, public and institutional

properties, and major water bodies. Are these areas zoned, and do zoning boundaries fall on centerline or on property line? This is important in calculating density for projects where right-of-way abandonment or surplus public property disposition is a factor.

The text associated with each zoning district will usually describe the purpose of the zone. This determines the intended magnitude and character of the development, as well as the intended service or market area of nonresidential uses. In addition, the text will provide a listing of land uses allowed in the district. These include:

- Uses that are permitted by right
- Uses that are permitted by right if certain specified conditions apply
- Uses that are permitted by special exception
- Uses that are permitted by special permit
- In some cases, uses that are prohibited in that zone

In undertaking a project where the use is allowed by right, the land developer need only show compliance with

the requirements of the zoning ordinance. This usually applies whether the development is a single home, an entire residential subdivision, or a commercial facility. To secure approval from the jurisdiction, plan review is accomplished through the normal administrative procedures incorporated in the subdivision ordinance and building permit procedures. These ordinances are discussed later in this chapter.

When the intended use requires review as a special exception or special permit, approval authority usually rests with an elected or appointed body, or both. These are most frequently the jurisdiction's governing body, its planning commission, a zoning appeals board, or a zoning hearing officer. In addition to a review by the professional planning staff, the governing body usually holds public hearings to assess the attitudes of neighbors and civic groups towards the project. Using plans prepared by the land development consultant, the developer must justify the appropriateness of the proposed use. The approving authority may impose conditions to lessen potential impact, whether perceived or real. Following approval, the normal administrative procedures for subdivision and construction will apply.

Uses requiring approval of a rezoning or special exception application must conform to the recommendations incorporated into the comprehensive plan. The zoning ordinance provides details on the administrative and legislative approval procedures for these development proposals. These specify the required information needed for the preparation of plans and supporting documentation for rezoning and similar applications. Ordinance provisions also provide the methodology for amendments to the zoning ordinance text. Text amendments can change development potential as much as map amendments. Both amendment procedures may be initiated by either the governing body or individual appellant. Chapter 11 describes these procedures in more detail.

Minimum or maximum standards cover virtually every detail in project design for each type of land use. Typical criteria identified in the zoning district text include lot size, density, coverage, and bulk requirements. These requirements strive to protect and preserve the character of the environment by preventing overcrowding, reducing conflicts between neighbors and neighboring properties, and guarding against encroachment on streets and intrusion of traffic noise. Examples of these are listed below.

- Minimum of average lot size and lot width.
- Maximum density or intensity of development, usually expressed in dwelling units per acre for residential uses and floor area ratio (FAR) for nonresidential uses. Floor area ratio is the relationship of building area to parcel size. For example, on a 40,000 ft<sup>2</sup> parcel, if the zoning ordinance allows a maximum FAR of 1.0, then it would be possible to construct a building or buildings totaling 40,000 ft<sup>2</sup> of floor area. This could result in a single-story building covering the en-

tire parcel, a two-story building covering half the parcel, or some other configuration. Yard and height requirements affect the actual parcel layout. An FAR of 0.5 for this same parcel would allow 20,000 square feet of building area. Figure 2.4 is a graphic depiction of FAR.

- Maximum lot coverage, either by buildings or by impervious surface, often expressed as a percentage.
- Minimum required open space, again as a percentage, referring to parcel area unencumbered by buildings or parking.
- Minimum front, side, and rear yard requirements that define the building envelope. These refer to the distance of building setback from property lines, including street rights of way and waterfront. In some communities with waterfront property, the yard facing the water is considered the front yard. The ordinance may specify when structures or architectural features, such as porches and steps, may extend beyond the setback line. The front setback line is also called the building restriction line.
- Minimum lot width at the building restriction line. If the lot is shaped so that its width at the normal setback line does not meet the minimum standard, the building restriction line is forced to be located farther from the front property line, to a point where the width complies with the standard.
- Maximum or average building height, in feet and/or stories.
- Angle of bulk plane or similar representation of relationship of building height to property line. This protects adjoining property views or access to sunlight and airflow. Figure 2.5 illustrates angle of a bulk plane as applied to building restriction lines.

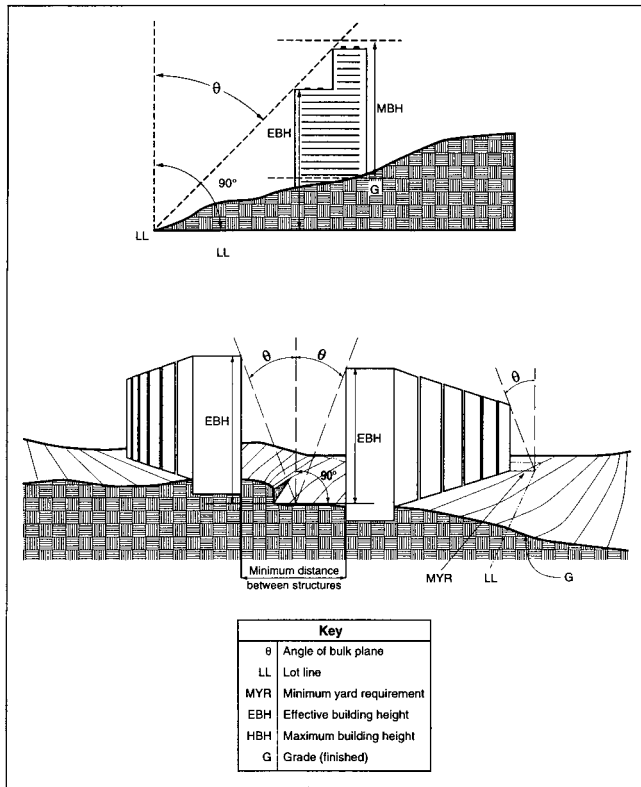
The ordinance text identifies general criteria that might apply to all development, such as required setback from highways and railroads or other unusual conditions.

The ordinance describes the functional requirements or standards applicable to all development. These include off-street parking, landscaping, screening, buffering, and tree preservation. These are frequently located in separate subtitles of the ordinance.

Special exception or special permit uses have minimum standards and criteria listed separately in the text. Identify criteria for size and intensity, proximity to other properties or facilities, landscaping, screening, and similar conditions that mitigate impact.

### Zoning Administration

An important principle of zoning is that a property can be developed to any land use that is allowed by right in the district, provided that the development meets district stan-



**FIGURE 2.5** Graphic showing setback and building restriction lines and angle of bulk plane.

dards and complies with other development policies and regulations. By-right uses are those that are explicitly permitted in each zoning district and are fundamental to the district's character. How those uses are determined differs, as does the subsequent process of development review by the jurisdiction. The land development practitioner must understand how the local government applies its ordinance. The variations dictate what land use options exist and how project design is undertaken and affect the way in which local government approves development projects and the durability of that approval. A special exception is a specific use permissible within a zoning district subject to the granting of a site-specific permit request. A special exception is one that generates additional impacts to the surrounding neighborhood and warrants special consideration. Despite the potential for impacts, the principle underlying the classification of special exception uses is that, if properly regulated, they are acceptable uses in the zoning districts where they are permitted. They adhere to the character intended by the zoning district, as well as the comprehensive plan. They may be desirable to the community and add to its convenience, but due to possible impacts such as noise, traffic, or visual disruption, most zoning ordinances establish conditions that apply in the development of special exception uses. Such conditions may include a limitation on the hours of operation, landscaping, buffers and barriers

that exceed the standard specified for the underlying zoning district, and a specification of the activities permitted in conjunction with a special exception use. The authority empowered to grant rezoning approval is usually the same one that grants special exception approval as well. As with rezoning, this may be the governing body, a planning commission, or a zoning hearing officer. These bodies also hold public hearings to solicit input from community residents.

In contrast to uses allowed by right, most ordinances specify that uses needing special exception approval must comply with the community's comprehensive plan. These requirements subject development proposals needing a special exception to the broad variety of policies and restrictions in the plan that might not otherwise apply. These could include density controls, requirements for specific environmental measures, constraints on traffic generation, or desirable public improvements. Frequently, these apply even to existing land uses proposed for redevelopment or expansion of long-established uses. As with rezoning applications, because of this direct connection to the requirements of the plan, many local governments try to expand the application of special exception procedures to a wider range of uses.

Some jurisdictions administer a separate special permit process. Projects that involve uses listed as needing a special permit must go through an approval process not unlike the special exception. However, the governing body frequently delegates its authority to a board of appeals or zoning staff administrator. Approval of a special permit comes with conditions attached that, like a special exception, help to reduce incompatibility between the use and adjoining neighborhood. Typically, opportunities for public hearings are incorporated into the process, allowing affected neighbors to voice support or opposition to the issuance of the permit.

A variance to zoning ordinance requirements may be granted by the governing body, a subordinate body to whom that power has been delegated, or, in some jurisdictions, a quasijudicial body appointed by judicial officials. As with other zoning applications, opportunities for public comment are normally provided. Most variance applications address building setbacks or similar technical requirements of the ordinance. These usually occur in expanding or reconstructing existing land uses, where changes to the zoning ordinance make it difficult to adhere strictly to the current ordinance. In some instances, complying with newly adopted ordinances imposes a hardship on the property owner, who is otherwise unable to use the property in the manner permitted prior to the change in the zoning ordinance.

A variance should not be confused with rezoning, although a few jurisdictions allow use variance applications as well as the more common variance to standards. A variance may be granted if the applicant can prove that, because the owner is unable to comply exactly with the

zoning ordinance, an undue hardship exists. The hardship must be peculiar to the property, and the variance must be necessary to allow reasonable use of the land. A purely economic hardship does not normally satisfy this hardship test. The approving authority may impose conditions on development. In granting the variance, it must ensure that the variance will not change the basic character of the area or weaken enforcement of the ordinance. Financial loss is usually not considered a sufficient factor in determining hardship.

A nonconformity is an existing lot, use, or structure that does not fully comply with the requirements of the zoning district in which it is located. This situation often occurs a new zoning ordinance is adopted, text or map amendment is made after a subdivision plat is recorded, or a land use is legally established. In most instances, the goal of the community and governing body is to eliminate nonconforming uses or structures. Where that is not practical, most ordinances also include provisions that explain the conditions under which such uses may continue to operate or expand.

There are two types of nonconformities, nonconforming as to standards and nonconforming as to use. The first case addresses uses permitted in a district whose development standards have since been revised in the ordinance. This frequently occurs in older residential areas or established commercial areas when the lot size, density, parking, or setback requirements have been changed in the zoning ordinance. Most ordinances usually allow these uses to continue, provided they are not expanded or enlarged, but require that future additions comply with the new requirements. For instance, a new garage added to a house would have to comply with new setback provisions, unless a variance is granted. A developer constructing a new department store addition to an existing strip shopping center might be required to adhere to the newer off-street parking requirements. In a zoning district requiring lot sizes of one acre, the zoning ordinance will usually allow a house to be built on a smaller existing lot of record; however, the structure must otherwise fulfill all other requirements of the ordinance.

Sometimes the actual use of a property becomes nonconforming due to a text amendment eliminating the once-permitted use. Typical ordinance provisions allow the use to remain but prohibit any expansion or enlargement. Repairs and limited reconstruction is allowed, although most require a nonconforming use to be terminated if a substantial, greater than 50%, portion is destroyed by fire or other casualty. Some communities also adopt a schedule to enforce the removal and/or amortization of nonconforming uses that they consider undesirable, such as billboards or mobile homes.

### Zoning Concepts

Zoning ordinances fall into two general classes, although many communities' ordinances have evolved into a hybrid.

Among these two types of zoning ordinances, there are several important distinctions in ordinance structure and administration. Conventional ordinances, often called prescriptive or Euclidian<sup>1</sup> zoning, set rigid parameters. These prescribe very specific land uses and building configurations that are permitted on a parcel of ground and the standards applied to development. Performance zoning incorporates flexibility into land use control. This type of zoning addresses more the impacts of the final development and use of a parcel, based upon its unique physical and environmental characteristics. Rather than imposing strict standards, such an ordinance requires that the developer meet specific standards of performance. These standards often relate to stormwater runoff, industrial pollution or noise, traffic, population, or other factors important to the community. Like Euclidian zoning, a performance zoning ordinance usually specifies use and intensity. However, with performance zones, the land developer has greater latitude in determining the detailed requirements, such as lot size and building arrangement. To secure project approval, the developer must show how the proposed project meets the performance standards.

Conventional Euclidian zoning has the advantage of greater certainty in both end product and project approval, but it suffers from rather uniform developments that often result. This is particularly true in residential uses, where large developments commonly have a cookie-cutter appearance. This approach to zoning does not allow the flexibility that a developer needs to address environmental concerns in site development. Nor can the developer respond to shifting market activity or preferences. In addition, the number of zoning categories and individual zoning districts is greatly increased with Euclidian zoning.

Performance zoning, on the other hand, provides flexibility and often succeeds in attracting more variety and innovative projects. Greater protection of environmental resources and open-space areas can be achieved through the performance zoning process. Its disadvantage, however, is that it may result in greater expense in upfront analysis for both the community and the developer. To community residents, it sometimes does not provide the desired clarity and certainty with respect to expected development.

**Cumulative Zoning.** Variations of this type of zoning may be most common. As the density and land use intensity increases in the zoning district hierarchy, the range of permitted uses expands. Thus, a more intense district will permit all uses listed within that district, plus all uses in districts of lower hierarchy. One of the shortcomings of cumulative zoning is that the community may not attain the desired separation of uses and may suffer the resultant im-

<sup>1</sup> Named after the landmark 1926 Supreme Court decision in the case of the *Village of Euclid, Ohio v. Ambler Realty Company*, 272 U.S. 365 (1926), which established zoning as a valid exercise of police powers to protect public health, safety, and welfare.

pacts caused by lack of separation. In a commercial district, for instance, it could be possible to construct single-family residences.

While separation is one of the basic intentions of zoning, economic and market forces usually work to discourage significant disparity in use. The principle of highest and best use generally works to discourage development to a use with lower economic return. Full development potential is the most widespread indicator of land prices. In addition, marketability of the lesser use declines if users perceive the proximate higher use to be incompatible or an undesirable neighbor. In some respects however, the mix of uses permitted by this type of zoning may be beneficial because the extreme separation of residences from employment or services has, in large part, been responsible for the rapid growth in suburban sprawl and increased dependence on the automobile. This dependence has worsened roadway congestion by forcing more traffic onto major roadways. Those communities that recognize this respond by establishing mixed-use districts that encourage a variety of compatible uses. Strong government controls usually accompany the development of mixed-use districts. However, even as some jurisdictions attempt to encourage mixed use, many local governments now try to narrow the range of permitted uses in each district. The rationale for this position is the need for greater certainty in planning for the community's infrastructure.

**Transitional Zoning.** Many communities draw zoning district boundaries in a step-down fashion. This pattern seeks to provide gradation between two disparate uses, usually residential and commercial or office. In one approach, the community may create zoning districts of intermediate intensity between the incompatible classifications. This reduces the most significant conflicts between the more extreme uses. Figure 2.6 represents a zoning map with property divided into several districts between the most intense commercial zones and least dense residential zones. The permitted intensity of commercial use reduces in each of several districts as distance from the most intense commercial area increases. Similarly, residential density may also increase for properties in districts located closest to commercial areas.

Some jurisdictions establish districts that provide for flexibility as well as a mix of uses and appearance where zoning classifications meet. These transition zones serve as buffers against the intense use. For example, townhouse-style offices may be required on commercially zoned property adjacent to residential areas. This encourages architectural compatibility in both scale and configuration with the existing residential community. The ordinance may also provide for changes in housing type; for example, townhouses or garden apartments may be allowed as part of the transition. In either case, the text of the zoning ordinance often imposes additional landscaping and screening requirements as additional protection for the lower density/intensity uses.

**Floating Zones.** Many ordinances describe zoning districts that are unmapped but become fixed in the process of rezoning. These zones have a special purpose and most usually respond to a specific comprehensive plan recommendation. Floating zones may provide transition between uses, as described above, or they may address problems in geographic areas that have unique opportunities or constraints, such as an environmental resource or commercial or transportation center. Planned unit development is a typical type of floating zone, although communities apply it in varying ways.

**Planned Unit Development.** This is a development form that attempts to enhance both flexibility and functional relationships between land uses. With conventional zoning patterns, the land developer cannot easily respond to the changing market demand associated with shifts in the economy of the region. Nor can the developer address consumer preferences for other than a single lot size or housing type. Individual projects, undertaken in a small geographic area by different developers sometimes lack cohesiveness, causing community identity to suffer. In addition, the extreme separation of land uses that results from conventional zoning, often places unnecessary stress on community facilities.

The planned unit development, a mix of land use, building type, and intensity can be constructed within a single project. It is most often applied in the development of larger tracts of land, undertaken by a single land developer. Typically, a planned unit development can include single-family detached, single-family attached, multifamily housing, and light neighborhood serving commercial retail uses. In larger projects, local zoning regulations may permit other more intense uses.

Planned unit development allows the land development consultant the opportunity to consider design in a comprehensive manner, rather than treat each use individually and separately. Planned unit development also presents design opportunities to address broad functional relationships, both between land uses and between land use and facilities. These include entrances to the community, vehicular and pedestrian traffic, stormwater management, and other systems. The land development consultant has greater control over placement of uses, their design, and access between them. In-depth discussion on these issues is found in Chapter 1.

Unlike those in conventional development, the uses in a planned unit development interrelate in a common and controlled setting. The consultant addresses elements that affect community appearance and identity with a unified approach including architectural style, landscaping, street furniture, open space, and signage. This comprehensive approach to design and construction of planned unit developments allows for a more efficient use of the land. In addition, the developer is in greater control of product marketing and the business environment. Due to this design flexibility, cost savings in streets and utilities design, con-

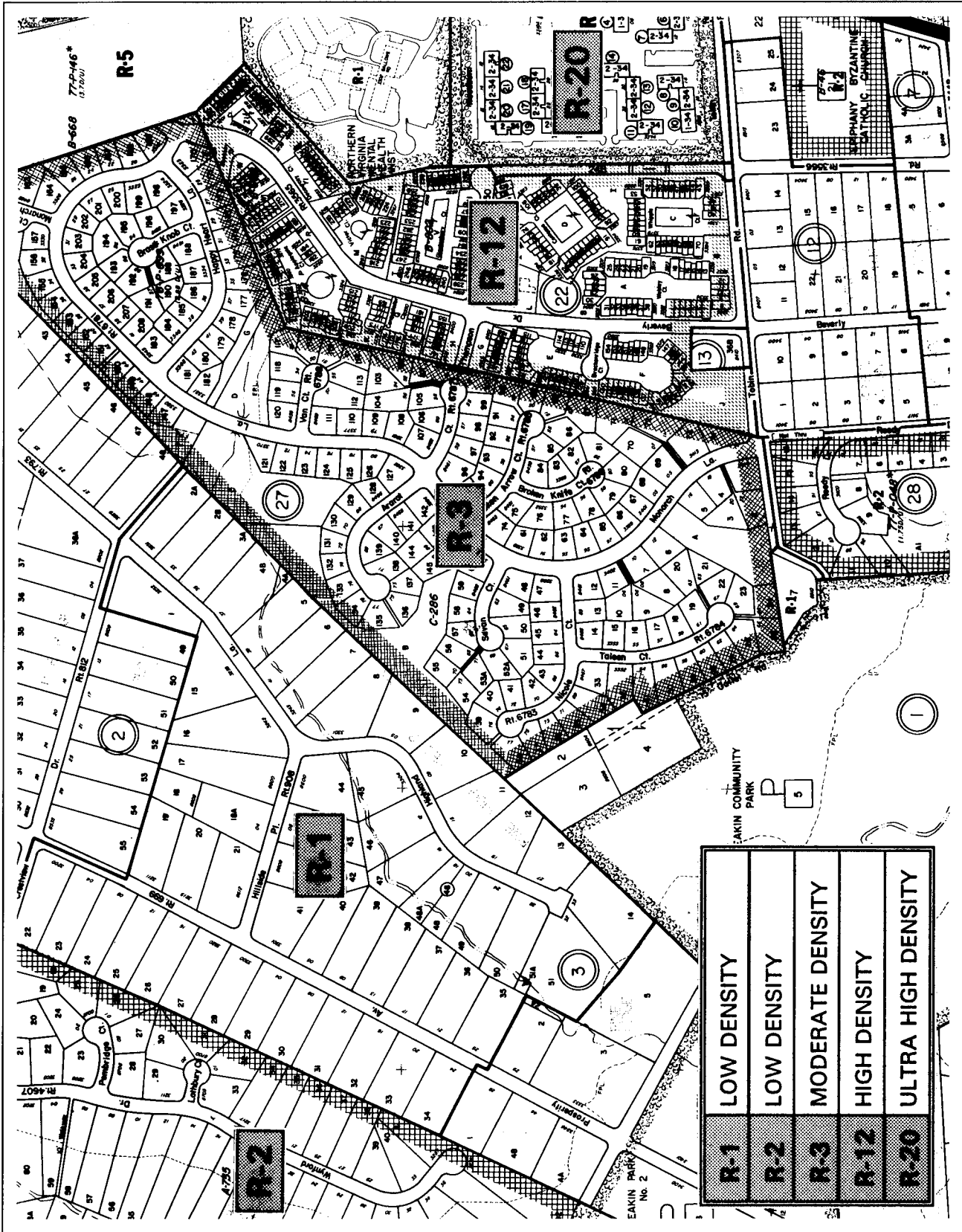


FIGURE 2.6 Zoning map showing zoning transition.

struction, and maintenance may be a result to benefit both the developer and public agencies.

Some communities delineate planned unit development districts on their zoning maps. Ordinance provisions set use and intensity conditions that reflect the district's relationship to other uses. Other communities have the zone float. It becomes fixed during the rezoning process, frequently where the applicant has assembled several properties and the project may take on the general characteristics of the underlying zones. A planned unit development in an otherwise residential area may still be required to incorporate primarily residential uses. However, housing type and arrangement may deviate from conventional district requirements. Often, limited neighborhood serving retail areas will also be allowed. Planned unit development provisions may also address areas intended to be primarily shopping centers and industrial and office parks. Some jurisdictions allow planned unit development as a special exception that most closely resembles the floating zone, with the processing and administration differing among jurisdictions.

Conversely, the flexibility associated with planned unit development often means that the final development may be viewed as less predictable to the community. Consequently, planned unit development approvals often involve extensive review procedures that might discourage their application. Considerable up-front design costs may be encountered by the developer, with no guarantee of project approval. Frequently, planned unit development provisions also require the developer to provide additional on-site amenities, such as open space, recreational facilities, and land for public facilities such as schools and fire stations.

However, many jurisdictions recognize the benefits of planned unit development to the community. Zoning ordinances often provide land developers with incentives to assemble smaller parcels of land into larger tracts. These incentives come in the form of higher densities, relaxation in lot size requirements, more intense uses, and increased flexibility in design.

**Cluster Zoning.** Cluster is an alternative control for residential developments. Like planned unit development, it is an attempt to allow greater variety and flexibility in land development. Cluster regulations frequently allow smaller lots in exchange for more community open space and a more pleasing community setting. At the same time, cluster can provide greater public benefit, such as preservation of sensitive environmental features. Cluster has also been successfully applied to save historical sites and productive farmland.

Cluster differs from planned unit development in that it is a change in neither land use nor housing type. If the underlying zoning district specifies single-family detached residences, townhouses or apartments usually cannot be built. Those cluster ordinances that do allow a change in housing type clearly identify the conditions and requirements that apply. This is merely a reconfiguration of the technical lot size and setback standards that apply in

subdivision design. The density specified in the zoning ordinance is unchanged. Communities that allow cluster development by right set specific standards that are applied in subdivision or other administrative ordinances. This gives the development team the flexibility to design a plan that can accommodate unique site conditions or address market desires. As a result of the design flexibility, the developer can realize construction cost savings using cluster development. The finished product reflects these savings. Final purchase prices can be reduced, or site amenities and housing design features can be added that would normally be more expensive to the consumer.

Despite this, some communities that once allowed cluster by right now require either rezoning or special exception approval for cluster development. Where this change has been made, local legislators have acted in response to citizens' fears that the smaller lots somehow diminish adjoining property values. Others are concerned that cluster may allow the developer to create more lots than otherwise possible, despite the overriding control of zoning district density. The delay and exposure, in terms of both time and money, associated with these processes often serves to discourage land developers from using cluster development, despite its benefits. The development team must balance these competing demands. Chapter 13 describes cluster development design in detail, including a graphic comparison of the differences between the traditional and cluster zoning.

**Overlay Zones.** Communities sometimes use overlay zones to address areas with special problems or unusual physical characteristics. In addition to the conventional district boundaries, overlay zones are either mapped or described in the ordinance text. Typical situations that suggest the use of overlay controls include airports, historic areas, highway corridors, and floodplains. The development must conform to both sets of zoning district criteria, or the more restrictive if conflicts are apparent. Figure 2.7 provides a graphic representation of a typical overlay map.

**Transferable Development Rights (TDRs).** Historic, agricultural, and similar resources are often prime development sites. Most communities place a high priority on the protection of these properties and often attempt to discourage their development. Consequently, owners of these properties could not realize the value of these properties if development were to be prohibited. Therefore, some communities have adopted a system that enables the transfer of development potential elsewhere in the community, while leaving the property itself undisturbed. These development rights are usually sold to developers of property elsewhere in the jurisdiction, allowing the original landowner to keep the underlying property and profit from its value. This practice also allows the jurisdiction to focus its financial resources for public facility improvements in its more developed areas, where they can efficiently serve the population. It should be noted that TDRs involve a land regulatory process outside the scope of traditional zoning

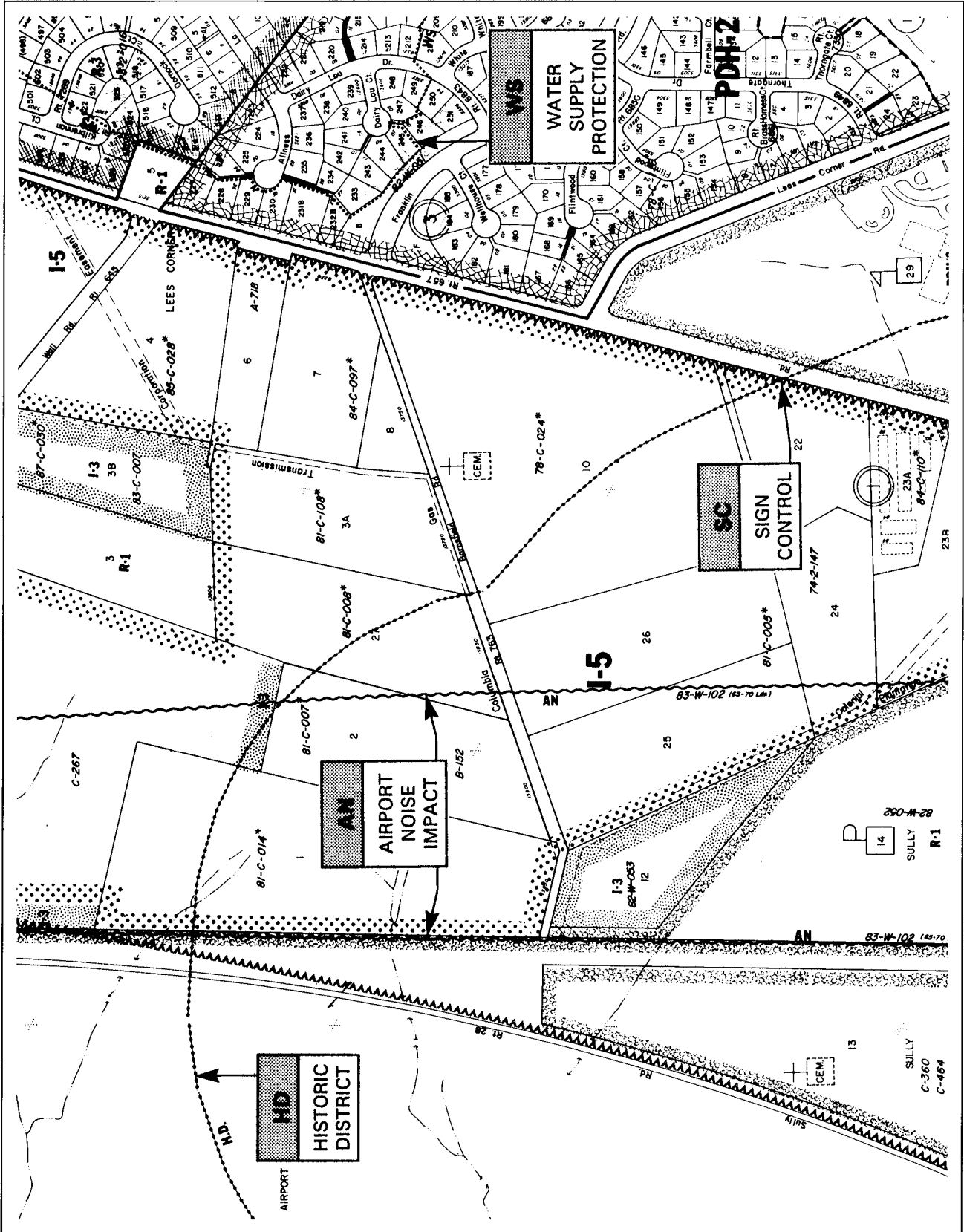


FIGURE 2.7 Example of an overlay zone on a map.



enabling acts. Therefore, special legislative authority is normally required to legitimize TDR regimes.

In its comprehensive plan, the community defines sending and receiving areas. Sending areas are those areas to be preserved. Underlying zoning or a similar indicator determines the development potential of the property. Property development rights are made available for sale to landowners in receiving areas. Selling the development rights of a property prevents the sending property from undergoing future development.

Developers who acquire TDRs may increase project intensity accordingly. Maximum densities and other conditions established in both the comprehensive plan and the zoning ordinance regulate development of the receiving property. This type of zoning most closely resembles the floating zone. Additional development rights need not be purchased, but new criteria apply when they are. Development conditions in the community's ordinance protect property owners near the receiving area, who may face greater impacts and higher densities than anticipated.

**Exclusionary Zoning.** Exclusionary zoning specifically prohibits a particular use from a district that might otherwise permit it. This prevents a use that might have extreme impact, such as noise, odor, or pollution. However, the more familiar use of the term has a negative connotation that applies to ordinances that limit housing choices for low and moderate-income families. Exclusionary zoning may restrict large lot zoning, require minimum house sizes, prohibit mobile homes, or exclude apartments. Such ordinances indirectly discriminate against minority and economic groups, and, as such, exclusionary zoning is becoming the subject of court challenges. Coupled with high land prices and development costs, exclusionary ordinances also work to restrict a community's labor force. By limiting housing opportunities for office and service workers, they force employees to commute greater distances, which places an increased burden on regional transportation systems.

**Inclusionary Zoning.** Inclusionary zoning tries to expand access for households of low and moderate income. This is accomplished by adopting provisions that encourage developers to provide housing and facilities in affordable price ranges. The simplest way to achieve this is to amend the zoning ordinance to allow greater variety of housing opportunities other than single-family homes on large lots. However, because of other factors that affect housing cost, this is not necessarily effective in and of itself.

One approach that is gaining in popularity is the adoption of so-called affordable dwelling unit ordinances. These require the developer to provide for a certain number of controlled-price dwellings in exchange for a density bonus. The additional density compensates the developer for the added construction cost of the affordable units. To implement such an ordinance, the community must modify zoning district design requirements to accommodate the increased number and type of units that may be constructed on the site. These changes most frequently include lot size,

setbacks, and maximum building coverage and housing type. Take the example of a 100-acre site that is zoned to permit a density of four dwelling units per acre. If a 20% density bonus is granted, it may not be physically possible or desirable to build 480 single-family detached homes on the property. Modifying the ordinance to allow a percentage of the units to be built as townhouses means that a more attractive project may be possible and still allows the developer to earn a profit, while accommodating the affordable units.

**Zoning Changes.** While Chapter 11 discusses the actual rezoning process, discussion about the effect of zoning change is presented here. In order to secure rezoning to a different classification, the land developer uses plans and applications prepared by the land development consultant and files an application or request for legislative change with the local governing body. These materials describe the proposed rezoning and demonstrate the rationale for the change. The application often represents that the developer intends to construct a specific use on the subject property. The plans and drawings show how the project will look once built. If the governing body approves the rezoning, future development of the property is controlled by the new district regulations.

However, some communities have found that while the developer is bound to the new zoning, there may be no obligation to develop the property as shown in rezoning application. Consider the example of a commercial zoning district that permits both an automobile service station and a lumberyard. The developer may already have a service station operator as a client, to whom he intends to sell the property once it is rezoned. During the rezoning process, the developer may have successfully demonstrated that the service station was both appropriate and needed in the community. The land development consultant prepared the client's plans showing how the final product would look once completed. After the zoning is approved, the client of the developer may build the service station, but backs out of the project for economic reasons. In view of the fact that the zoning district allows a variety of uses by right, the developer is free to establish any of the land uses that the district permits. He or she may sell the property to another developer, who finds that construction of a lumberyard may be more appropriate. This flexibility enables the developer to respond to changing market conditions. However, community residents may find the lumberyard incompatible with their community and lead to further commercialization of the neighborhood.

In another situation, the comprehensive plan may recommend that an area zoned for low-density residential use is more appropriate as a multifamily project. However, the increased density will create the need for new roadways to serve the community, funding for which the local government cannot afford. When the rezoning is approved, residents of the jurisdiction become frustrated because traffic congestion is worsened by new projects in the area. These

scenarios have led many communities to seek the adoption of state enabling legislation to create several hybrids. These are characterized by the granting of rezoning to districts of higher density or more intense use in exchange for greater certainty, increased amenities, or facilities that serve the public health, safety, and welfare. These typically involve extensive negotiation between the developer, the governing body, and, in some instances, the citizens of the community.

These types of ordinances, and rezonings granted under their provisions, are occasionally the subject of court challenges on constitutional grounds. Citizens sometimes believe that the governing body has bargained away too much, and developers are concerned that they have been unfairly treated. Therefore, a local land use attorney frequently takes a lead role on the land development team. However, the land development consultant should become familiar with the administration of local ordinances, past practices, and existing case law.

In states that allow conditional zoning, the governing body has the power to grant a rezoning if the developer agrees to meet certain conditions. Most often, the local government imposes or allows the developer to offer these conditions voluntarily to reduce the impacts of the proposed use or existing uses and future public facilities. Conditions can improve compatibility between adjoining uses or reduce the project's demand on support facilities and infrastructure. The developer is compensated for the extra cost of these conditions by the higher economic returns expected from a project of greater intensity or density. Critics of conditional zoning suggest that it may be abused by the governmental agency to obtain numerous concessions while exceeding reasonable expectations of the developer.

**Proffered Zoning.** Used extensively in the urbanizing portions of Virginia, proffered zoning is similar to conditional zoning but mandates that the development conditions be volunteered in writing by the developer prior to action on the rezoning request. The significant difference is that the developer proffers (pro-offers) to bind development of the property to a specific development plan or set of textual conditions. In addition, the developer sometimes offers to construct improvements to public and private facilities that reduce demand generated by the development. The developer may also provide operational and aesthetic incentives to the community as a condition of rezoning approval. The final plan and proffers are usually the result of extensive negotiation with community residents and the governing body.

The proffered conditions must be in writing, be signed, relate to the rezoning itself, and be in accord with the comprehensive plan. If the governing body accepts the proffers and approves the rezoning, the proffered conditions then run with the land. Thus, both current and future owners or developers are bound to the approved plan and conditions agreed to during proffer negotiations. Any change to the development plan or the actual proffers must be accomplished through a subsequent rezoning.

**Contract Zoning.** Unlike conditional zoning, which binds only the landowner, contract zoning binds both the landowner and the governing body to agree on conditions in exchange for certain zoning concessions. Some courts have ruled this type of zoning invalid, arguing that the governing body does not have the power to bargain away its police powers. This form of rezoning can result in spot zoning to accommodate private interests and is often challenged as being capricious and arbitrary.

## RELATED TECHNIQUES IN DEVELOPMENT REGULATION

### Adequate Public Facilities Ordinances

As noted earlier, some communities have found that rapid growth has outstripped their ability or commitment to provide the public infrastructure needed to support development. One technique used to address this problem is the adoption of an adequate public facilities ordinance. Such an ordinance requires that the developer defer a project if the projected demand on public services exceeds the total capacity or funding approval of any of the systems. The typical systems covered by an adequate facilities ordinance include transportation, water, sewer, and school systems. As with the TDR's process, adequate public facilities regulations involve a land regulatory process outside the scope of traditional zoning enabling acts. Consequently, special legislative authority is normally required to legitimize adequate public facilities ordinances.

Implementing such an ordinance first requires that the governing body conduct studies to determine the total capacity of public systems. This is derived by adding to the capacity available in existing systems those facility improvements that are planned, budgeted, or funded within a specified period. Most ordinances use the time span covered by the community's capital improvements project budget, typically four to six years. For some systems, capacity can be determined by direct measurement, such as by measuring design capacity for sewage or water treatment. For others, a policy must be established concerning level of service before the calculation of projected use is made. For instance, the governing body must determine the number of students per classroom that is acceptable to the community before determining total capacity of the school system. Similarly, determining the capacity of the transportation system requires considering the level of service/congestion that will be tolerated by the community.

The jurisdiction must then determine the existing use of those systems, again either by direct measurement or by estimated usage. In estimating current demand, the jurisdiction might determine the average use generated by each residence and equivalent nonresidential operation. It is possible to calculate per unit demand by analyzing historical data, field research, and demographic information such as housing type or number of bedrooms.

The community must also estimate the projected demand for projects that are committed but not completed.

These are projects that are approved but not built, or over which the government has no control because a discretionary approval, such as rezoning, is not required. These are called projects in the pipeline.

The final step is determining available capacity and system adequacy. If, after current and committed demand from the total system capacity is subtracted from capacity remains, then the facility is considered adequate. However, each development proposal must demonstrate that its demand on systems will not exceed that available capacity. Again, this is done by determining per-unit demand, such as average gallons per day of water use or number of vehicle trips per day of traffic generation.

In communities with an adequate public facilities ordinance, a project cannot be approved if its demand will exceed the available capacity. In some cases, however, the developer will be allowed to finance improvements that will increase system capacity, thus allowing the project to meet the adequacy test. Another option available to the land development team may be the implementation of measures that reduce the demand that will be generated by the project. An example of this is the creation of a shuttle system to transport residences to a mass transit station, thereby reducing the number of work trips that a new residential project will generate.

### Development Review Point Systems

A criticism often lodged against zoning is that it is too inflexible. As noted earlier, zoning often does not allow sufficient leeway for the developer to adjust to unique site conditions, changing economic/demographic conditions, or consumer preference. In response, a number of communities have initiated comprehensive development review systems that rate projects according to the success or failure with which they address certain criteria. Fort Collins and Breckenridge, Colorado, are among those jurisdictions that have enacted this type of ordinance.

The criteria to judge development applications in these communities focuses on compatibility, design, and demand on public facilities and environmental impact. The jurisdiction attaches discrete weighting factors to each of the criteria, based on their relative importance to the community. Each project is then given points based on its success in addressing the issues identified. Those that achieve a minimum threshold of points will be approved for development.

### Deed Restrictions and Covenants

Government regulations are not the sole means of controlling land use and development. Private controls imposed by the landowner, developer, or subsequent lot purchasers are also an effective way to establish and enforce desired development patterns. Deed restrictions and covenants are typical private controls. These are agreements between individuals that create self-imposed restrictions on design, use, and alteration. Enforcement is pursued without inter-

vention from the jurisdiction, usually through civil court suit by the parties to the agreement.

The most common examples of restrictions and covenants are the homeowner and condominium association agreements that a developer creates as part of a development project. The agreements often place limits on what purchasers may do with their property as a way of maintaining aesthetic controls. When homeowners take title to their property, they also assume certain obligations to consider the rights of others in the community. The restrictions may limit the size of structural additions and control paint colors, landscaping, and other features to ensure that individual actions are in keeping with the character of the community. These agreements may be more stringent than, and often supersede, government zoning regulations.

During the initial stages of project occupancy, it may be the developer who enforces the restrictions. While project sales are underway, maintaining the appearance and character of the project is essential. After project buildout, however, it falls to the residents of the community, who are parties to the agreement, to enforce the restrictions. Design review committees are appointed by many homeowners associations to control the activities of individual homeowners in a development.

Developers often use similar covenants as part of an overall marketing plan in projects where the developer is not the builder. When portions of the development are sold to a number of different builders, agreements control the actions of builders within a large project. These often address architectural style, amenities, and housing price range. The covenants frequently control builders' activities during construction, as well as the operation of builders' on-site model homes and sales offices.

In a commercial development, restrictions may be imposed on signage, operating hours, or similar activities. Covenants take on importance in a commercial venture because of the potential impact of one business on another. The success of a retail center relies heavily on tenant mix, and operations and covenants are an important means of ensuring that the character of a retail center is maintained. In addition, the developer may have been required to establish them as a condition of the government's rezoning or subdivision approval. Again, these are privately enforced, although the government may play a role if conditions of approval are violated.

Covenants are established in two ways. They are often indicated on the final subdivision plat that is recorded in the jurisdiction's land records. If the covenants are extensive, a plat reference will indicate where in the land records the covenants are located. They form a private contract agreement that applies to all the lots in the subdivision, and any purchaser of the land automatically becomes party to the contract. The covenants have an important role in community design and appearance. Consequently, the land development consultant should assist in drafting them. In order to ensure that the covenants are legally binding and

in conformance with the state's real estate laws, an attorney must review and conduct final drafting of the documents prior to recordation.

A second way of establishing covenants is through the action of individual landowners, who can place restrictions within deeds of conveyance. Groups of landowners can also act together and attach restrictions to their lots, usually for their mutual benefit. These covenants are also recorded in the land records of the jurisdiction. It should be noted that a limited number of jurisdictions in the country have chosen not to adopt zoning ordinances. In the early 1990s, a zoning ordinance in Houston, Texas, was defeated in a referendum, and Houston consequently remains perhaps as the largest American city without a zoning ordinance. In such areas, deed restrictions and covenants regulate land use and development. While the city relies on covenants and deed restrictions to regulate land use, it also exercises covenant enforcement powers.

## SMART GROWTH

### Introduction

Since the end of World War II, land use patterns have encouraged low-density developments where the American Dream of owning a home could be realized. However, a partial result of capturing the American Dream has been an abandonment of our major cities by the middle and upper middle class and the resultant "racial and economic segregation, . . . disinvestment in urban cores, environmental damage, and inaccessible housing, employment, transportation, and services" (National Neighborhood Coalition, 2000, p. 1).

Further, due to a rapid increase in population over the past four decades and the continuing expansion of the suburban rings around major metropolitan areas, communities throughout the nation are debating methods and laws to address suburban sprawl. The issue of how to address suburban sprawl has been on scores of ballots in cities and states across the nation during the past decade. Due to this concern with suburban sprawl, local, state, and federal growth management groups have focused on the concept of "smart growth."

Dependent on one's views and social and economic interests in growth patterns, *smart growth* has a wide range of definitions and interpretations of what is and how it should be implemented. According to Anthony Downs, those debating the essence and implementation of smart growth fall into four basic groups: (1) anti- or slow-growth advocates and environmentalists; (2) pro-growth advocates; (3) inner-city advocates; and (4) better-growth advocates. Although these four groups are diverse in their interests and goals and certainly do not agree on many of the elements of smart growth, "there is always cause for optimism. Even with different goals, the different groups may be able to reach a middle ground, especially if they keep in mind that each

region of the country has unique needs and wide choices" (Downs, 2001).

The smart growth debate has intensified during the late 1990s and into the new millennium. "Last November there were no fewer than 240 anti-sprawl ballot initiatives around the country, most of them passed. Some stripped local authorities of the power to approve new subdivisions without voter assent. Others okayed tax money to buy open land before the developers get it" (Lacayo, 1999).

### Fundamentals and Principles

Smart growth principles, according to the National Association of Home Builders (NAHB), include meeting the nation's housing needs, providing a wide range of housing choices, a comprehensive process for planning growth, and both planning and funding infrastructure costs. Other interest groups place more emphasis on protecting the environment, preserving open space, redeveloping of inner core areas, encouraging new forms of urban design which create and/or restore a sense of community. The range of principles is wide and varied, but the common thread is to provide the housing need for the increasing population while at the same time protecting the environment and enhancing the quality of life for all. "The goal of Smart Growth is not no growth or even slow growth. Rather, the goal is sensible growth that balances our need for jobs and economic development with our desire to save our natural environment" (Glendening, 1997).

The literature on smart growth presents a wide variety of issues, statistics, and legal analysis, and suggests methods of implementation of smart growth principles. Each interest group has its own set of priorities in terms of the issues of most importance and methods by which the principles of smart growth should best be implemented. David O'Neill of the Urban Land Institute (ULI) suggests that although there is no universal definition, there are the following smart growth features with common characteristics (O'Neill, 1999):

- Development is economically viable and preserves open space and natural resources.
- Land use planning is comprehensive, integrated, and regional.
- Public, private, and nonprofit sectors collaborate on growth and development issues to achieve mutually beneficial outcomes.
- Certainty and predictability are inherent to the development process.
- Infrastructure is maintained and enhanced to serve existing and new residents.
- Redevelopment of infill housing, brownfield sites, and obsolete building is actively pursued.
- Urban centers and neighborhoods are integral components of a healthy regional economy.
- Compact suburban development is integrated into commercial areas; new town centers, and/or near existing or planned transportation facilities.

- Development on the urban fringe integrates a mix of land uses, preserves open space, is fiscally responsible, and provides transportation option.

O'Neill goes on to observe that in order to truly understand smart growth, one must overcome a number of myths that have been perpetuated by the various interest groups. The following discussion of eight myths associated with smart growth has been adapted from the ULI booklet.

**Myth #1: Smart Growth Is a Code Word for No Growth.** Growth is inevitable. By the year 2020, the U.S. population is projected to increase by more than 21%, or nearly 58 million people. Increases in population signify an unavoidable demand for residential, commercial, and industrial development.

Growth is not only inevitable; it is also an integral part of the U.S. economy. Real estate capital accounts for about 20% of the nation's total gross domestic product per year. Clearly, real estate development strengthens the economy and enhances community prosperity. This prosperity enables the public and private sectors to invest more broadly in infrastructure, community amenities, education, and open-space preservation. These types of public and private investments, in turn, support the achievement of smart growth objectives.

Smart growth does not seek to stop or limit growth, but rather to accommodate it in a way that enhances the economy, protects the environment, and preserves or improves a community's quality of life. It moves beyond the no-growth versus pro-growth debate to a more enlightened discussion on how best to accommodate growth.

**Myth #2: Smart Growth Is Anti-suburb.** The idea that smart growth is anti-suburb is based primarily on the perception that smart growth is seeking to change current locations of development. According to the Joint Center for Housing Studies at Howard University, between 1990 and 1997, homebuilding activity exceeded 100,000 units in 21 metropolitan areas, with most of that development taking place in medium- and lower-density counties at the metropolitan fringe. The pace of population growth outside metropolitan regions is approaching that in metropolitan regions.

Suburban growth will continue; however, the form and type of future suburban development is less certain. Numerous studies have documented the inefficiencies of providing services and infrastructure to unmanaged growth. According to *Emerging Trends in Real Estate 1999*, standard suburban development may not be able to sustain itself. In fact, this report predicts that many low-density suburban communities will suffer lower land values because of poor planning, increasing traffic, and deteriorating housing stock.

On the one hand, people want to live in the suburbs, but on the other hand, they are fed up with suburban problems. Furthermore, their communities often cannot afford the public services and infrastructure to support this form

of growth. To address these issues, smart growth encourages suburban development that takes place in the context of a local and regional land use strategy. Smart growth seeks to accommodate this growth in a manner that promotes economic vitality, protects environmental resources, and preserves each community's quality of life.

**Myth #3: Smart Growth Creates Another Layer of Government Regulations That Slows the Development Process and Increases Project Costs.** The myth that smart growth adds to the already burdensome regulatory process is inaccurate. Smart growth should not be subject to the regulatory conditions that can slow development and increase its costs. Smart growth, therefore, seeks to reform strict regulatory policies and streamline procedures so that desirable projects are permissible and easier—not harder—to build.

The current predominant patterns of growth and development—low density, separate commercial and residential land uses—can be attributed in part to decades of government policies. Local zoning and subdivision ordinances are a case in point. Over the past seven decades, these policies have separated commercial, industrial, and residential land uses to ensure the integrity of each and to preclude potential nuisances. This philosophy, called single-use zoning, isolates the different activities in which people engage and increases their dependency on the car.

Under these stringent regulatory conditions, it is virtually impossible to develop or redevelop a community that integrates commercial and residential uses, provides walking and bicycling opportunities, and protects environmental resources—all of which are essential principles of smart growth.

Supporters of smart growth are seeking to reform unmanageable codes and ordinances and to streamline procedures that will make it easier to build projects that reflect smart growth characteristics. Communities can streamline procedures by clarifying the language of ordinances, periodically updating and eliminating redundant provisions, and simplifying the permitting process. Smart growth can overcome an essential stumbling block, cumbersome regulatory processes.

**Myth #4: Smart Growth Does Not Sell.** The belief that there is little or no market for smart growth development is a common misconception. To the contrary, smart growth development—at higher densities, in cities, and in better-planned communities—is an option that is attractive to many homebuyers and businesses.

Underlying the demand for the type of development that smart growth encourages is a shift in demographics. Smaller families, empty nesters, childless marriages, and singles all are growing demographic groups seeking housing that reflects their lifestyle and lifecycle. Developers and others are recognizing that these groups favor higher-density housing, which can offer convenience, low-maintenance living, and many amenities.

A combination of factors, including a shift in demographics, low interest rates, and a dynamic economy, has created a renaissance in urban living. The amenities and conveniences provided in traditional cities are attracting a growing number of homebuyers and businesses.

Undoubtedly, homebuyers increasingly want to live in communities that offer amenities and conveniences that foster a better sense of place. On the suburban fringe, well-conceived master-planned new communities that integrate open-space protection and pedestrian links, balance jobs with housing, and provide a mix of land uses meet both consumer demands and achieve smart growth objectives.

The market for smaller-lot, higher-density, mixed-use suburban development also appears to be growing. Concerns about increasing traffic congestion, growth in telecommuting, and demographic trends should ensure that high-density, mixed-use development will continue to emerge as an important component of the suburban housing market.

The market for the type of development that reflects smart growth principles continues to expand.

**Myth #5: Smart Growth Equals Growth Boundaries.**

Despite claims to the contrary, smart growth does not simply prescribe urban growth boundaries (UGBs) as the answer to growth pressure. When it comes to smart growth, one size does not fit all. There is a wide range of incentive policy and programmatic choices from which to choose.

While it may be appealing, both for its perceived simplicity and for initial political acceptance, to draw a line around a developing community, urban growth boundaries present complex technical, political, and legal issues and often cause controversies.

Urban growth boundaries and urban service areas should be considered as one element of a smart growth strategy only if they are consistent with the political, economic, and cultural environment of a region and are comprehensively and thoughtfully applied.

Smart growth does not endorse one policy, but rather challenges communities and regions to select those that reflect their unique conditions and help them achieve their region-wide vision for future growth and development.

**Myth #6: Smart Growth Will Eliminate the Need for New Roads.**

Between 1983 and 1990, it was estimated that more people with more per capita travel accounted for about one-third of the increase in driving. Growing dependence on the automobile because of less car pooling and transit use accounted for another third of the increase in driving time, while growing travel distances, due to the increased separation of homes, jobs, recreational facilities, and other places, accounted for the final third.

An effective smart growth program primarily will address the latter two factors. Better-planned, mixed-use communities will reduce travel distances, helping to limit the growth in trip lengths. Concentrating growth in established areas also will reduce the need for long trips and will offer greater choices in travel modes, especially walking and tran-

sit. Positive trends also exist for the first factor, disproportionate increases in per capita trip making.

The implication is that smart growth can help bring the growth in work commute time more in line with the underlying population growth, a trend confirmed by shifting demographics and the aim of smart growth policies and practices to improve the jobs/housing balance. That could reduce substantially the investment needed in new highways, although it certainly would not eliminate it.

Smart growth will not eliminate the need for new roads and road improvements, but it does encourage better-planned, mixed-use communities that can provide an array of transportation alternatives.

**Myth #7: Smart Growth is Bad for Business.** The claim that smart growth is bad for business overlooks the fact that a growing number of businesses are taking leadership roles in advancing smart growth programs and/or benefiting from smart growth policies. The business community also recognizes the emerging markets that complement the principles of smart growth, namely those in urban communities, along abandoned waterfronts, and near public transit.

Today's business community is becoming more aware of the impacts of unplanned development on its ability to remain competitive. In Atlanta, for example, when corporations were asked what was the most serious impediment to business in the metropolitan area, the overwhelming response was "traffic congestion."

While many businesses have made a clear commitment to mitigate the impacts of unplanned development on their bottom line, others are recognizing that they can enhance their businesses because of smart growth programs. Brownfields (former industrial or commercial sites that are perceived to be or are environmentally contaminated) are a case in point. Because of the cost and the risk involved, few brownfields have been redeveloped. Due to federal and state legislation, however, certain brownfield developments now are attracting investors.

Commitment to smart growth by the business community is growing. Businesses have taken an interest and, in certain cases, a leadership role in improving the quality of life within communities through smart growth initiatives. Several businesses and business organizations have formed partnerships and made investments to advance smart growth.

**Myth #8: With So Much Undeveloped Land, There Is Limited Value in Open Space Preservation.**

Smart growth respects both the rights of private property owners and the value of open spaces and natural resource lands. Open spaces are important components of communities. They provide recreational opportunities, habitat for wildlife, buffers for pollution, and places for people to gather and enhance their sense of place. Smart growth seeks to preserve these open spaces in the context of achieving a vision for a community's desired growth and development.

Underscoring the value the public places on open-space protection, voters approved 72% of the 240 state and

location conservation ballot measures in 1998. These measures supported a wide range of conservation and community enhancement activities throughout the country. These results demonstrate that open-space preservation is an essential community value and therefore a critical element of any smart growth initiative.

While environmental protection objectives traditionally have been viewed as conflicting with economic goals, smart growth seeks to identify those areas where both environmental and economic objectives can be achieved simultaneously. In many respects, as smart growth demonstrates, open-space protection complements economic development objectives.

**Representative Projects**

Now that we have discussed some of the fundamental principals and myths attributed to smart growth, it is interesting to examine some projects that have implemented various elements of smart growth. The representative projects discussed below reinforce the notion that there are a variety of methods of implementation of smart growth policies. The implementation of smart growth policies is as varied as the social, economic, and political environment of a given jurisdiction. There is no one-shoe-fits-all solution, nor do all or the majority of smart growth principles need to be implemented to be successful in achieving the goals of smart growth.

There are countless opportunities and challenges inherent in the implementation of smart growth. Change is never easy, and is certainly even more difficult, when it involves perceived or real issues of property rights and fundamental changes in how future development is to take place. The implementation of smart growth policies influences the way we do business at all levels of government, and the results will not be seen for years to come. However, strides are

being and have been made, and the following are some notable examples.

**Maryland.** The state of Maryland is one of the nationally recognized leaders in the implementation of smart growth policies and strategies statewide. “Instead of growth boundaries, the state directs its funding, including that for roads, sewer, schools, and other public investments, to designated growth areas. These are designated for growth because they either have or are located near existing infrastructure. As a result of this policy, hundreds of millions of state dollars will be channeled into these designated areas to accommodate future growth and development. Development still can occur outside designated areas, but state funds will not be used to support such projects” (Maryland Office of Planning, 1997).

According to Governor Glendening of Maryland, there are six reasons why the Maryland Smart Growth Program has been successful (Glendening, 1998):

1. The smart growth package was balanced and addressed both urban and rural concerns, which had the practical political benefit of getting support from both the urban and rural legislators.
2. Mayors of 157 municipalities were supportive as they saw opportunity to get state recourse to improve these cities and towns.
3. The program preserved counties’ decision-making authority, but the state would not pay for sprawl resulting from bad land use decisions.
4. Environmental groups supported the program because it comprehensively addresses land use.
5. Rural groups supported the program because it protects the rural settings, lifestyles, and rural economics.

**TABLE 2.2 Land Usage in the United States; Contiguous 48 States Only<sup>a</sup>**

	1945	1954	1964	1974	1982	1992	1997
Total cropland <sup>b</sup>	451	465	444	465	469	460	—
Cropland used for crops	363	381	335	361	383	337	353
Grassland pasture and range	659	632	636	595	594	589	—
Forest land	602	615	612	5987	567	559	—
Urban land	15	19	29	35	50	58	—
Recreation/wildlife areas	23	28	50	57	71	88	—
Miscellaneous farmland areas <sup>c</sup>	15	12	10	8	8	6	—

<sup>a</sup>Data in millions of acres.

<sup>b</sup>Includes cropland used for crops, cropland idled, and cropland used for pasture.

<sup>c</sup>Includes farmsteads, farm roads, etc.

Note: 1997 data are only available for “Cropland used for crops.” Rest of data will be available in late 1999.

Source: U.S. Department of Agriculture, Economic Research Service, Natural Resources and Environment Division, Agriculture Handbook, No. 712.

6. The smart growth approach gave businesses certainty about where the state will support growth and where it will not.

**Utah.** The state of Utah established Envision Utah, a partnership of government, business, civic and environmental leaders, in an attempt to address its growing population and corresponding issues associated with significant growth. “Since 1997, the Envision Utah partnership has prepared growth and development projections and various growth scenarios that demonstrate how the state might accommodate future growth” (Envision Utah).

“The state of Utah passed the Quality Growth Act, which created a commission to review growth strategies and make recommendations to the legislature in 2000. The act also created a critical land conservation fund, with a \$6 million cap, to preserve agricultural and open space land” (Smart Growth Network). The Envision Utah program seeks wide public input, which hopefully will provide a future growth scenario resulting in a balanced economy, environment and quality of life for many years to come.

**Orenco Station, Portland, Oregon.** The 190-acre new community in the western suburbs of Portland, Oregon, is an example of smart growth in the suburbs.

Orenco Station is a pedestrian-oriented, mixed-use community containing housing, retail establishments, and office space. Homes are selling for 20 to 30 percent higher than the cost of the area average. The compact design includes a wide range of housing types, from single family detached houses to accessory units over garages to live/work lofts and townhouses over retail shops. The common thread of the community is a formal system of open spaces and miniparks terminating in the recently opened Orenco Station stop of the Tri-Met Max light rail line, which connects downtown Portland to its suburbs. The project’s architects and developer conscientiously scripted a pedestrian-friendly environment to encourage walking and a more community-oriented lifestyle (“Orenco Station,” 1999).

**Prairie Crossing, Grayslake, Illinois.** Prairie Crossing is located approximately 40 miles north of Chicago, Illinois. It consists of a planned clustered community of 317 single-family detached units on 667 acres, which results in a density of 0.47 dwelling units per acre. Prairie Crossing is an example of how a well-designed community, with emphasis on the preservation of open space, can be an economic success. “Once completed, the 667 acre project will contain 317 single family homes on lots ranging in size from 6,000 to 20,000 square feet. The project preserves more than 450 acres or 70 percent of the land as open space” (Arendt, 1999).

The open-space system includes more than 10 miles of trails, encouraging its residents to bike or walk throughout the community. The trail system also connects to a commuter rail system, which gives many residents of Prairie Crossing the opportunity to get to the workplace by a transit system. Additionally, a large portion of the development’s open-space abuts a 2500-acre natural protected reserve,

which adds to the functionality of the ecosystem. “Home sales and premiums indicate that Prairie Crossing’s innovative design and amenities are creating value in the market place. According to Prairie Crossing marketing overview, homes are selling for \$139 per square foot, which is 33 percent higher than comparable homes in the 35 competitive market areas (CMAs). Furthermore, 35 homes are being sold each year (at the high end of the entry-level sales pace in CMA), at an average of \$350,000” (Linberger, 1999).

In support of the smart growth principle that the meaningful preservation of open space areas can be economically beneficial, Prairie Crossing is a notable example.

## REFERENCES

- Anne Arundel County Office of Planning and Zoning, 1978. *General Development Plan*, Anne Arundel County, Annapolis, Md.
- Arendt, Randall. 1999. *Growing Greener: Putting Conservation into Local Plans and Ordinances*, Island Press, Washington, D.C.
- Downs, Anthony. 2001. “What Does ‘Smart Growth’ Really Mean?” *Planning*, April.
- Envision Utah. The Envision Utah Process, website <http://www.environmental.org/main/.html>.
- Glendening, Parris, Governor of Maryland. 1977. Remarks at Partners for Growth Conference, December.
- Glendening, Parris, Governor of Maryland. 1998. Remarks at ECOS/AASHTO Conference on Smart Growth, December 1.
- Linberger, Christopher B., 1999. *Marketing of Current Product Program of Prairie Crossing, Grayslake, Illinois*, Robert Charles Lesser & Company, Santa Fe, N.M.
- Lycayo, Richard. 1999. “The Brawl over Sprawl,” *Time Magazine*, March 22.
- Maryland Office of Planning. 1997. *Smart Growth and Neighborhood Conservation: Preserving What’s Best about Maryland*, Office of Planning, Baltimore, Md.
- National Neighborhood Coalition (NNC). 2000. *Connecting Neighborhood and Region for Smarter Growth*, NNC, Washington, D.C., April.
- O’Neill, David. 1999. *Smart Growth: Myth and Fact*, Urban Land Institute (ULI), Washington, D.C.
- “Orenco Station,” 1999. ULI Project Reference File, vol. 29, no. 1.
- Smart Growth Network. Governors for Smart Growth, website <http://smartgrowth.org/indexframeset.html>.

## ADDITIONAL READING

- Bauman, Gus, and William H. Ethier, “Development Exactions and Impact Fees: A Survey of American Practices,” *Law and Contemporary Problems*, volume 50, no. 1, Winter 1987, pp. 51–68.
- Fairfax County Office of Comprehensive Planning. 1990. *Comprehensive Policy Plan*, Fairfax County, Fairfax, Va.
- Finn, Robert F., *An Introduction to Zoning, Site Plan and Subdivision Law in Virginia*, Northern Virginia Builders Association, Vienna, Va., 1985.
- Fischel, William A., *The Economics of Zoning Laws*, Johns Hopkins University Press, Baltimore, Md., 1985.



- Frank, James E., and Robert M. Rhodes, *Development Exactions*, Planners Press, American Planning Association, Chicago, 1987.
- Goodman, William I., ed., *Principles and Practice of Urban Planning*, International City Managers' Association, Washington, D.C., 1968.
- Mandelker, Daniel R., and Roger A. Cunningham, *Planning And Control of Land Development*, 2d ed., Michie, Charlottesville, Va., 1985.
- Maryland-National Capital Park and Planning Commission, *Carrying Capacity and Adequate Public Facilities*, Maryland-National Capital Park and Planning Commission, Hyattsville, Md., 1977.
- Meck, Stuart, and Edith M. Netter, eds., *A Planner's Guide to Land Use Law*, Planners Press, American Planning Association, Chicago, 1983.
- Munkacy, Kenn, and Tom Sargent, "Strategies for Dealing with Development Exactions," *The Real Estate Finance Journal*, Summer 1987, pp. 6–12.
- National Association of Home Builders (NAHB). "Cluster," *Builder*, August 7, 1978, pp. 24–38.
- National Association of Home Builders (NAHB). *Cost Effective Site Planning*, NAHB, Washington, D.C., 1978.
- National Association of Home Builders (NAHB). *Land Development Manual*, NAHB, Washington, D.C., 1974, 1981.
- National Association of Home Builders (NAHB). *Subdivision Regulation Handbook*, NAHB, Washington, D.C., 1978.
- Porter, Douglas R., "Flexible Zoning," *Urban Land*, April 1988, pp. 6–11.
- So, Frank S., ed., *The Practice of Local Government Planning*, International City Management Association. Washington, D.C., 1979.
- Urban Land Institute (ULI), *Residential Development Handbook*, ULI, Washington, D.C., 1978.
- Wright, Robert R., and Susan W. Wright, *Land Use in a Nutshell*, West, St. Paul, Minn., 1985.

## CHAPTER 3

# SITE PLAN ORDINANCES, SUBDIVISION REGULATIONS, AND BUILDING CODES

*Lawrence A. McDermott*

### INTRODUCTION

With few exceptions, the development of land normally involves the subdivision of land, a site plan, and/or a building permit. With respect to the subdivision of land, most ordinances define subdivision as the division of land into two or more parcels, whether for immediate or future use. While this division usually implies transfer of ownership of the divided portions, most jurisdictions consider the definition to include condominium and cooperative ownership. Divisions of land and buildings for long-term leasehold also are frequently included in the definition. The local governing body imposes standards and procedures for the subdivision of land with the adoption of subdivision regulations. Like the zoning ordinance, subdivision regulations are an exercise of government's police powers. They protect the health, safety, and welfare of its citizens. In some jurisdictions, the conveyance of an unsubdivided portion of a master tract is a criminal violation. Therefore, pre-subdivision conveyances, leases, or establishment of condominium or cooperative regimes should be avoided.

As mentioned earlier, a use permitted by right in the zoning ordinance requires no specific approval under its provisions, short of a building permit. The latter can only address construction requirements for a single building on a single lot or parcel. Subdivision regulations are an important mechanism to ensure that a development complies with the requirements of the zoning ordinance. In addition, the regulations provide protection to users by ensuring that subdivided lots are suitable for their intended purpose.

Similarly, the jurisdiction's governing body retains a vested interest in knowing that its residents and business operators are provided with durable and efficient develop-

ments. It must also have assurances that land is used safely and efficiently. Land is an exhaustible resource, and its development is a primary generator of revenues needed to provide public services and maintain fiscal health. Government, therefore, assumes an important role in protecting that resource and demands that development projects remain marketable and a continued community asset.

As also discussed in this chapter, site plans and building codes play an important role in ensuring proper review of development plans and the safety and structural integrity of proposed buildings. In many jurisdictions, site plans are synonymous with subdivision plans and require only administrative review. In contrast, some jurisdictions require site plans to go under full scrutiny of not only an administrative review staff, but also the governing body, with public hearings as part of a very formal approval process.

The review processes for building codes and building plans have been established nationwide to ensure the health, safety, and welfare of future occupants of a structure. There are a number of nationwide building codes that are routinely used as the foundation for local jurisdictions to augment, in order to meet and reflect local concerns and issues.

The underlying purposes of land use regulations discussed below are to protect the large community from negative impacts of a forthcoming development and to ensure that time-tested standards are honored in the construction of new structures.

### SUBDIVISION REGULATIONS

#### Components of Subdivision Regulations

Subdivision regulations perform several specific functions. Of paramount importance is the adoption of design, con-

struction, and material standards for facilities that serve and support the development and the larger community. Although jurisdictions vary on the listing of systems addressed by their subdivision regulations, the most typical features, systems, and facilities are listed below.

- Street design, including arrangement, right-of-way and pavement requirements, geometric design, and intersection design
- Sidewalk and trail design
- Street and traffic control signage
- Parking standards and geometrics
- Storm drainage and stormwater management system design, including inlet, pipe, manhole, swale, pond, culvert, and outfall
- Water distribution system, including trunklines, mains, and laterals
- Sewerage collection, including trunklines, mains, laterals, and pumping stations
- On-site and small off-site sewage treatment design
- Utility system and easement design
- Erosion and sediment control design
- Lot and open-space grading
- Installation of survey monuments and markers
- Protection of environmental features

The subdivision requirements attempt to offer protection to the consumer, who has a perception of the quality, design, and functionality of a project that has been or is planned to be developed whether it be commercial, industrial, or residential. It provides assurance that systems will be adequate and working and that the investment is sound. Standards provide for safe and proper design of transportation facilities and other critical public facilities, such as adequate and safe water supply and sewer capacity.

The jurisdiction desires protection because it will own, operate, and maintain many of the facilities provided by the developer. By setting minimum standards, the jurisdiction can better predict lifecycle maintenance costs and avoid costly reconstruction of otherwise less durable facilities. Maintenance and repair will be made easier because the regulations provide for a record of the design of roads and drainage facilities. Final construction documents record the location of underground utilities. In addition, the regulations help achieve continuity and compatibility of systems. Since a residential or commercial subdivision is part of a larger neighborhood and community, the infrastructure and public facilities must coincide efficiently with the entire pattern. Not only must new development connect with the existing community, it must anticipate, and in some cases

serve, future development. R. Anderson noted that “Subdivision controls are imposed on the supportable premise that a new subdivision is not an island, but an integral part of the whole community which must mesh efficiently with the municipal pattern of streets, sewers, water lines, and other installations which provide essential services and vehicular access” (Mandelker, 1985, p. 508).

Public infrastructure design criteria and construction standards ensure that the public facilities and transportation network are compatible from one project to the next. Provisions in the subdivision regulations address nearly every component of those systems to ensure that individual projects do not disrupt the community. For example, improperly designed and located driveway entrances and intersections can create hazardous situations that may severely impair the efficiency of the transportation network. Similarly, stormwater management systems can cause flooding and damage on both upstream and downstream properties if those properties are not considered in the design. Literally every component of a development project has an effect on neighboring properties and the surrounding community. During the construction of one project, uncoordinated development on adjoining properties can have severe consequences.

### **Development Management**

Subdivision regulations fulfill an important management function for local government through the review of development proposals in an organized, efficient, and uniform fashion. The regulations set minimum standards for preparation and submission of design and construction documents and set clear administrative procedures for their review. This makes it possible to budget manpower in both number and level of training as well as facilities. The land developer benefits by procedures that discourage unfair or discriminatory treatment. Review personnel can operate with greater efficiency since they can expect similarity in document format and completeness in the information provided. Even requirements as mundane as sheet size and number of copies for applications can be included that facilitate storage, handling, and distribution to those who must review applications. The land development consultant can operate more efficiently if the jurisdiction sets clear procedures and requirements for plan preparation and review.

### **Guaranteed Performance**

Subdivision regulations usually include provisions requiring the execution of performance guarantees, such as bonds, letters of credit, cash escrow accounts, or similar financial instruments, to underwrite the installation of public infrastructure. Most jurisdictions allow residents and business to occupy buildings before all of the land development improvements are completed. In larger projects, this practice allows the developer to balance income and expenditures throughout the construction period. This also helps reduce damage by construction equipment to finished improve-

ments, such as curbs, storm drain inlets, and street pavement. Performance bonds are often not released until after a project is completed, occupied, or accepted by public authorities. This ensures that infrastructure will be finished, maintained and, if damaged, repaired by the developer. This protects both consumers and the jurisdiction that could otherwise incur these expenses.

The bonds guarantee construction of public improvements in accordance with approved plans. The financial institution that issued the instrument makes funds available to the local government if the developer is forced to abandon the project. The jurisdiction is then able to construct the improvements if the developer is unable to complete them. Institutions that issue performance bonds usually examine the creditworthiness and past performance record of the developer and typically charge substantial fees for their service and risk. The bond instruments have the effect of committing the capital and credit available to the developer for future ventures. This provides strong motivation to complete a project and fulfill the obligations of the bond so that it can be released.

### **Durability of Information**

Subdivision regulations enhance the clarity and durability of information about property ownership and transfer by setting standards for property description, control of field surveys, property boundaries, easements, and other information related to property ownership. Filing final subdivision plats and related information in permanent land records of the jurisdiction simplifies the conveyance of ownership for lots and parcels. In addition, the information contained on the plats serves as public notice to future occupants about conditions and requirements affecting their occupancy.

### **Benefits to the Land Development Design Team**

Despite the costs that the regulations impose on the land developer, subdivision regulations offer protection to land developers. Land development is an extremely competitive industry in which many firms and individuals operate. Subdivision regulations establish a uniform standard of design and construction to which all developers must adhere. Competing firms that provide an otherwise similar product cannot undercut prices by using a lesser standard of infrastructure. In addition, the regulations offer a predictable framework for allocating development costs.

Most subdivision ordinances also set maximum time limits within which the local government must act to approve or disapprove the submittal. Where properly applied, these ordinances can guard against costly project delays and affirm that plans will be reviewed in a timely and predictable manner. Such provisions also ensure that jurisdictions maintain adequate manpower levels based on projected workload volumes. Mandatory time limits also prevent unfair treatment of developers, since reviewing authorities

cannot set aside a plan viewed to be unfavorable for an indefinite period as a tactic to discourage or delay a project.

For the land development consultant, the regulations define essential parameters that help to guide or, to some, constrain the project site design. In some regards, they eliminate the need to perform original research and analysis, which expedites project design and budgeting. Information concerning plan processing also guides in the preparation of project work programs and budgets.

### **Related Development Regulations**

Throughout the 1970s and '80s, the country experienced a heightened concern with respect to the effect of land use on the environment. Attention not only focused on the flooding caused by the grading and pavement required to develop land, but was accelerated by past regulatory practices. Early storm drainage requirements led to the design of systems intended to collect and carry site runoff quickly, without regard to downstream property. Recognition of the effects of this type of system led to the adoption of stormwater management ordinances requiring on-site collection, trapping, and slow release of storm runoff. Stormwater management ponds have become a common feature and are often mandated in land development. Even these early management regulations were shown to consider impact on natural systems improperly, leading to further refinement in some communities.

Concern about sediment damage from all land-disturbing activities, agriculture as well as urban development, led to the adoption of grading and erosion and sediment control regulations. Often made a part of the subdivision regulations, or adopted as an independent ordinance, these provisions require greater care in both design and construction of site facilities.

More recently, local, state, and federal legislators have focused on the impacts of land use on ground and surface water quality. Water quality was found to be degraded not only by sediment runoff, but by the many chemical and metal byproducts of both agricultural and urban activities. Coupled with greater interest in the use of natural features, such as wetlands to filter runoff, greater investigation and mitigation of environmental impact is now required by many communities.

These issues have led to an evolution in site planning practices. They have been altered to the point where the land development consultant must consider the size and placement of permanent facilities during the design process. In addition, sensitive environmental features now play a more central role in project layout. Where they must be preserved, either by regulation or by good design, they are a key factor in decisions about project feasibility. Rather than viewing them as constraints, the land development consultant should view features and facilities as an opportunity to enhance project appeal and marketability.

In some regions, the link between land use and the environment has been made stronger where the latter's deg-

radation has been shown to threaten a significant economic resource. Comprehensive programs to protect these features seek to balance private property development rights with broad public and economic concerns. Two examples of this are in Texas, where protection of groundwater recharge areas is a high priority, and in the Chesapeake Bay region. In the latter case, administrative subdivision reviews are being supplemented, and in some cases supplanted, with more extensive discretionary reviews. The goal is to ensure that land-disturbing activities, such as clearing and grading, are kept to absolute minimums. The land development consultant must design protective devices with greater efficiency to prevent pollutant and sediment damage. In some cases, public policy shifts towards allowing greater flexibility in project design have failed to occur. This places greater challenge on the land development consultant to recommend design solutions that achieve these public objectives in a cost- and time-effective way.

### What to Look for and How to Use Subdivision Regulations

- As with other regulations, obtain the most current version of the ordinance. A facilities design manual usually supplements the subdivision ordinance and provides policies, standards, and design details for the various methods and requirements for construction within the jurisdiction. The manual may be an independent/separate document or may be a section contained in the ordinance. The community amends these from time to time by either administrative or legislative action with the latter being preferable. It offers an opportunity for dialogue with the governing body on the need for and cost-effectiveness of new standards and procedures.
- Identify and compile other ordinances or subordinate documents referenced in the subdivision regulations that control the physical development of land. These include stormwater management, grading, and erosion and sediment control ordinances.
- Note the effective dates of ordinances and amendments; they may have different applications or may exempt preexisting conditions. Policies concerning existing recorded plats for otherwise undeveloped subdivisions vary. These plats sometimes predate even the earliest version of regulations, or more restrictive ordinances that might now be in effect. Whether or not a project can proceed and what required public improvements apply varies depending on the jurisdiction. In some jurisdictions, plat approvals have an expiration date if no development takes place. "Grandfathering," or exemption of projects from later ordinance changes, is discussed in Chapter 11.
- Some jurisdictions provide exemptions for certain types of developments. Among those often excluded

are so-called minor subdivisions that sever one or two parcels from a larger tract. Ordinance or local policy may define a tract using the parcel boundaries as they existed as of a certain date. Family conveyance provisions may allow for a certain number of gift lots to be given to immediate family members. The ordinance may exempt large-lot subdivisions and permit estate or agricultural use of these properties. Boundary adjustments between adjoining properties and divisions caused by government condemnation are also frequently excluded from full compliance with subdivision regulations. Simple plat preparation and recordation procedures often apply to these exempted divisions of property. Minimum standards to protect public health and safety may also apply.

- Identify sections of the ordinance that address site planning and design criteria.
- Identify provisions addressing system construction standards.
- Identify ordinance provisions regarding application requirements and preparation of submittal documents. These contain specifications concerning drawing size and scales, required information, number of copies, and distribution. The land developer may be required to post signs and mail notices to adjoining property owners and other interested parties.
- Procedures for administrative waivers of certain requirements, appeals, and judicial challenges to administrative or legislative actions by agency personnel or the governing body are described in the ordinance.
- The ordinance or related documents describe fees required from the land developer for review of applications.
- Identify provisions for execution of performance bonds and bond release procedures. These include preparation of as-built drawings.

### SITE PLAN ORDINANCES

The phrase "site plan" has several meanings. In its generic sense, a site plan is a graphic representation of the layout of buildings and facilities on a parcel of ground, with its detail, accuracy, and scale dependent upon its purpose. Although "site plan" is sometimes used synonymously with "development plan," many jurisdictions attach a more discrete meaning to "site plan." The differences are based largely on the specific language of the enabling legislation given to local jurisdictions by their state legislatures. The land development consultant must become familiar with the local definition for "site plan" and the implications it has for the client's development project. Many ordinances refer to a site plan simply as one of the items of information included in an application submitted for review. Its form and purpose are usually prescribed in the various ordi-

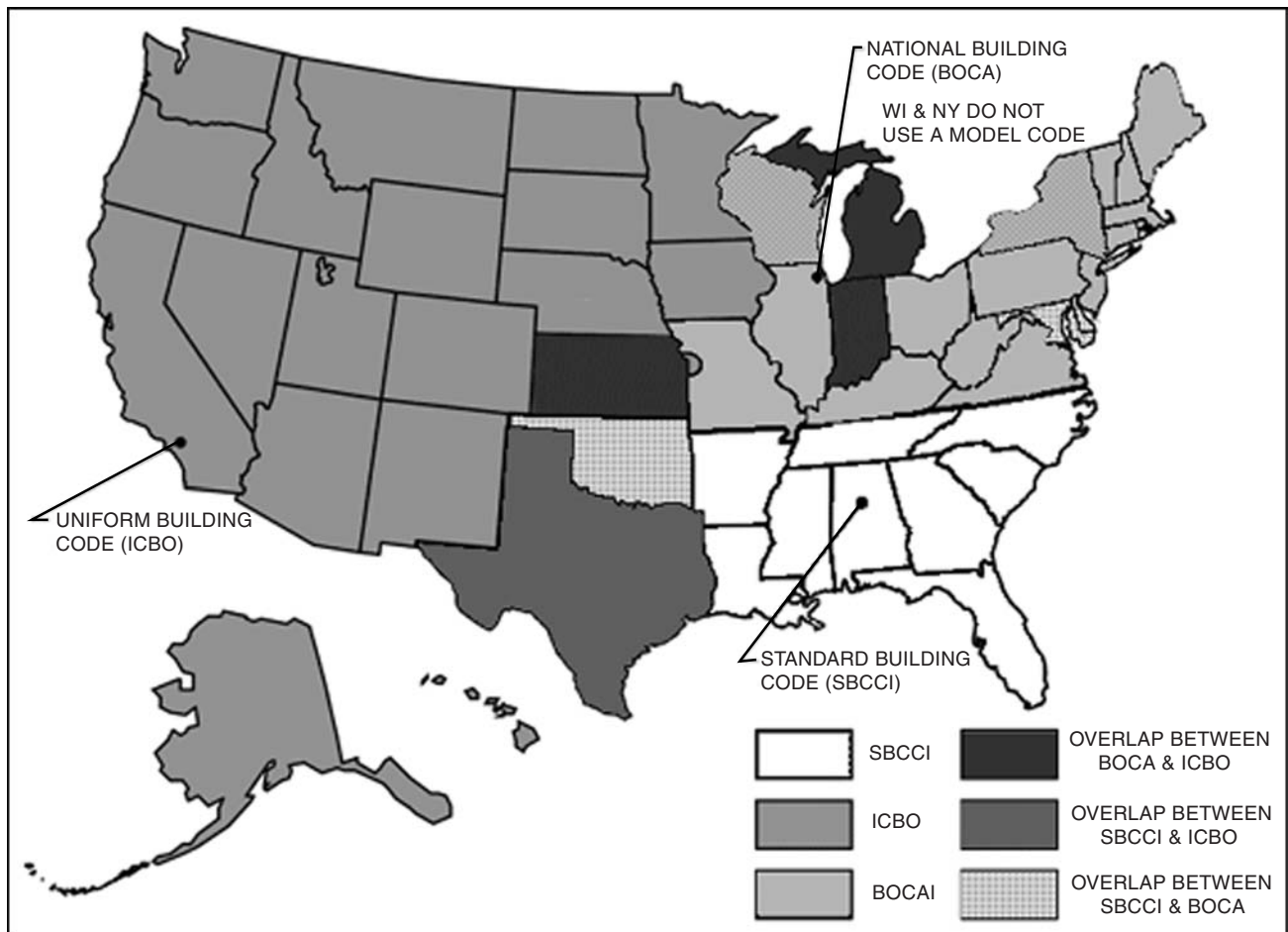
nances administered in the community, and the level of detail required may vary among ordinances even within a single locality.

In some jurisdictions, a site plan is a separate administrative instrument apart from other development plans. Its purpose is to ensure that construction projects are accorded the same level of review and are subject to the same standards of performance as subdivision plans. In communities without site plan review authority, a development that does not fall under the definition of subdivision need only show compliance with the building code and zoning ordinance. This would apply to many apartment projects and most free-standing office, commercial, or industrial structures. Absent controlling regulations, on-site private facilities are designed at the discretion of the land development team, with only a cursory review during the building permit stage. When subject to the site plan process, however, these developments must fully comply with both zoning and infrastructure standards, just as if they were actual subdivisions.

In some cases, developments are subject to both subdivision and site plan process, which may result in repetitive reviews. In such cases, the subdivision processing is aimed at ensuring compliance with the zoning ordinance, producing durable records of ownership and proper consideration of fees and exactions. The site plan process provides control of on- and off-site infrastructure, access, and other features to ensure compatibility and connection with the larger environment.

Some jurisdictions refer to a site plan as having a special role in securing development approval. It is neither a technical subdivision plan-type document given an administrative staff review nor an elaborate rezoning application given extensive public hearing by the governing body. It is somewhere in between, and is usually applied in circumstances demanding greater control to guard against incompatibility.

In Montgomery County, Maryland, for example, the site plan is used when the zoning ordinance specifies some discretionary authority in the physical arrangement of a pro-



**FIGURE 3.1** U.S. map indicating model code organizations area of influence. (Courtesy of Southern Building Code Congress International, Inc.)

ject. One of the county's ordinances, for instance, calls for the mandatory provision of affordable housing in projects with more than 50 dwelling units with an automatic density bonus. Normally, a site with residential zoning would only require subdivision and construction plans, approved administratively. However, the increased density triggers the requirement for site plan review. This gives the professional staff and the county's Planning Board greater leeway to consider a project's internal physical arrangement and relationship to its neighbors. The ordinance describes the information that must be shown, and the parameters within which it must be reviewed.

The Montgomery County, Maryland, process contrasts with Arlington County, Virginia, where both the Planning Commission and County Board review the site plans and public hearings are held as part of the approval process. Although the county is limited in the off-site improvements that it can require from the developer, site plans are given much the same public scrutiny as a rezoning application.

In this manual, unless specific reference is made to a site plan ordinance, use of the phrase "site plan" refers to the common, generic definition, that is, the design of a project layout that is the end product of the planning process.

## **BUILDING CODES**

Building codes are adopted to protect the lives of building occupants and to guarantee safe, habitable, durable structures. While most jurisdictions adopt and modify one of the national model codes published by long-established building code organizations, some develop extensive amendments to reflect local concerns and experiences. Figure 3.1 illustrates the different code organizations and the regions where their model codes are prevalent.

With a few important exceptions, the building codes play little role in land development planning. Their primary

focus is on structural integrity, fire prevention, control and safety, suitability of materials, and support system operations. However, building codes do affect site design in three ways. First, to control the spread of fire, they place limitations on the proximity of buildings to each other and to adjoining property lines. These restrictions become important considerations in the design of clustered single-family detached homes, townhouses, and multiple building complexes, such as garden apartments. Secondly, to improve building occupants' chances of surviving a fire, building codes specify maximum building height, the distance a person must travel to exit a building, and the number of building or unit exits. Both height and distance are often expressed as the number of stories above grade. These may vary, depending on type of construction and presence of automatic fire suppression systems. However, the land development consultant must determine final grade elevations during the planning and engineering phases of a project to ensure these regulations can be satisfied. In addition, the location of building entrances affects placement of walkways, parking, lighting, and landscaping. Lastly, new federal regulations concerning access for persons with physical handicaps will play a role in site grading and development. These regulations are incorporated into building codes, which specify that ground floor units of all apartment buildings must be accessible to people in wheelchairs. Codes further specify maximum grades for walkways to assist the disabled and handicapped. Provisions in the building codes also address the placement of handicap parking in residential, retail, and employment facilities.

## **REFERENCES**

Mandelker, Daniel R., and Roger A. Cunningham. 1985. *Planning and Control of Land Development*, 2d ed., Michie, Charlottesville, Va.

## CHAPTER 4

# EXACTIONS, INFRASTRUCTURE ENHANCEMENTS, AND FEES

*Lawrence A. McDermott*

### INTRODUCTION

In today's world of increasing population, containing suburban sprawl, higher expectations from citizen activists, glaring environmental and transportation concerns, increased demands are placed on public infrastructure and services. In recent decades, a significant portion of the responsibility for the improvement of public infrastructure has been transferred, in a large manner, from federal, state, and local governments to the land developer and other entities within the private sector.

Given what can be a considerable expense and impact on a given land development proposal, the land development team must evaluate closely the laws, ordinances, and standards of the locality in which it is proposing a development, to determine what exactions, fees, and infrastructure enhancement can be expected. The type, form, and number of such requests or mandates for public improvements and facilities are numerous and varied in their nature. Depending on the size and impact of a development proposal, they can range from simple road frontage improvements to dedication of land for school or park sites. Nevertheless, whatever dedications, reservations, fees, or other monetary retributions are required, it is important that they be identified early in the process so that design and *pro forma* economical analysis can be undertaken to account for these exactions. This chapter highlights the most common methods by which infrastructure improvements, fees, and other exactions become part of the development process.

### EXACTIONS, INFRASTRUCTURE ENHANCEMENT AND FEES

As communities experience population and economic growth, ever-increasing demands are placed on existing

public infrastructure, facilities, and services. These public facilities range from subdivision streets to major commuter highways, from tot lots to regional parks, from water lines to fire stations, and from sewer lines to treatment plants. As these systems reach their capacities, their ability to function efficiently or accommodate new growth is diminished. The consequences range from a government leader's frustration to dissatisfied constituency or hazardous, even life-threatening, conditions created by inadequate facilities.

Historically, it has been the responsibility of government to provide and improve many of these facilities. Reductions in federal funding have placed considerable burden on local government with corresponding budget impacts. Reaction to government spending by community residents, resulting in rethinking by political leaders, has led government to find alternative and sometimes innovative ways to pay for growth. Frequently, these efforts become the subject of lengthy court challenges as important public goals and policies begin to infringe upon strong private property rights. While substantial public obligation remains, the private sector has been given an increased share of the burden. Development regulations play a critical role in allocating costs among private developers, a community's taxpayers, and prospective consumers, who may ultimately bear the cost of growth.

This allocation formula varies widely among jurisdictions. Significant differences exist in the form of the applicable regulations, the timing of contributions, and the magnitude of financial commitments that must be made. In most cases, the subdivision regulations identify much of the developer's obligation, which are most typically limited to on-site improvements. In some instances, requirements are determined during the project approval negotiation process,



usually at the rezoning phase. Some jurisdictions have adopted separate ordinances that relate development projects to existing community public facilities. These tie improvements to existing capacities and deficiencies of systems beyond the boundaries of the development.

*Exaction* is the term applied to the provision by the land developer of something that establishes public, communal, and even consumer benefit in exchange for and as a condition of development authorization. More specifically, exaction includes the following activities (adapted from Frank and Rhodes, 1987, p. 3):

- Reservation, preservation, or dedication of sites for public or common facilities and services
- Construction, dedication, or operation of facilities and services for public or common use
- Purchase of land, facilities, goods, and services for public or common use
- Payment of monies to the governing body to defray the cost of its purchase of land, facilities, equipment, and services
- Restriction of development potential or provision of features and facilities intended to achieve social, aesthetic, or economic goals or other public benefit

The exaction may provide either direct benefit to the occupants of the proposed development, or indirect benefit by addressing an important public goal or mitigating an undesired impact created by the project. The item provided may have either tangible cost or intangible value. The developer may incur economic loss from the exaction but may recoup value, both from approval of the project and from improved marketability. Depending on the legislative authority of the jurisdiction, the proposed development may or may not create the need that the exaction will satisfy.

Exactions add both direct and indirect costs, which must be identified in preparing land development project budgets. Direct costs include those costs associated with design and engineering, hard construction, direct cash payments, operating, and plan review. Indirect costs may include lost project yield and interest-carrying charges incurred in review and delay of approval. These costs will be passed on to the consumer by way of purchase price or leasing fees.

### Land Reservation

Many communities have the authority to require the land developer to reserve private land that may be needed for public use, including parks, school sites, or major road rights of way. Typically, the need and location of these facilities will be identified in the community's comprehensive plan or capital improvement projects budget. The land reservation prevents the loss of these sites as their availability becomes threatened by development. Notice of the reservation usually comes during the earliest stages of project

review. Depending on the size of the project, the reservation may be for the entire property or a small portion of it. Such a reservation does not immediately involve the transfer of ownership. It does, however, obligate the developer to keep the reserved area free from construction for a specific period, during which the local government considers acquisition of the property. The reservation is not a gift, as the jurisdiction will actually pay the owner the land's fair market value if it takes title to the property. Sometimes the jurisdiction will release the reservation after considering the cost of the property, available funding sources, or alternative sites for the intended public use. This allows the developer to proceed with site development activities unencumbered by a reservation.

While the reservation may not create financial obligation or loss for the developer, the requirement sometimes creates difficulties for the land development team. The reservation creates a period of delay or uncertainty, often for several years, until the issue of property transfer is resolved. The land development consultant may be called upon to prepare two sets of development plans to identify the development potential both with and without the reservation. Issues created by a reservation include the developability of the land remaining after the taking as well as how the residue can be designed in terms of access and utilities. Further, the proposed use of the reservation portion of the site must be considered in conjunction with the proposed use of the remainder of the property. For example, if the reservation was for a sewage treatment plant and the residual land was planned for large-lot, expensive single-family detached dwellings, the marketing of such homes becomes questionable at best. Who wants to build or buy a home next door to a sewage treatment plant? In such a situation, the obviously diminished value of the potential of the residual land would become part of the negotiations with the local jurisdiction.

Depending on parcel configuration and the location of the reserved parcel within the overall site, site design opportunities and flexibility may be severely limited. This creates a major challenge for the land development consultant in preparing the most suitable layout for the project. Subsequent release of the reservation allows reconsideration of earlier design alternatives. However, project construction plans may have already been approved or will be well along in the approval process. Therefore, the developer should encourage the municipality staff to allow the processing of two sets of plans simultaneously, in anticipation of early disposition of the reservation. The decision to incur this expense must be balanced against the likelihood of the reservation release and the potential financial benefits from the alternative plans.

### Dedications

Mandatory dedication is a mechanism that is common to most jurisdictions and is usually among the provisions contained in the subdivision regulations. Mandatory dedication

requires the developer to transfer ownership or property rights of certain privately held lands to the governing body at no cost. Mandatory dedications, however, must not violate the Fifth Amendment's ban on taking of property without just compensation. Land most often subject to mandatory dedication includes rights of way for streets and public utilities, floodplains, easements, park and recreation facilities, and similar uses. Such dedications usually are made a condition of subdivision approval. Quite often, localities seek dedications of land above and beyond that which is required to support the required development itself. In these instances, a land use lawyer may need to be consulted.

Once the dedication is made on the subdivision record plat, the government may not always accept the land since immediate acceptance could result in the transfer of maintenance obligations to the jurisdiction before construction of required improvements were completed. Damage from construction vehicles would add to the community's liability. In addition, the governing body must determine whether the property will serve the public good and provide economic benefit. The cost of long-term maintenance of the dedicated parcel is a primary factor in the government's decision whether or not to accept the property. Still another consideration is the fact that the transfer to public ownership also reduces the land area subject to real property taxes.

Most jurisdictions advance density credit when dedication is made. However, some jurisdictions consider certain dedications as a reduction of the amount of land available for development with a commensurate reduction in project density. Transfer of ownership in some cases reduces maintenance and other obligations that may be incurred by the developer or by occupants of the project. In addition to a reduction of operating and maintenance costs, dedicated features can add to the market appeal of certain projects. Often, setting aside land for schools, libraries, parks, recreational facilities, and fire and rescue makes the project becomes more attractive to potential buyers.

Perhaps the most frequent dedication is that of additional right of way along roadways bordering a project. The local government intends this widening to bring existing roads up to current standards and accommodate the new traffic movements associated with the project. Whether construction of improvements is performed by the jurisdiction or required of the developer, the added right of way improves the efficiency of the transportation system and enhances the safety and convenience of site access.

### Preservation

Community residents usually place a high priority on natural, cultural, or man-made features that add to the health, beauty, or character of the community. Often, these features become threatened by a proposed development, prompting the community's desire to protect them. However, the local government may not have the financial resources or legal

authority needed to acquire and maintain them through reservation or dedication. To fill this void, some communities are empowered to preserve these features as an exaction in the development process.

Environmental preservation is often a goal contained in a community's comprehensive plan. Plan language can direct that important environmental or cultural property be set aside in lots or private open space with covenants and restrictions that prohibit future disturbance. This language is implemented as a condition of approval for rezoning or special exception applications.

Environmental and historic protection has also been achieved with the adoption of preservation ordinances that allow reasonable use of the property while ensuring that the desired feature is protected. Most common among these are historical or archaeological preservation ordinances, which may require a developer to carve out the site in question from the project and treat it in a different manner. In some instances, developers have been required to restore historic facades as part of a larger construction effort, such as an office complex.

Recently, communities have sought to adopt tree-preservation ordinances, which limit clearing and grading on portions of the site. Although preserved areas remain in private or communal ownership, covenants and restrictions provide permanent protection against future disturbance by builders or lot owners. Such ordinances often limit the development potential of a site or may involve added costs to accommodate their provisions. Despite these problems, however, these restrictions often present the developer and consultant with unique design opportunities that can enhance a project's appearance and marketability.

### Payments in Lieu

As noted in the prior discussion of subdivision regulations, many jurisdictions commonly require that the developer construct improvements that connect to and/or directly serve the project. Depending on local policy and industry practices, these may be completed prior to the issuance of building permits, or performance bonds may be used to guarantee completion after construction. However, in some instances, the jurisdiction may not require the developer to construct the improvements proposed for dedicated lands. Typically, this happens when the improvement is of little immediate value, such as the widening of a section of abutting roadway where there are no existing or proposed road improvements on either side of the subject parcel. A needed facility may be located off-site, such as a major road or regional stormwater management facility that is not under the developer's control or serves a much broader population. In such cases, a cash payment representing actual construction cost, a prorated share, or a flat fee may be required of the developer. The jurisdiction may wait until sufficient funds are collected and available in conjunction with other developments before it actually constructs the facilities. It may use the payments as reimbursement for public expen-

ditures incurred in advance construction. The funds usually are earmarked for improvements that serve the project or an improvement in proximity to the project. The jurisdiction may also require or allow one developer to construct the improvements and direct subsequent developers to reimburse their prorated share of the costs.

If the project is small, the jurisdiction may also waive the requirement for land dedication. This occurs in instances where the dedication of land for facilities, such as a recreation area, is indexed to the size of the project. If the resulting parcel will be of insignificant size, the jurisdiction may offer the option of cash payment in lieu of the land dedication. In this case, the payment represents the project users' impact on existing or future facilities that will be provided by the municipality.

### Impact Fees

An increasing number of jurisdictions are seeking authority to allow the adoption of impact fees. These are a direct payment by a developer to a jurisdiction and are intended to reimburse the community's actual capital costs of expanding public infrastructure and facilities to service new development. The facilities are not necessarily located on-site and may not be used exclusively by the development that pays the fee. Coupled with the potential for double taxation and discriminatory treatment of new community residents, controversy has developed over government's increasing rejection of its obligation to provide public facilities and services.

Some communities have long had a limited form of impact fee relating to the provision of water and sewer. In this case, a residential or commercial user is assessed a tap fee, also called a connection or availability charge, usually at the issuance of a building permit. The actual fees are derived through an elaborate analysis that projects the capital costs of expanding treatment plants, pump stations, and trunklines to serve new growth. These are facilities that would not otherwise be needed to serve existing users.

However, more recently, impact fees are being considered for such items as roads, schools, libraries, and similar services. What may distinguish sewer and water service from these other facilities is perhaps the degree of certainty with which the demand from and benefit to new users can be isolated and quantified. Those who argue against the adoption of impact fees for roads, for instance, suggest that even where the traffic demand from a new residence is calculable, the benefit of the new roadway is community-wide. Moreover, the inability to separate that residential demand from that created by employment and retail uses further complicates the issue. Lastly, many argue that impact fees for roads and schools are used to correct existing system deficiencies. These are more clearly the responsibility of government, or at least existing residents.

Where the courts have ruled impact fees to be legal, they have adopted several controls on their application. Courts most often employ the rational nexus or relationship theory

to justify the exactions. This requires that it be possible to determine the costs generated by new development and thus avoid charging the newcomers more than a proportionate share. The costs must be uniquely and directly attributable to the development. To implement an impact fee, the community must isolate existing deficiencies and budget separately for their improvement. Fees collected should not be commingled with general funds. They should be earmarked for use in service areas that relate to the development for which the fees are paid. Like transferable development rights (TDRs) and adequate public facilities regulations, special enabling legislation is required to establish impact fee regimes.

Despite the controversy and concern that the fees can add dramatic costs to home ownership and businesses, impact fees for services other than water and sewer are proliferating. Even those communities not authorized to collect impact fees for infrastructure improvements may impose a "voluntary" fee to finance certain major facilities. The community bases the fee calculation on the estimated cost of improvements needed to support the total development projected to occur within a specific service area. This total dollar figure is converted to a unit cost, using the project number of dwelling units or nonresidential building area. The fee applicable to each development project is then based on the project size. Typically, the fee is offered in conjunction with a discretionary rezoning process, with payment being one of the conditions attached to the approval.

### Linkage

Previously, the discussion of affordable housing focused on the adoption of inclusionary zoning ordinances. These ordinances require that residential developers provide affordable, price-controlled housing as a condition of development approval. As an extension of that trend, nonresidential developers may also be required to provide affordable housing. A growing number of jurisdictions argue that new job creation in the community generates demand for new housing for office workers and other service employees. Due to the high land and construction costs, it is increasingly difficult for these workers to find suitable housing in price ranges they can afford. In communities where this linkage is required, the developer must mitigate this impact, either by building affordable housing units or by paying, on a per-square footage basis, into a housing trust fund established by the community. Money deposited in this fund is used by the community to purchase or construct affordable housing.

### Review and Processing Fees

Most state zoning and subdivision enabling acts authorize jurisdictions to collect development review fees from the developer. These fees offset community expenditures for review personnel. In some communities, this fee represents only a nominal fixed charge. However, urban jurisdictions

that perform extensive reviews often index their fees according to project size or the estimated cost of public improvements. These fees may represent a substantial expense. However, they also help to guarantee that sufficient revenues will be generated by new development. This enables the government to maintain an adequate work force of qualified personnel. It provides some assurance to the land development team that project reviews will be timely and thorough. Some jurisdictions also impose inspection

fees to offset the costs associated with that activity. These fees may be charged for inspection of the public improvement construction, as well as for the actual structure. Like review fees, these add to the cost of the development project and must be considered during project budgeting.

#### REFERENCE

Frank, James E., and Robert M. Rhodes. 1987 *Development Exactions*, Planners Press, American Planning Association, Chicago.



## CHAPTER 5

# REAL PROPERTY LAW

*Carlyle W. Blomme, R.L.S. / J. Randall Minchew, Esq. /*

*Craig N. Thomas, Esq. / Edited/Updated: Curt Sumner, L.S., and*

*Gary Kent, L.S.*

### UNDERSTANDING LAW TO GUIDE IN BOUNDARY DETERMINATION

Real property law is a combination of constitutional law, common law, and statutory law. Constitutional law is the supreme legal source in a jurisdiction, taking precedence over common law or statutory law. Common law is that group of rules and standards of action originating from customs and usage as determined in actual cases. The common law derives its authority from judgments and decrees of courts affirming those customs and usage. Statutory law is created by acts of legislative branches of government. Legislative branches of government exist at the national, state, and local level. The surveyor should note that within the broad categories of constitutional law, common law, and statutory law there are many specialized areas of law concerning real property.

Real property law directly affects the practice of surveying. The surveyor works within the established legal system, and therefore the surveyor must have knowledge of real property law to be able to recognize potential legal problems that might arise as a result of the surveyor's findings. Knowledge of real property law also provides a foundation to the surveyor for making decisions to resolve potential problems.

When a surveyor makes his decisions based on the law, a surveyor's clients are best served because such decisions are less likely to be challenged than decisions with no legal foundation. To make decisions founded on law, the surveyor's knowledge of law must include not only real property law, but also the legal rules governing surveyors. Surveyors are governed by rules of state registration boards. State registration boards establish rules under the authority

of enabling statutes. Anyone wishing copies of such rules should contact the appropriate state board.

To make decisions based on law, the surveyor may have to engage in legal research to find the relevant case law. Case law provides court decisions applying the law (whether constitutional law, common law, or statutory law) to a specific set of facts and circumstances. A surveyor must search for case law that applies the law to facts and circumstances similar to the situation confronting the surveyor. While a working knowledge of legal research techniques, constitutional law, common law, statutory law, and the related case law is helpful, the surveyor should be mindful of the fact that the rendering of legal opinions or hypotheses to others may constitute the unauthorized practice of law. The surveyor should never review sources of law and attempt to provide a client with what could be construed as a legal opinion or legal advice.

### Sources of Law

In order to uncover applicable law, the surveyor should be cognizant of the sources of law. As indicated below, there exists a multitiered system of law and sources of law in the United States.

**Constitutional Law.** The U.S. Constitution is the supreme law of the land. The Constitution, technically speaking, is the result of sovereign states granting certain enumerated and specified powers to a federal government while reserving the residue of their powers to themselves. The U.S. Constitution trumps and controls over any state or local laws which conflict with the U.S. Constitution's express or implied provisions. Therefore, the first and primary source of law is the U.S. Constitution and its 27 amendments.

The U.S. Constitution is applied primarily through court orders stemming from actual cases filed in the federal and state courts. The final arbiter and interpreter of the U.S. Constitution is the U.S. Supreme Court. While inferior federal and state courts have jurisdiction to interpret the U.S. Constitution, the U.S. Supreme Court's word is final and controlling. Interpretive questions concerning the U.S. Constitution may be found in the issued written opinions of courts that hear and decide constitutional questions. A listing is included below of those courts that can issue opinions and orders concerning constitution questions. These written opinions and orders may be located in the various reporters (compiled casebooks) in any law library.

Written opinions of the U.S. Supreme Court from 1790 forward may be found in compiled volumes of the *United States Reports* (official reporter) and in the compiled volumes of the *Supreme Court Reports*, *Supreme Court Lawyers' Edition Reports*, or the *Supreme Court United States Law Week Reports* (unofficial reporters).

Written opinions of the U.S. Courts of Appeals from 1789 forward may be found in the compiled volumes of the *Federal Reports* (official reporter) or in the *Federal Courts United States Law Week Reports* (unofficial reporter). There are 12 sitting U.S. Courts of Appeals. The U.S. Courts of Appeals are appellate (nontrial) courts and hear appeals from the U.S. District (trial) courts. Appeals from the U.S. Courts of Appeals go directly to the U.S. Supreme Court. The primary federal courts to which persons bring cases and controversies concerning constitution questions are the Federal District Courts, which conduct trials, collect evidence, and, in some instances, issue written opinions. There exists at least one Federal District Court in each state or territory of the United States. Written opinions of the Federal District Courts may be found in the *Federal Supplement Reports* (official reporter).

Each of the 50 states has a sitting state Supreme Court that occasionally decides cases and issues concerning the U.S. Constitution. The decisions and opinions of these state supreme courts may be found in the various state reporters, such as *Florida Reports*, and *Virginia Reports*, and in the regional reporters compiled by West Publishing Company, such as *Northeastern Reports*, and *Southeastern Reports*.

**Federal Statutory Law.** The U.S. Congress exercises a wide range of legislative powers as allowed under the U.S. Constitution. Bills adopted by Congress and signed into law by the President are compiled in the codified statutes of the *United States Code* (U.S.C.) and, once codified, are identified by a numerical cite, such as "42 U.S.C. § 1983." The many volumes making up the *U.S. Code* may be located in any law library. Once legislated into existence, *U.S. Code* statutes may be the subject of interpretation in the federal courts as is the U.S. Constitution.

**Federal Regulatory Law.** The many departments and administrations of the U.S. government Executive Branch issue periodically regulations and orders concerning the application of federal statutory (*U.S. Code*) law. These regulations, while not rising to the level of Legislative Branch

law created through the Congress-to-President process, are sources of binding law greatly affecting the lives of U.S. citizens. These regulations may be located within the codified *Federal Register* (Fed. Reg.) and, once codified, are also identified by a numerical cite, such as "42 Fed. Reg. § 1983." Once promulgated, federal regulations may be the subject of interpretation in the federal courts as are U.S. Code statutes.

**State Statutory Law.** All 50 states have enacted codified statutes containing laws passed by the respective state legislatures and signed by the respective state governors. These state codified statutes possess names such as *Code of Alabama*, *North Carolina General Statutes*, or *Virginia Code Annotated* and may be located within any law library. These state codes are highly important sources of law because most statutory and regulatory schemes concerning real property originate on the state, not the federal level. State enabling acts concerning the planning, zoning, and subdivision powers of localities may be found within these state codified statute volumes. Individual statutes are identified by a numerical cite, such as "Virginia Code Ann. § 15.1-475." Once legislated into existence, state statutes may be the subjects of interpretation in the state courts and occasionally in the federal courts. The individual state supreme courts have the final interpretive say on what the individual state-codified statutes mean. For example, if the U.S. Supreme Court and the Iowa Supreme Court reached a difference of opinion on what an Iowa codified statute really meant, the opinion of the Iowa Supreme Court would control.

**State Common or Case Law.** As previously mentioned, state supreme courts issue written opinions and orders concerning the interpretation or application of state statutory law and report their decisions in the various state reporters and in West's regional reporters. Almost all of these reported decisions stem from appellate cases originally heard by state trial courts. In understanding how a certain state codified statute is applied or interpreted, one needs to read the reported decisions wherein the statute was reviewed and analyzed by a state supreme court in an actual appellate case situation. In order to assist in this effort, most of the state-codified statutes contain case annotations after each individual statute listing those reported decisions where the statute was reviewed and applied.

**Local Charters and Ordinances.** Individual counties, cities and towns have the power to adopt local codified ordinances within the scope of authority granted by their respective state governments. Zoning ordinances, subdivision ordinances, and site plan regulations are almost always found within these local ordinances. These ordinances are adopted by the respective local governing bodies, namely county boards, city councils, or town councils. As such, the surveyor should own a copy of the local ordinances for each jurisdiction in which he or she works. Most of the legal research into codified law conducted by surveyors deals with local ordinances as opposed to federal or state codified law. In addition to their own local ordinances, incorporated

municipalities (cities and towns) have individual charters adopted by the state governments. These charters are usually found within the local codified ordinances or in a separate volume and contain specific grants of authority or powers to the municipality as determined by the state government.

**Introduction to Property Law.** Laws form the basis upon which land ownership exists. Proof of the limits of this ownership requires gathering evidence by surveying the boundary. The surveyor should remember that for each law there are many exceptions. Each land survey is unique. This is because the physical characteristics of the land vary and the circumstances surrounding the conveyance govern the interpretation of the intentions of the parties.

In addition to this basic knowledge of law, the surveyor must be familiar with state statutes and local ordinances. He or she must have knowledge of case law that refers to boundaries. When a conflict over a boundary develops, the surveyor should investigate any related surveys and cases that may exist.

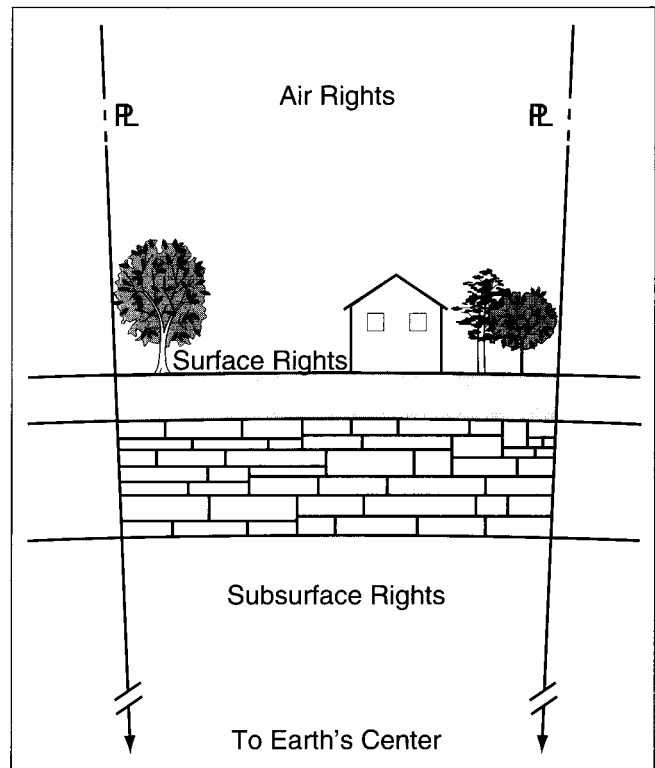
The practice of land surveying requires some understanding of laws related to property ownership. This understanding should include laws related to boundaries, adverse possession, and easements. This basic introduction to law is necessary for understanding of case law related to specific situations. The references to sources of law previously provided should be helpful in this regard.

Property law in the U.S. derives authority from many sources. The U.S. Constitution, laws passed by Congress, and the laws passed by state legislatures are the basis of statutory laws concerning property ownership. Federal boards and commissions pass regulations that affect and control property rights. State constitutions and property laws passed by state legislatures also contribute to the vast number of laws dealing with property ownership rights. Other sources of law include state and municipal regulations, as well as local ordinances. Perhaps most important to the surveyor are court decisions and rulings related to land and its ownership. In reading statutory law together with relevant court-made (common or case) law, one may understand the totality of the law applicable to a given situation.

**Classification of Property.** Property, in the legal sense, is that which belongs exclusively to someone. It is an aggregate of guaranteed and protected rights. Those rights covered by the word "property" fall into various subdivisions. Absolute property, common property, personal property, community property, private property, public property and real property are but some of the categories. "Property" is synonymous with "ownership."

The surveyor or engineer involved in a land development project must be familiar with real property because this category involves the rights of ownership of land and anything erected on, growing on, or affixed to the land. Real property involves the rights associated with the land.

The word "land" in law usually means tenements and hereditaments. A tenement in the proper context is every-



**FIGURE 5.1** Real property rights extended above and beneath the surface of the land.

thing of a permanent nature that one holds exclusively as his or hers. Hereditaments are everything inheritable including both real and personal properties. Hereditaments are much broader in scope than tenements. Land includes the soil and everything attached to the soil. The attachment can be natural, as with a tree, or attached by man, as with a building. In theory, this ownership extends outward to the heavens and inward to an apex, or point, at the center of the earth.

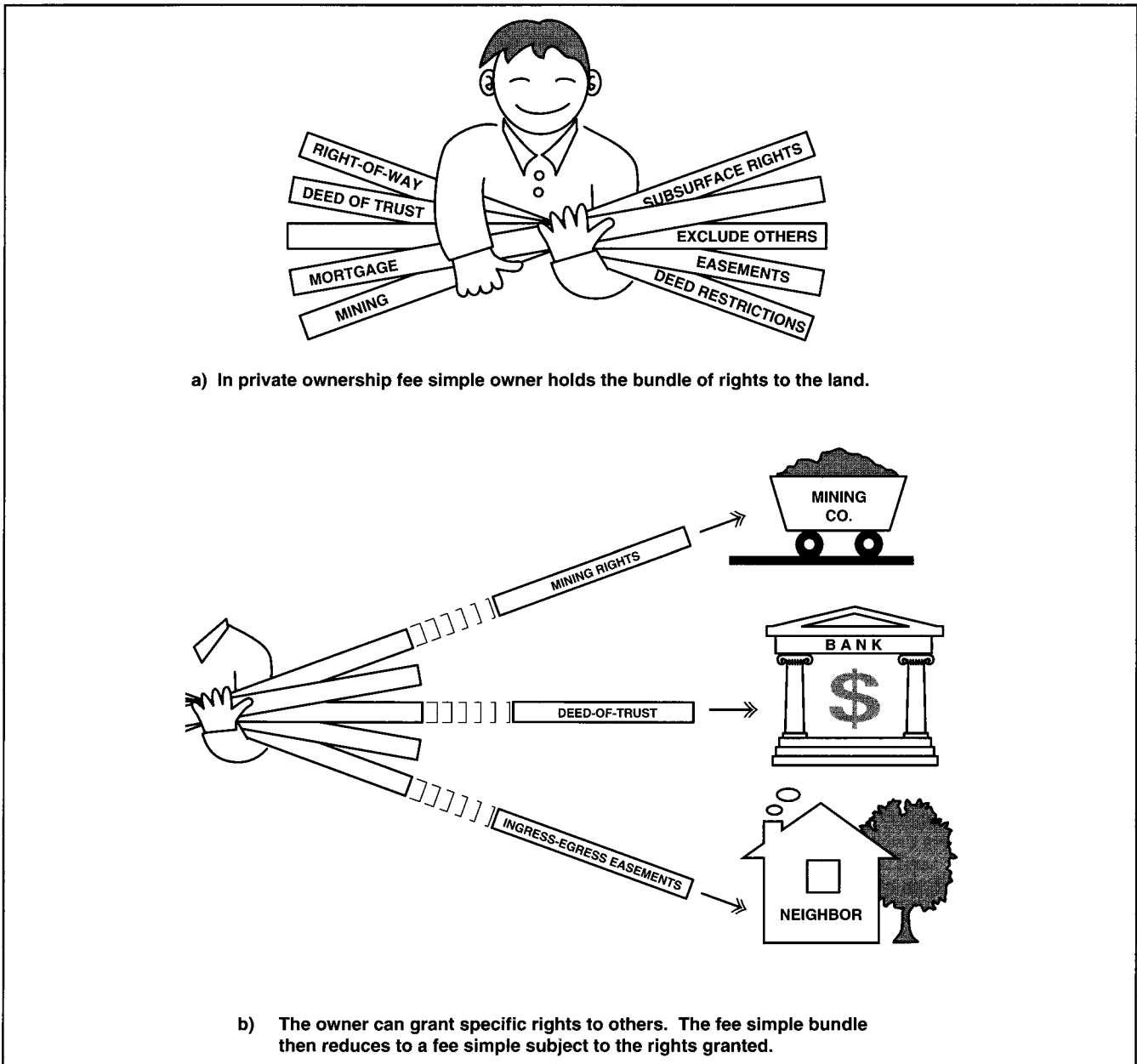
The definitions above are the first step in the classification of property. Ownership is a collection of rights that one has assembled and can use for the enjoyment of property. This includes the right to transfer these rights to others. This collection of rights can and often is a complex one. It is this complexity that generates the need for further classifying rights or property. Property ownership may be described as the possession of a bundle of rights. Therefore, an inquiry into title will include an analysis of which rights a property owner possesses.

Real property and estate in lands include "lands, tenements or hereditaments."<sup>1</sup> A tenement "signifies everything that may be *holden*, provided it be of a permanent nature."<sup>2</sup> For example, a building on a parcel of land would be considered a tenement, as well as the land itself. Hereditaments mean things capable of being inherited whether corporeal

<sup>1</sup> *Black's Law Dictionary*, rev. 6th ed.

<sup>2</sup> *Black's Law Dictionary*, rev. 6th ed.





**FIGURE 5.2** Property ownership is a bundle of rights which can be conveyed in whole or in part.

or incorporeal. Therefore, hereditaments would include tenements and land because tenements and land may be inherited. Corporeal hereditaments are simply “substantial permanent objects which may be inherited.”<sup>3</sup> Incorporeal hereditaments are “anything, the subject of property, which is inheritable and not tangible or visible.”<sup>4</sup> For example, the rents from a parcel of land are an incorporeal hereditament.

The degree of ownership that exists completes the classification. The legal term for this degree of ownership is the type of estate. This is the interest held in land or other

subject property and is possessory or may become possessory. Since this interest can vary in degree, the estates vary in type.

**TYPES OF ESTATES AND THE HISTORY OF THEIR ORIGIN**

William the Conqueror’s conquest of England resulted in all the lands of England being held by the king as sovereign ruler of the realm. No property law existed. The land and everything on it was the king’s. Only the king could grant what was his.

In exchange for a pledge of personal loyalty which included the obligation to fight in defense of the king’s realm, the king would grant to each of his men (the lords and

<sup>3</sup> *Black’s Law Dictionary*, rev. 6th ed.  
<sup>4</sup> *Black’s Law Dictionary*, rev. 6th ed.

barons in his service) the use, possession, and profits of a portion of the king's land. Personal relationships, loyalty, and obligation, not law, governed.

Initially, the king was the sole dispenser of justice among his men, then the king in his court. Likewise, the king was the sole legislator, and later the king in parliament. The only limitations on the king's sovereignty were those the king placed upon himself whether voluntarily or involuntarily. Law is a limitation on a king's sovereignty in the sense that the king has chosen to accede to certain external rules.

However, long before property law existed, the king, to ensure loyalty, would exercise care in his treatment of his men. How the king treated one man created expectations among his other men. Land was no exception.

In time, the personal relationships between the king and his men and between his men and their men grew into a complex web of customs and practices. The result was the creation of a vast structure of land tenure with laws governing personal obligations, interests, and rights concerning land.

The English land tenure<sup>5</sup> system was the "system of holding lands or tenements in subordination to some superior."<sup>6</sup> For example, a knight may have held an estate in land in return for services to the king. Those who held an estate in land, such as knights, sometimes granted an estate in their land to others, who also granted to other persons subordinate to them. This gave rise to middle lords. The tenant in possession of the land was subordinate to all other estate holders.

The tenure system distinguished between free men and those not free. If the man was free, he received a free tenure; otherwise, the tenure was non-free.

Free tenures included spiritual and lay tenures. An example of a spiritual tenure is *frankalmoigne*, which roughly translates as "free alms." *Frankalmoigne* was a tenure that permitted a person to grant lands to religious orders or ecclesiastical bodies as a charitable act for the benefit of the grantor's soul and usually in return for prayers on behalf of the grantor and his family.

Lay tenures included tenures granted for chivalry or knight service, sergeanty, and free socage. Chivalry and knight service were held in higher esteem than the other forms of lay tenure. Tenure by sergeanty included services such as the keeping of the accounts for the land. Free socage tenure included services such as husbandry and baser services. Other types of free tenure also existed at different times in English history.

When this tenure system existed, a person attached to a manor to perform work on that manor was a class of serf

known as a villein. These tenants were slaves. This situation led to a tenure in villeinage or an unfree tenure.

Originally, the common law courts did not protect unfree tenure. Eventually the courts protected the interest of the tenant. The rights and obligations of tenants became a matter of official record; thus, the tenure was copyhold tenure (Cheshire, 1949, pp. 22–26). These copyhold tenures were estates at the will of the lord. The titles to these estates consisted of court rolls prepared by the steward of the manor.

### Modern Classification

The property law system in the U.S. developed out of the English system, with the notable exception of the state of Louisiana, which developed out of the French (Napoleonic Code) system. The United States's property law system is composed of various estates, which can be broadly classified as estates of freehold and estates of less than freehold. Estates of freehold can be divided into two categories: estates of inheritance and estates not of inheritance, specifically life estates. A life estate is an estate created for the duration of the life of some person.

### Fee Simple Estates

Subdivision of fee estates depends on the interest held in the estate. Fee simple absolute is the most complete unencumbered estate known in law. The legal definition of the word "simple" means there are no restrictions on the inheritance characteristics of such an estate. The use of "absolute" in this context means that the estate is not dependent on any happening or event. Transfer of these estates traditionally uses the phrase "to John Doe and his heirs." Court rulings state that the use of the word "heirs" is necessary to create an estate of inheritance in land.

Fee simple determinable estates, by comparison, automatically terminate upon a happening of an event. An example of this type of estate is a deed with a clause that would cause the estate to revert to the original owner or his or her heirs. A statement limiting use of the estate, such as "to be used only as a school," would cause a termination of that estate if the property ceased to be used as a school.

Other fee estates exist, such as fee simple subject to some later condition and fee simple subject to an executory limitation. The first of these is similar to fee simple determinable in that it confers the power on a successor in title to terminate an estate. This termination would take place based on a stated event, but only if the successor exercises his power. Fee simple conditional and fee tail estates were contingent on or applying to heirs. In most states, these estates have been eliminated by statutes.

### Estates Not of Inheritance

Allowing for limited use of estates while at the same time prohibiting inheritance by the user comes about through estates not of inheritance. These estates can be for one's own life. Estates granted to one for the life of another also exist.

<sup>5</sup> Tenure is the direct result of feudalism, which separated the *dominium directum* (the dominion of the soil), which is placed mediately or immediately in the crown, from the *dominion utile* (the possessory title), the right to use and profits in the soil, designated by the term *seisin*, which is the highest interest a subject can acquire. *Black's Law Dictionary*, rev. 6th ed.

<sup>6</sup> *Black's Law Dictionary*, rev. 6th ed.

The legal term for this type of estate is *per autre vie*. Such estates created by acts of the parties are known as conventional life estates. Estates not of inheritance may come about by operation of the law. Operation of the law means that the law provides for estates for one's own life and for the life of another.

Methods used to create conventional life estates include the usage of instruments such as deeds, wills, and reservations. Creation of marital life estates comes about by operation of the law. Record checks for nonpossessory interests such as easements and licenses are always important. Easements that provide access for various services in land development projects make use of such interest.

Other types of estates are estates for years, estates from period to period, estates at will, and estates at sufferance to complete the list of estates. These estates are time-dependent and usually of little consequence to the surveyor involved in land development.

### Concurrent Ownership

Usually individuals hold estates in their own right (severally) without any other party joined with them in the estate. Estates held by a plurality of tenants with a common interest do, however, exist. Such ownerships are joint tenancies, tenancies in common, and tenancies by the entireties.

Conveyance to two or more persons creates a joint tenancy. Each joint owner possesses an undivided interest in the whole property. No joint tenant owns any fractional interest.

Tenancy in common provides that each of the common tenants holds a separate and undivided interest in the land. The possession is, however, in unity with the other joint tenants with equal right of possession under distinct title.

An estate by the entirety is a form of coownership. A clear example of such an estate is that held by a husband and wife where each party has the right of survivorship. Such estates result when a husband and wife obtain land jointly after their marriage.

### Wills and Descent

The will of a party is the wish or desire of that party. In the legal sense a will is the expression or desire of a party to have a disposition of his or her property made. This disposition is to be according to the thoughts in his or her mind. Such an instrument takes effect on the death of a person.

All heirs have equal standing when they receive land by a will, unless the will contains a specific statement to the contrary. This means that those heirs take their parts as assigned in the will proportional to the actual whole part. A simple example of this is where two sons were each willed 50 acres of land from a tract erroneously thought to be 100 acres. The actual area reported from a survey is 104 acres; therefore, each son would receive 52 acres.

Descent is the "succession to the ownership of an estate by inheritance, or by any act of law."<sup>7</sup> Descent can be lineal or collateral. Lineal is a descent from a grandfather or a father to a son or grandson. Collateral descent connects persons not directly related to each other as between brothers or cousins. Descent is the controlling factor when a person dies intestate (without having made a will). The laws of the state of residency regarding descent dictate the division of the deceased estate.

## OWNERSHIP AND TRANSFER OF TITLE OF REAL PROPERTY

### Deeds

A deed is a written instrument used to transfer an interest in land or real property. A deed is a form of contract, and as such a legally valid deed must meet certain requirements. A deed must have at least two parties: a grantor and a grantee. The grantor must own the interest and rights being conveyed. The interest and rights being conveyed must be described with reasonable specificity. The laws of the jurisdiction in which the land or real property is located must permit the transfer of the interest or rights described. The interest or rights being described must be current at the time of conveyance.

Unlike some contracts, which may be oral or written, a deed must be a written instrument. Because a deed is in writing the possibility of fraud or perjury is reduced. This is the underlying rationale of England's Statute of Frauds of 1677, which required that a contract for the sale of land be in writing. The writing shall be in legal words that clearly express the intentions of the parties. These words must identify both grantor and grantee. The written words must convey the intent of the parties. The deed must include a descriptive clause and statement of the consideration involved. This is the *premises*. The deed must also contain a description, called the *habendum* clause, which defines the limits of the estate. This clause must agree with the premises. Words used to close the deed are the *tenendum*.

Deeds must state exceptions and reservations when such exist. The exceptions exclude parts of the estate described from the conveyance. These exceptions must not be greater than the whole estate. The reservations retain rights to the grantor of an estate.

The benefit of consideration to the grantor must exist as stated in the instrument of transfer. Consideration must pass between parties, and that consideration must be adequate compensation. The courts have ruled that a deed is valid only when the grant causes an exchange between parties. The reason for this requirement is to prevent fraud and misrepresentation. It also ensures that one party will not gain unfair advantage over the other party.

Laws require that parties to the deed have the opportunity to read and examine the document before its exe-

<sup>7</sup> *Black's Law Dictionary*, rev. 6th ed., s.v. "descent."

cution. Finally, the deed transfer consists of the signing of the instrument by the parties to the agreement. This step takes place before a notary or other person authorized to witness signatures. There is a requirement of delivery, or the actual placement of the instrument into the hands of the grantee. Strict interpretation of this requirement is rare.

Deeds used in the transfer of ownership in the U.S. are warranty deeds, quitclaim deeds, and deeds of bargain and sale. The warranty deed contains a covenant of title. This covenant is a guarantee by the grantor that the deed conveys a marketable title. Quitclaim deeds convey only the present interest of the grantor and do not warrant or guarantee good title. Deeds of bargain and sale convey definitive estates in land but do not imply a warranty.

### Deed Descriptions

The description is a vital element in the deed. It must be clear in meaning and distinct in location, and the location must be unique to only one property. Two descriptions in a deed define the extent of one's land ownership. The form of the description can vary. Graphics and written words provide two basic methods for identifying land parcels. The metes and bounds or written form is common. This is a combination of measured values (metes) with the bounds (boundary of the land), which when used together define the extent of land ownership. In these descriptions metes comprise the direction and length of travel. Monuments and calls of adjoining landowners fix the bounds.

Metes and bounds descriptions must begin at some known and easily identified point. The surveyor must establish this beginning point in a manner that facilitates its relocation with certainty if destroyed or removed. An additional requirement is that the courses and distances in a legal description proceed from corner to corner and return to the beginning point.

Reference to prior deeds or maps is an important part of the procedure of writing land descriptions. Land description by reference to maps and deeds is a common method used to describe property that is subject to transfer. Strip descriptions and area descriptions are other less-used forms for defining land ownership. Proportional conveyances that call for parts of larger tracts often describe land in states that use the public land survey system. Combinations of various types of descriptions appear in the transfer of ownership of a parcel of land.

The being clause, the body, the qualifying clause, and the augmenting clause comprise the total description. "The being clause of a deed denotes the origin of the history of the present deed" (Brown et al., 1969, p. 347). The body is the actual description. Qualifying clauses are those statements used for exceptions and restrictions among others in the deed. Descriptions incorporate augmenting clauses for easements and other items when necessary.

Any form of description must meet the requirements of the law for a conveyance to be complete. Deeds must be in

writing and must include the grantor (vendor) and grantee (vendee). The description must clearly identify the interest conveyed. There must be an expressed intent to convey the property identified by the description.

Some items can invalidate legal descriptions, but courts have been hesitant to declare legal descriptions ineffectual because of uncertainties. Supportive information often helps to clarify descriptions. Such information is available from the official records, the assessor's office, old maps, and extrinsic evidence.

### Adverse Possession

Ownership under the doctrine of adverse possession comes from statute of limitations related to recovery of real property interests. The modern principle of specifying a uniform number of years as the limitation period for real property actions began during the reign of Henry VIII. Early American colonies followed a limitation period of 20 years as it was set in the 1623 Statute of James I. Several states today have continued that practice. Under the early statute, a dispossessed freeholder had the chance to regain possession and claim to the estate of freehold (seisin).

Property claimants today may recover an interest in real property by bringing an action similar to the ancient writ previously described. The dispossessed party must seek a writ of right or possession before the statute of limitations expires. If the statute of limitations expires before the dispossessed party acts, the adverse possessor's rights in the property become absolute. The modern principle of adverse possession differs because the record owner does not lose title or ownership when the adverse possession begins. He can lose title only after the statute of limitations has run.

On first thought, this concept appears rather harsh, but upon closer examination, it is necessary. There are many reasons to support the doctrine of adverse possession. Among those often advanced is the curing of conveyancing defects that have gone undetected in the possessor's chain of title. Limiting the length of a reasonable title search is a justification used by some. Enhancing the ability to transfer the land and make productive utilization of property is an additional reason. Another is to prevent a record owner from claiming improvements made by the adverse possessor in reliance on the record owner's lack of challenge to the adverse possessor's activities. A final reason is giving certainty to longstanding boundaries.

When the claimant proves that the requirements for adverse possession have been met, the record title owner loses that title. The person with the adverse possession becomes owner of the property. Gaining of title by adverse possession is dependent on many factors. It does not, however, depend on a deed or patent. The possession must be open and notorious, continuous, hostile, actual, visible, distinct and exclusive, and under a claim of title. To satisfy the required statute of limitations, the statutory period to defeat title is necessary (Brown et al., 1969, Sec. 2.34).

It is imperative that elements inherent to this type of possession exist before the courts will uphold claims under this doctrine. The record owner must have actual knowledge of the adverse claim against his holdings. Such terms as *open*, *notorious*, *actual*, and *visible* raise the presumption at law of notice. Intentionally infringing on the rights of the true owner with notice is necessary to meet one of the requirements.

Exclusive possession is also an essential ingredient for adverse possession. One manner of showing is if the adverse claimant denies right of entry to or use of the property by anyone without his permission. The claimant must deprive the rightful owner of his possession. Land used in common with others is obviously not exclusive possession, but if that use is only by permission of the adverse claimant it is still exclusive use.

Legal rights are not the only basis for hostility. Hostility exists when an adverse claimant occupies property as his or her own with full intentions to hold it regardless of any other person's rights or title. *Hostile possession* is a term often misunderstood. Case law provides example and clarification of the term. The majority rule is that possession based on a survey mistake is sufficiently hostile, as long as the person in possession is claiming the property as his own (Powell and Rohan, 1987, ¶ 1013[2][f][i]).

The requirement of continuous possession is necessary so that the true record owner of the land has had suitable cause while the time of possession runs its course. This allows him to bring legal action to evict the adverse claimant. Any break in the possession by an adverse claimant before the statutes of limitations expires, no matter how short the break in possession, reestablishes possession to the title owner.

Good faith requires that the person making an adverse possession claim truly believes that he or she has obtained a good title. He or she further believes that the taking of the title was proper. This requirement is not always necessary but is contingent upon the type of possession.

While ownership may be established by adverse possession, a surveyor may only survey lines based on and described in record documents (except when a new parcel is being created). Thus, when adverse possession is a possibility the surveyor's responsibility is to show the evidence thereof in relation to the boundary lines as described in the records. The final determination of a successful claim of adverse possession lies with the courts, not with the surveyor.

## Easements

The general definition of easements is a right granted by the owner of a parcel of land. These rights, granted to another party, are for use of the land for a specified purpose. Easements are critical to land development, and this section will include more details concerning them. Various types of easements exist for various purposes.

The manner and reason for creating an easement determine its type and category. There are many common examples of easements, and nine prevalent means exist to create them: grant, dedication, condemnation, statutory cartway, prescription, necessity, implication, expressed reservation, and estoppel. This list is provided to apprise those persons involved in land development of the many types of easements that may exist. The surveyor or engineer must uncover existing easements before the commencement of design work on the development project. Knowledge of the various means of creating them should assist in this task.

Easements appurtenant and easements in gross form two broad categories that include the various types. An easement created for use with specific land, the dominant estate, is an easement appurtenant. Such an easement consists of privileges of a person to use the land of another, the servient estate. This use must be in a particular manner and for a particular purpose. Such an easement can restrict the rights of the servient owner and may prevent him from using his own land for certain activities. Failure to restrict an easement to a dominant estate creates an easement in gross. Easements in gross are personal interests or rights to use land belonging to another. These uses are independent of ownership of any specific property.

Creation of easements by expressed agreement can rely only upon a deed of grant. This is true regardless of the duration of the interest conveyed.

Dedication of an easement consists of the appropriation of land to the public use. The true owner must make such dedications and acceptance must take place for use by the public. Easement by condemnation is the process whereby property of a private owner is taken for use by the public. When this occurs without consent of the private owner, that owner does receive an award of compensation for his or her loss.

Acquisitions of easements by statutory cartway proceedings provide for ingress and egress to public highways over intervening land. These easements provide for this access when no other reasonable way is available for cultivation, timbering, mining, manufacturing plants, or public or private cemeteries. State statutes vary, as do the methods for establishing these cartways.

Easements acquired by long and continual use by an individual are easements of prescription. The required period of use is usually the same as that for accomplishing adverse possession. This period is a prescriptive period and varies among states.

An easement of necessity comes about when parties grant land but fail in providing outlet to a highway exit except over the land remaining with the grantor. In such instances, an easement of necessity provides for an outlet over the seller's remaining land.

Quite often, public and private cemeteries obtain access through easements of necessity. As a matter of sound practice, however, express deeded easements should be provided when cemeteries are created.

Implied easements evolve through implication, prior use, necessity, or prior map or plat dedication. The crucial element for an implied easement is that of prior uses. Often nondocumentary, these easements must be recognizable through a reasonable inspection of the property. A property survey should therefore reveal easements of the implied type. The requirement of appearance and visibility of such easements extends beyond professional scrutiny and includes the grantee of property. They constitute necessary and reasonable use of the property subjected to these easements. Such easements can affect land development.

An example of an implied easement follows. Party A owns two lots. Lot one contains the home of party A. A sewer line runs from this home across the second lot owned by party A. There is a catch basin located on the second lot. Party A sells the second lot to party B. The visible catch basin is sufficient evidence and notice to party B that an easement for drainage, over the lot mentioned, exists.

Creation of easements by reservation and exception allows an owner who conveys a possessory interest to a party or parties to except or exclude a corporeal interest from the terms of his grant.

Estoppel forms the basis of creation of some easements. These easements can restrict the grantor in the use of his land. The following example from the case of *Battle Creek v. Goguac Resort Ass'n*<sup>8</sup> illustrates such a situation. In this instance, the Goguac Resort was a riparian owner on a lake. The resort sold land for an easement to Battle Creek knowing that the city wanted water from the lake for municipal purposes. The resort company was later estopped from use of the lake because the resort use contaminated the lake making the water unfit for the city.

Affirmative and negative easements are both common in occurrence. Affirmative easements are those that allow activity on the estate burdened by the easement. Some activities mentioned in easements restrict burdened estates. When such is the case, negative easements result.

Examples of affirmative easements include alleys, the approach to airports, private roads through subdivisions, and railway rights. Other easements include pipelines, utility poles, and electrical transmission lines. Party wall agreements and public utility dedications are also easements. When appropriate, use easements to cover the use of springs and wells. Important, too, are surface rights to serve oil and gas leases. Easements provide a way to grant rights to flood land or drain land. Other uses for easements are to allow for encroachments and to provide for excavation along boundaries. They also exist to carry out temporary building construction beyond the limits of the project. Some easements listed above are nonaccess easements, but most include rights of entry for reasonable maintenance.

If a current title search, commitment, or policy is not provided, the surveyor may search the public records to

find documentation of express easements. An express easement would constitute a separate grantor-to-grantee conveyance in the title search. If the surveyor does not undertake such a search, the client should be so notified.

Negative easements are those that prevent specific activities by the servient estate owner. These usually prevent certain types of improvements to protect the easement owner's rights. Such rights might include scenic views and access to sunlight.

Locations of future easements, which are required because of the development project, should receive consideration while the boundary survey is in progress. To identify potential future easements, the surveyor should reference service lines of different types that adjoin or cross the subject property. The owner of property burdened with an easement and the benefited dominant interest (the easement owner) can seek remedies. Typical of remedies available to aggrieved burdened property owners are self-help, award of damages, injunctive relief, declaratory judgment, quiet title judgment, and replevin. Actions available to aggrieved easement owners include all the above, with one exception, replevin.

The definition of self-help is an action that falls short of judicial action. Such action cannot result in a breach of the peace. A good example of self-help develops when an easement owner removed a garden from an area covered under some type of easement. The garden, planted by the burdened landowner, interfered with rights of the easement owner. The action of the aggrieved easement owner in no way breached the peace.

Damages result from unreasonable interference with easement use or unwarranted use of easements. Granting an easement usually makes the easement owner responsible for the maintenance and care of the area covered by that easement. Such an example existed when a young boy entered onto property burdened by an easement for a power line and climbed a tree growing within the easement. In climbing the tree, the boy touched the power lines and sustained injury. In this case, the court upheld damages sought against the easement owner. The court reasoned that the power company was the rightful easement owner and therefore unable to shift liability to the burdened property owner.<sup>9</sup>

The most common relief available to easement owners or burdened property owners is injunctive relief. Through this type of relief the easement owner would seek the enforcement of the rights of the easements. A burdened landowner would likely seek such an injunction to stay the use of the easement. Remedy by injunctive action will not preclude awards of damages.

Declaratory judgments, in particular, provide clarity when defining the limits of easements and their location. One may seek the assistance of the court in the determi-

<sup>8</sup> 181 Mich. 241 (1914).

<sup>9</sup> *Gnau v. Union Electric Company*, 672 S.W.2d 142 (Mo.Ct.App. 1984).

nation of the extent and dimensions of easements not clearly defined. This is the method of accomplishing a quiet title action. Such matters can become crucial. Easements left undefined when producing a boundary survey will often subject land development projects to unwanted problems.

A purchaser can bring legal proceedings to clear a purported easement through a quiet title action. For the buyer to take action, he needs neither to give notice nor to wait for the owner of the easement to respond to a complaint. The purchaser can take the initiative for removing a purported easement from his property. This is the most effective means of clearing property from encumbrances to convey title without objection. A quiet title action can also serve an easement owner in the establishment and determination of extent of an easement.

In one illustrative case, *Castanza v. Wagner*,<sup>10</sup> the burdened property owners sought quiet title and an injunction. They were successful in restricting the easement across their property to the width of the easement (between 12 and 16 ft). A recorded real estate contract made at the time their predecessors obtained the burdened property contained the reservation. A few weeks following the initial real estate contract's execution, the same grantor entered into a contract to convey the parcel benefiting from the reserved easement. The contract purported to transfer a 60-ft-wide easement across the neighboring, burdened land. The owners of the 60-ft easement attempted to widen a roadway within their claim, but the court did not allow the widening.

Replevin is an action available to a burdened landowner enabling him to maintain control over his possessory rights. This control prevails in the face of the nonpossessory rights of the easement owner. One example of particular interest to those engaged in land development and concerning replevin involves public dedication of subdivision streets.

In this example, the landowner of a development dedicated the streets in a subdivision. The city of Lawton, Oklahoma, then annexed them. The owner of the subdivision asked for reimbursement for water and sewer mains constructed by him. These lines lay within the dedicated limits of the streets. The city maintained that these installations were part of the dedication. The Oklahoma Supreme Court held that a landowner does not usually convey fee title and possessory rights when making a street dedication. The court allowed the developer to recover the value of the improvements under the street. This recovery of the value of improvements came about in an action of replevin.<sup>11</sup>

The term *profit à prendre* is defined as a nonpossessory interest in land with the right to take soil or substance from that land. This right to take something distinguishes *profit*

*à prendre* from easements that only confer a right of use. Use of this profit often determines easement type. When profit derived from the easement is of benefit only to the dominant estate, then the easement is appurtenant. Use of the profit in ways other than to benefit the dominant estate creates a profit in gross. Since appurtenant means belonging to the easement, it is also an accessory to the subject tract. An easement appurtenant will pass to a grantee even if the transfer deed fails to mention it because of its relationship with the dominant tract.

### Title Report

Identifying and addressing issues affecting title to real property is a critical element in the transfer of ownership during the land development process. A title examination by an attorney will track the chain of title to real property over a specified period of time and may uncover conveyance of a portion of the property either in fee simple or in rights of use, such as easements. Typically, however, the potential purchaser or the entity lending the money to purchase property will require that a title report be prepared by a company from whom a commitment for title insurance is to be issued. The owner of property not being sold, but intended to be developed, may also wish to obtain a title report to uncover potential pitfalls that may affect plans for development.

Title insurance that protects the investment of the lender is the most common form, although insurance to protect the interests of the buyer is also available at an additional cost. From the perspective of the lender, only insurance that protects his investment is required in most circumstances.

The title report is intended to reveal any information that may have an effect on the ability of the purchaser or owner to use the property. It will include a description of the property, any conveyances of ownership or use, liens or judgments against the property, and any special exceptions that may affect how the property can be used.

Special exceptions are shown in a separate category from the other items listed; typically the category is called Schedule B, Section 2. The items listed in this section are typically referenced as survey exceptions and are reviewed by a surveyor when asked to provide a survey of the property. The surveyor will determine which of the special exceptions he is able to comment on and what effect (if any) they have on the use of the property.

When the surveyor is able to determine that certain of the special exceptions listed do not affect the property at all, the title insurance company will more than likely remove those items as exceptions to title for the purpose of issuing a commitment for title insurance. Of the special exceptions determined by the surveyor to affect the use of the property, those that are capable of being graphically depicted on the survey drawing will be shown. Special exceptions that affect the property but are of a general nature, or those whose location is inadequately defined, will be

<sup>10</sup> 43 Wash. App. 770, 719 P.2d 949 (1986).

<sup>11</sup> *Selected Investments Corporation v. City of Lawton*, 304 P.2d 967 (Okla. 1956).

noted by the surveyor, but their location cannot be depicted on the survey drawing.

The title report is an important element in the feasibility analysis conducted to determine whether a property is suitable for the user's intended purpose. A survey of the property will support the title report by identifying whether its boundaries actually match the description noted, and by addressing whether or not special exceptions listed affect the property. The standards usually required for the performance of the survey are those established by the American Land Title Association (ALTA) and the American Congress on Surveying and Mapping (ACSM). ALTA/ACSM Land Title Survey requirements will be discussed later in this chapter.

### **Eminent Domain**

This is a right of the state that affects the rights of the private property owner. Eminent domain is the power held by governments and certain quasipublic entities to take private property for public use. Appropriation of private land for road construction, drainage channels, and laying of water lines is common. This provides another way of obtaining access across private land. Normally, this power is reserved to governments. Many states, however, have adopted legislation extending this power to private individuals and other legal entities. In these instances, the power allows private owners to gain access to inaccessible (or landlocked) land.

Eminent domain differs from easement by necessity. The former requires compensation from the party claiming the power. Just compensation for land interests condemned is required by the U.S. Constitution and by some state constitutions. Easement by necessity requires no additional compensation other than the amount paid in the conveyance of the property involved. Land developers might use this power to gain rights of way in some situations, but the method is one of last resort because of the cost involved.

### **Dedication**

Dedication is the voluntary granting of land interests by the rightful owner for public use. This topic is important because it is dedication that provides the means of transferring land from private ownership to public use. Land development projects include various public services within planned areas. Dedications to city or county governments provide for their long-term maintenance. Only the fee owner or his authorized agent can make these dedications.

Common law and statutory law provide for dedications. Common law dedication confers only an easement. This dedication is not a transfer of rights. Such dedications do prevent the burdened landowner from exercising his rights in a manner inconsistent with the rights of the public. This form of dedication is not a grant of land because there is no grantee. Common law dedications transfer the land by estoppel in pais. Estoppel in pais is an estoppel by conduct

of the parties, compared to estoppel by deed, which rests on public records.

Statutory dedications occur when there is a grant of rights. These differ from common law dedications because they pass legal title covering the area so dedicated to a governmental body or agency. Statutory dedication must comply with statute law. Laws concerning dedication vary from state to state. To constitute a dedication either expressed or implied there must be an intention on the owner's part to grant the property to some public use. A dedication is a voluntary action on the part of the fee owner. The party alleging a dedication must prove the intentions of the other party to do so (Skelton, 1930, p. 435). Open and visible conduct by the parties allows courts to determine their intentions to dedicate land for a particular use.

Plats show actual intention to reserve any portion of lands for the public good. If such a reservation is not on the plat, then an equally certain method is essential for establishing the intention to reserve for dedication. Public notice of the intention to reserve is equal to that given by a plat. Intentions to dedicate land without supporting evidence or acts signify nothing.

Before dedications become binding on either party, there must be certain proof of the acceptance of the dedication. The acceptance can be actual, expressed, or implied. Until there is an acceptance by the public, the public has no rights, and neither has the public assumed any responsibility.

### **Created Boundaries by Agreement**

When it becomes necessary to ascertain a boundary other than by reference to legal descriptions contained in deeds, then agreements between parties serve as a basis for establishing them. Establishing the boundary line in this manner will not qualify as a conveyance of property. Such an agreement supports the boundary location as intended in the deed of conveyance.

Boundary line agreements can occur in cases when the boundary location is ambiguously described in the conveying deed. In these instances, the agreement still does not result in a conveyance of the land that lies between the agreed line and the true line. The agreement between parties does, however, establish a new line. The descriptions of lines set out in this manner carry title up to the agreed line regardless of their accuracy. The intention of the parties not to claim ownership according to the true line is consistent with the doctrine of agreed boundaries.

Boundaries by agreement are dependent on finding a mutual agreement on the boundary location. Boundary agreements entered into by adjoining owners recognize that three legal obstacles must be overcome before such agreements have force. The English Statute of Frauds, adopted by a great majority of the states, requires that all agreements affecting an interest in land must be in writing if they are to be legitimate. Protection of the rights of an innocent third



party is a concern. It is essential that they have the same knowledge of agreements as the parties to the agreements. All states accomplish this by maintaining a system whereby deeds are recorded and wills probated. These systems of recording documents must comply with the doctrine of constructive notice. This doctrine requires that other parties may review records that can and often do affect them.

Courts often require that certain factors be present before establishment of boundaries based on such agreements. Boundary agreements must involve lines or corners that are uncertain or disputed. These agreements must set out a specific line as the boundary. Execution occurs by acts of the parties occupying adjoining land, and the agreement must include land to the agreed line. Recognition of the agreement for a considerable period is the final requirement. This period is often equal to that required for adverse possession.

### Created Boundaries by Acquiescence

Acquiescence is similar to boundaries by agreement, but this type of ownership is unique because it does not require proof that the boundary location resulted from an agreement. The justification for this ownership is similar to adverse possession. For this doctrine to be enforceable, occupation must be visible up to a definite marked line. Typically, a long time must elapse before ownership by acquiescence exists.

The case of *Provonsha v. Pitman*,<sup>12</sup> is a good example of establishing boundaries by acquiescence. The disputed tract dated back to 1884. The disputed area was within the metes and bounds description claimed by the plaintiff, Provonsha. The area was also included in a deed for 160 acres south of the plaintiff's tract. The area involved in the dispute was 15.1 ft wide at one end, 7.5 ft wide at the other, and 462 ft long. It represented about 5% of the plaintiff's land but was an infinitesimal part of the total land value.

Construction of a barbed wire fence took place in 1898 to divide a 462 ft square block equally between spouses in a divorce action. The fence remained at the time of the trial in 1957. Repair of the fence had occurred over the years and sections had been replaced several times. Ownership of the tracts on both sides of the fence had changed hands many times. Adjoining owners respected the fence and occupied and used land to this physical barrier. In 1935, a new owner had a survey made that showed the fence to be in error. Neither this owner nor any of his successors in title took any action other than making statements about the erroneous location.

In 1955, the plaintiff sued to regain the disputed tract. He based his claim on the conversations, incidents, and understandings of recent owners. The court found no evidence of anything but occupation and use of the land up

to the fence by either of the parties and therefore said acquiescence established the boundary.

As with adverse possession, ownership may be established by acquiescence, but a surveyor must only survey the record lines. Thus, when acquiescence is a possibility, the surveyor's responsibility is to show the evidence thereof in relation to the record lines.

### Created Boundaries by Estoppel

Estoppel is a legal act resulting in a person being forbidden by law from speaking against his own actions and deeds. It is a remedy that prevents allegations and denials of certain facts previously stated as true.

Boundaries by estoppel come about when the true owner knowingly misrepresents his boundary and causes a neighbor to rely on the representation and therefore incur detrimental cost. In such cases the boundary becomes that as represented. Estoppel manifests itself when "a man's own act or acceptance stops or closes his mouth to allege or plead the truth."<sup>13</sup> It is to bar, stop, impede, prevent, or preclude denial of a certain fact. In the situation of boundary by estoppel, the following example illustrates the principle. Should a property owner know that an adjoiner is making improvements next to or along a line that the parties have falsely considered an actual boundary line, then estoppel prevents him or her from later claiming a true line running through the improvements.

This situation existed in the case of *Pitcher v. Dove*.<sup>14</sup> In this case, the defendant (Dove) had taken possession of land by a sheriff's deed. Dove had lived on the land before purchasing it at the sheriff's sale. The description called for beginning at a particular corner and running a particular distance and direction. Dove based his claim on running a particular direction to a stone set by the county surveyor. This claim gave Dove a greater distance than called for in the description included in the sheriff's deed.

On half a dozen occasions during this period, Dove and the plaintiff (Pitcher) had physically traced the property lines on the ground. On those occasions, Pitcher had pointed out the stone set by the county surveyor as the corner. He further advised Dove to purchase the land in question. When Pitcher and Dove inspected the property, they found a fence on the boundary line. This fence agreed with the set stone. Pitcher had stated that the fence was on the line. Pitcher later moved the fence to correspond with the call in the sheriff's deed. The court found for Dove, although no mention of the stone appeared in his deed. The court found on the grounds of estoppel. Dove had based his actions on Pitcher's representation of the stone as the corner.

The four criteria necessary for a judgment by estoppel were present in this case. The estopped party must know

<sup>12</sup> 6 Utah2d 26, 305 P.2d 486 (1957).

<sup>13</sup> *Caulfield v. Noonan*, 229 Iowa 955 (1940).

<sup>14</sup> 99 Ind. 175 (1884).

the facts that form the basis of the estoppel. This party must have acted in a manner such that the second party had a right to believe that he could act on the statements and conduct of the estopped party. The party claiming the estoppel must have no knowledge of facts to the contrary from those represented by the other party. Finally, because of the statements and conduct of the estopped party, damage must have occurred to the person claiming estoppel.

As with adverse possession and acquiescence, ownership may be established by estoppel, but a surveyor must survey the record lines. Thus, when estoppel is a possibility, the surveyor's responsibility is to show the evidence thereof in relation to the record lines.

### Liens and Mortgages

Both liens and mortgages affect property ownership and therefore are concerns when land is in the process of development.

Liens are a claim on property. They occur because of some obligation or debt on the part of the owner of the property. The lien is the means of securing the claim or debt owed to the lien holder. The operation of the law provides a means of acquiring a lien. This happens when the law advances a lien without having the stipulation of the parties concerned. There is a period between the reasons for a lien and the actual filing of the liens.

Title companies often depend on survey reports to point out possibilities from which liens can evolve. Such items as building construction, alterations, and repairs not paid for can result in the filling of material and labor liens. The surveyor should note improvements to street and sidewalks because they too can be a source of liens. Construction of water and sewer lines is also an indication of possible liens. Determination of the actuality of these possible liens is a matter of law and beyond the realm of the survey.

A mortgage is an estate created to secure an act by a creditor. In most instances, the act is the payment of a sum of money by the grantor of the mortgage to the grantee of the mortgage. The mortgage becomes void when the grantor meets the terms of the mortgage.

### Action for Ejectment

An action for ejectment is a legal action for prevention of possessory interference with a property owner's title and provides a means to test the right of possession. Ejectment has evolved into a legal remedy for the protection of an owner having good legal title. Such an action provides a means of recovery of land only on the strength of the plaintiff's title. One use of ejectment is the settling of boundary disputes before land development. Simply put, ejectment is a suit that provides a method of trying land titles. This, like all legal actions, is usually a long and costly process.

### Surveys for Transfer of Developable Real Estate

In 1962, the American Land Title Association and the American Congress on Surveying and Mapping jointly de-

veloped and adopted a set of standards for conducting surveys in conjunction with the closing of commercial loans on real property. These standards, the Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys, have been periodically revised. As of the date of this publication, the current version is dated 1999 (see Technical Appendix).

The ALTA/ACSM Minimum Standards provide for a nationally defined and accepted set of survey requirements. Today they are required in conjunction with most real estate transactions on properties contemplated for development. As such, the surveyor and other interested parties should be familiar with them. Additional information regarding these standards is given in Chapter 13.

### LAW OF EVIDENCE

The combined rules and standards that control the factors determining information and facts presented in legal proceedings compose the law of evidence. The law of evidence regulates the determination of admissibility, relevance, weight, and sufficiency of the evidence presented in a legal proceeding.

Evidence may be defined as "Any species of proof, or probative matter, legally presented at the trial of an issue, by the act of the parties and through the witnesses, records, documents, concrete objects, etc., for the purpose of inducing belief in the minds of the court or jury as to their contention."<sup>15</sup> Evidence is not proof but the consideration of all of it as presented and the deduction reached from it leads to truth. A preponderance of evidence will prevail in civil cases. Not all evidence carries the same weight in establishing proof.

Surveyors should be cognizant of evidentiary laws. If a surveyor ignores evidentiary considerations, he may find his work given no weight as evidence in a court proceeding. For example, courts have ruled that surveys started at monuments not lawfully established, or reestablished, under the rules of evidence have no probative force. A basic understanding of the law of evidence will prove a helpful tool to the surveyor.

### Indispensable Evidence

Indispensable evidence, as the name implies, is that evidence necessary to prove a particular fact. The following statement provides an example. Land boundaries based on adverse possession must establish the fact of that possession by presentation of evidence such as enclosures. The surveyor would therefore locate fences that would constitute indispensable evidence of such enclosure.

### Undisputed Evidence

Undisputed evidence is conclusive evidence. The clear contents of a written instrument, such as a deed, remain un-

<sup>15</sup> *Black's Law Dictionary*, rev. 6th ed.

altered by oral testimony. When trying to prove the location of a boundary, one considers in high regard those documents that are a part of the conveyance.

### Prima Facie Evidence

*Prima facie* evidence will stand court test until rebutted by other evidence. A probated will granting a party ownership to a tract of land is *prima facie* evidence of that ownership. Proof of the rebuttal of the will would be necessary to contradict such evidence.

### Additional Types of Evidence

Other classes of evidence consist of primary evidence, secondary evidence, direct evidence, indirect evidence, partial evidence and extrinsic evidence, and hearsay evidence. Primary evidence is original or first-hand evidence. It is that evidence that is given priority for ascertaining the truth. A copy of a document is secondary evidence because of its inferior position compared to the original document.

Direct evidence supports the existence of a fact without depending on proof of other facts. The testimony of an eyewitness to an event is direct evidence.

Indirect evidence, on the other hand, requires proof of one fact by proof of another. This evidence is circumstantial evidence because of its dependence on other facts. Evidence used to establish a detached fact is partial evidence. This type of evidence is subject to rejection unless connected to the fact in dispute by proof of other facts.

Extrinsic evidence lies beyond the body of a document for agreements or contracts. Such evidence is primarily for explanation of references in the documents. This evidence can clarify meanings of terms that vary within local context.

Hearsay evidence is not original or does not come from personal knowledge of a witness. This type of evidence comes secondhand by way of what a witness has heard someone else say. This evidence is admissible in only a few instances. Even in situations where such evidence is permissible, its credibility would be a matter for the jury to decide.

### Collecting Evidence

The four forms for gathering evidence are oral testimony, written documents, real or material objects, and judicial notice. Oral or parol evidence is that given by mouth by a witness in a court. Another method to obtain evidence is from contracts, wills, deeds, and other types of written documents. Documents presented as evidence are usually self-explanatory, but witnesses may give testimony to explain ambiguities when they exist. The courts place limitations on the types of documents allowed for presentation. This is a matter for consideration when preparing for a court appearance. Real or material objects such as monuments, fences, and other physical evidence are most important in support of the proof of a fact. Judicial notice is the act of recognizing the truth (universally established, such as a his-

torical event) related to the facts of a case. This recognition by a court takes place without supporting evidence.

### Consideration of Evidence

Other considerations in the rules of evidence include real and demonstrative evidence. The same chapter of the statute law addresses topics such as declarations and admissions, weight and sufficiency, and questions of law and fact. The understanding of evidence is most important to the surveyor involved in land development. His or her determination on the location of property lines and corners usually provides the basis for development and construction work to progress.

Often a surveyor's work is nearly complete before a legal problem is detected. Should the land under development become involved in litigation, the surveyor's interpretation is subject to admission or rejection in court, depending on his adherence to the rules governing the law of evidence.

One author has stated that "Everyone is presumed to know the law and the surveyor is no exception. If he agrees to monument a certain written conveyance on the ground, he also agrees to locate the conveyance in accordance with the laws regulating the interpretations of written conveyances" (Brown and Eldridge, 1967, Sec. 2.19).

### WATER LAW

Many facets of water law affect land development. Property rights and public rights are the predominate factors that control water management and usage. Navigability distinguishes private rights from public rights. Rights vary depending on whether the water is surface or subsurface. Flowing waters, contrasted with those found in a body, also affect rights.

Land development requires proper management of water. This management includes surface and subsurface water. Some knowledge of water law is necessary before water resources planning can begin. Failure to consider appropriate water law can be devastating to the land development process.

Water is always in motion. As part of the hydrologic cycle, water exists in varying states (vapor, rain, snow, dew, surface water or groundwater), but no matter the state, it is water. A property owner's rights concerning water use depend on a number of factors, including whether the water is atmospheric, surface, or subsurface. Laws control the use of water flowing in a stream or existing in a body such as a lake. The laws of riparian rights or of prior appropriations govern this control. When water seeps into the ground, rules for subsurface flow govern the manner of its control. Yet another set of laws determines the use of water flowing in a dispersed state across the surface of the earth.

When waters are navigable, they become public waters. The problem lies in the determination of what constitutes navigability. The rule often given for this determination is that if "water is navigable in fact it is navigable in law." Recent laws have defined navigability beyond this and

based the question on flow. To declare water navigable by statute law, when that water is not navigable in fact, gives the owner of the water rights due compensation. In most situations, statute law is more limiting on property rights than is navigability. In situations such as these, surveyors and engineers involved in land development should be familiar with the federal statutory law definition of “navigable waters.”<sup>16</sup>

State laws, as well as federal laws; determine the water rights of an individual owner. States base their laws on two basic, but different, doctrines. The two doctrines developed to control water uses are riparian rights and appropriations. In most instances, the courts have integrated and modified these two basic doctrines to develop many of today’s current water laws.

### Riparian Rights

Riparian owners’ rights include use of the water and encompass those rights for the enjoyment and development of his land. Issues and lawsuits related to riparian rights are much more common in the western states, where water resources are more scarce, and thus more valuable, than in the eastern states. These rights of use must not infringe on the rights of other riparian owners to exercise those rights. If all riparian owners have the same rights in the same body of water, then the use must be reasonable by each owner.

The property must adjoin a natural watercourse for these rights to exist. Additionally, the land must lie within the watershed of the riparian waters for riparian rights to exist. Once severed, these riparian rights remain lost from the land, and the loss is a permanent one. A littoral owner is a particular type of riparian owner. A littoral owner is one that joins a body of water such as a lake or sea. The rights of a riparian owner include access to the water.

Other rights include extraction and use of the water. The riparian owner only can preserve the water in its natural condition. The following example illustrates unnatural use of water. When a riparian owner takes water from a river for use in a cooling process at a canning factory belonging to the riparian owner, the result is “unnatural” if the water temperature is higher when returned to the river than when the water is first removed from the river.

Riparian owners, in most situations, have rights to all the water necessary for domestic purposes. Upland owners have this right of use even if it deprives lower riparian own-

ers of water required for that use. This domestic use includes water for the care of livestock but not for use by cattle raised for commercial purposes.

Riparian rights give to the owner of those rights the use of water in riparian watercourses for irrigation. The reasonable use rule and the claims of each riparian owner declaring his or her rights to those waters restrict the usage of each.

The conduct employed by the parties provides a test of the doctrine of reasonable use. The test, when applied by the courts, resolves conflicting rights of riparian owners. Temporary interruption in normal flow of riparian waters usually does not constitute grounds for liability for damages. The validity of this depends on the circumstances involved and the jurisdiction where the case occurs.

When riparian waters are navigable, in fact the public has a right to use the water for recreational purposes. This further modifies and limits the rights of the riparian owner.

Should the body of water change course gradually, leaving land exposed, the riparian owner gains property rights to this newly revealed addition. Such a process is accretion or reliction. The precise definition of the term *accretion* is a gradual increase of the land. Soil deposits left by the body of water adjoining the accretion land cause this increase. Reliction is a process that exposes the land previously covered by water. This exposure occurs by a gradual subsidence of that water. Boundary lines move with the shifting shoreline. *Avulsion* is a term that describes the sudden and abrupt change in a land formation. This action is an act of God. Such changes do not affect boundary locations and thus are different from accretion or reliction.

### Surface Water

In most cases, water classified as surface water allows for its consumption by the owner of the property upon which such water exists. The disposal of this water can pose problems, particularly in land development projects. Under early legal doctrine, landowners could dispose of unwanted surface water. Such disposal could take place artificially without incurring any liability from neighboring landowners. The legal name applied to this doctrine is the “common enemy” rule. Under this doctrine, adjoining landowners could protect themselves from surface water by any means available.

In some states, this rule still applies to surface waters that naturally drain from one property to another. The majority of the states have modified this rule. These modifications mean that only reasonable changes can take place, without liability, in the flow of these surface waters. Under the modified rule, land developers are liable to adjacent owners for increases in runoff.

Other states do not follow the common enemy rule but follow instead the civil law doctrine. This doctrine allows surface waters to flow from one property to another if that flow is in a natural channel. Lowerlying owners cannot obstruct this flow. The downstream owner may take reason-

<sup>16</sup> 16 U.S.C. § 796(8): “‘Navigable waters’ means those parts of streams or other bodies of water over which Congress has jurisdiction under its authority to regulate commerce with foreign nations and among the several States, and which either in their natural or improved condition notwithstanding interruptions between the navigable parts of such streams or waters by falls, shallows, or rapids compelling land carriage, are used or suitable for use for the transportation of persons or property in interstate or foreign commerce, including therein all such interrupting falls, shallows, or rapids, together with such other parts of streams as shall have been authorized by Congress for improvement by the U.S. or shall have been recommended to Congress for such improvements after investigation under its authority.”

able steps to protect against this flow. Landowners may seek protection under a modified version of the civil law doctrine. Combining this doctrine as modified and the common enemy rule as modified results in both becoming a reasonable use doctrine.

The degree of harm inflicted on the downstream owner, the foreseeable harm, good faith, and cost-benefit factors furnish the criteria used to judge reasonable use.

### Subsurface Waters

These waters include percolating waters, underground streams, and artisan waters. Under common law, the surface landowner has the absolute right to extract percolating water. This right exists without regard to the impact on adjoining owners. Many states modified this law into reasonable use. Bordering landowners share rights under this doctrine. The same rules of law as those that control surface waters control underground streams if both lie in the same government jurisdiction.

When following the common law rule, the surface owner has absolute ownership of percolating water. Landowners have corresponding rights in the percolating waters when the rule of reasonable use controls the jurisdiction of land ownership. The doctrine of prior appropriation gives ownership of the percolating water to the state. The water is available to surface owners based on respective state laws.

Landowners may become liable for obstructions in the flow of percolating water. Extremely hazardous activity that causes interruption of flow would likely cause such liability. If the surface owner acts in a nonnegligent manner while in the process of developing his land, then such acts carry no liability for damage to percolating water.

Differing views exist concerning the liability of a surface landowner. Responsibilities and rights in subsurface waters related to the surface owner might also hinge on the type of subsurface water involved (percolating waters, underground streams, and artisan waters). When a surface owner pollutes percolating water, he usually incurs liability related to the manner of his use. If the surface owner uses his land rationally, he will incur no liability for his actions. This often holds true even when his or her actions result in injury to another landowner. They can incur liability if the land use is reasonable but the use was negligent on their part. Some government authorities hold landowners liable for pollution regardless of the manner of their use. Owners can become vulnerable to claims without the presence of negligence and unreasonable use of the land. When this situation occurs, liability exists without fault.

The question of “reasonable” and “negligent” or “unreasonable” obviously can become a matter of hair-splitting. Complications in interpretation of water law also arise because of variation of interpretation between jurisdictions. The following court rulings shed some light on the dilemma involved in making a determination relative to usage. A case from the Arkansas courts illustrates the “reasonable use” of ground and surface water. It should be noted at the onset

that Arkansas is a riparian rights state and follows a modified doctrine that includes “reasonable use.” The case used for illustration is that of *Jones v. OZ-ARK-VAL Poultry Co.*,<sup>17</sup> where the Arkansas Supreme Court ruled that “reasonable use” applied to all underground waters as well as those on the surface. In this case the court went further and accepted the correlative rights doctrine set forth in *Hudson v. Dailey*<sup>18</sup>. Correlative rights further modify the rule of “reasonable use” to a proportionate share of the underground water to the surface landowner in times of short supply of the water. In effect, this ruling indicates that reasonable use of water relates to available supply. Therefore, as supply changes, what has been reasonable use can become unreasonable. As with all situations under law, each case involves circumstances peculiar only to the case at hand. It is not the intention here to imply that a rule exists for the determination of reasonable vs. unreasonable. The intention is to point out possible pitfalls that might arise out of the land development process.

### LAND USE

In a chapter dealing with property laws, it is appropriate to include a section on land use. Ownership of property relates closely with the right to use that property. Laws controlling those uses are worthy of mention in this section.

All phases of land development should begin with extensive planning. The first step in the planning process, preceding the technical planning, is a market analysis. This study can be the difference in success or failure for any development. A need assessment can determine the type of development having potential for success and the design suited for the site location.

Land and site planning involve evaluation of the site. Locations of specified land uses occur during this part of the planning process. Topography and access play a major role in these determinations. Engineering and surveying follow as they become necessary for the precise location of items set out in the planning stage. Most municipalities require engineering plans before site development can begin. These plans show street grades, earthwork, storm water drain lines, sanitary sewers, utilities, and water supply lines. Also incorporated in these plans are material schedules necessary for the development’s construction.

Planning and designing buildings require the services of an architect early on in the development process. In many instances, the architect can help with site planning. The architect assists in overall coordination of style used within a land development project.

If the planning process is to proceed in an orderly manner, all participants must consider legal implications. Legal topics discussed to this point are common to all property surveys. When a land development is pending, then legal

<sup>17</sup> 228 Ark. 76 (1957).

<sup>18</sup> 156 Cal. 617, 105 (1909).

topics related to that development become important factors. They can affect and control the planning and construction process.

### Land Use Regulations

Common law and statutory law provide for the regulation of land by private and public means. Private land use regulations, such as recorded covenants, usually exist through arrangements entered into by private parties. Laws and courts provide for enforcement of these arrangements.

The law of nuisance provides a form of private regulation also. Nuisance law pertains to the control of “that which annoys and disturbs one in possession of his property, rendering its ordinary use or occupation physically uncomfortable.”<sup>19</sup>

The law of nuisance is a doctrine that, while allowing a landowner to make reasonable use of his or her land, does not allow a landowner to deprive other landowners in the community of the reasonable use of their land. Enforcement of nuisance laws usually results from the actual physical activity causing the nuisance. The greater the physical impact on the land, the greater the possibility of the court granting relief to the affected party.

Public regulation of property is achieved through zoning laws. Governments possess, as an incident of their police power, the ability to regulate the use of land to safeguard public health safety and welfare. Particular land use zones established under zoning ordinances regulate land use.

### Ordinances

Ordinances are laws initiated by authority of a local legislative body such as a county board or a city or town council. Local governing bodies adopt subdivision ordinances to provide for a healthy living environment. These ordinances set up specifications that produce better design of the building lots while providing control over the creation of new lots. In many jurisdictions, an unsubdivided portion of a master lot cannot be conveyed to a new owner. In some cases, an unsubdivided portion can be conveyed, but only under strict guidelines that, for example, might limit the size of the parcel being created. Thus, subdivision and the creation of new lots are required to allow more individuals to become landowners.

### Restrictions

Restrictions can exist in the form of deed restrictions and restrictive agreements. Zoning laws are but one form of restriction. A grantor cannot pass title to rights that he does not possess, but he or she has the option of disposing of fewer rights than he or she possesses. When the grantor chooses to do this, he or she may destroy these rights or in some cases may retain them for himself or herself. Clauses in deeds, which place limits on rights, are restrictive

covenants. Most often these clauses are not restrictive but rather offer protection. If written for the community, these restrictions protect against undesirable use of the land.

Restrictions are either public or private control over the use of property. Simple examples of each follow. A private land developer purchases a tract of land and employs professionals to subdivide it into some smaller parcels. The surveyor or engineer will map and properly record the subdivision. A note on the face of the map designating all lots for residential use only is an example of private building restrictions. A city or county zoning ordinance that sets a particular area aside for industrial use or other areas residential use is public control.

Courts provide means of enforcing restrictions. The party seeking to enforce restrictions can petition the court to issue an injunction (order) to halt the violation. Anyone found to be in contempt of such an order is subject to contempt of court sanctions, which may include incarceration.

### Subdivision Development

Subdivision, broadly defined as the creation of smaller lots from a larger tract, falls under the jurisdiction of a local authority in most states. One writer, in referencing a locality's specific subdivision law, states, “any land, or portion thereof, shown on the last preceding tax rolls as a unit or as contiguous units which is divided for the purpose of sale, either immediate or future, by any subdivider, into five or more parcels within any one year shall be considered to be a subdivision and requires the filing of a map for the approval of the planning commission and the legislative body” (Gallion and Eisner, 1975, p. 266). Subdivision regulations, however, differ widely among local jurisdictions and surveyors should be well acquainted with these local ordinances.

Subdivision development tends to be thought of as a relatively new process. Shortly after the Norman Conquest of 1066, William the Conqueror established a form of subdivision control in England requiring permission from the Crown for divisions of baronial estates. History shows this activity to be one of the earliest forms of man's social coexistence with other men. Subdivision development and new cities on this continent trace their existence to the first settlements. The Royal Ordinances for the Laying Out of New Cities, Towns or Villages, circulated by Phillip II of Spain in 1573, are an example of early regulations intended to control land development. Many early town charters provided for methods of laying out church sites, town markets, streets, and lots. Colonial charters also controlled subdivision of lands for new towns. The Continental Congress regulated the division of western lands through the adoption of the Public Land Survey System.

By the 19th century, earlier subdivision and land controls, together with the exploitation of the land by developers in “get-rich” schemes, led to further regulation requiring accurate surveys and maps of proposed subdivi-

<sup>19</sup> *Black's Law Dictionary*, rev. 6th ed., s.v. “nuisance.”

sions and regulation, in many states, of street width and alignment. Before lots could be sold, local officials were required to verify surveys and maps. Public official review and approval of public dedication of streets was also required before lot sales could take place. The public dedication requirements and procedures became clearly established.

In 1928, the U.S. Department of Commerce published the Standard City Planning Enabling Act (Standard Act). State legislative acts followed this Act, and by 1961, nearly 1,000 cities with populations more than 10,000 had developed comprehensive subdivision regulations. Today, all 50 states have adopted some form of enabling legislation concerning subdivision law. Land developments are subject to more stringent controls than at any other time in history.

Subdividing land is not as simple as completing a survey and preparing a map that includes lots and streets. Subdivision regulations mandate the laying out of streets and division of land into blocks and lots. For the process to be beneficial and orderly, subdivision laws must be enforced in accordance with their tenor and not interpreted by political sentiments. While civilization exists, new land developments will begin and old land developments will expand.

Today, subdivision control goes beyond the size of lots and the horizontal and vertical requirements for streets. These controls also surpass the requirement of installing on-site roads, sidewalks, water and sewer lines, drainage, and other utilities. Some jurisdictions now require dedications of land for parks, schools, sewage treatment plants, and other public use, providing a sufficient nexus exists between the dedications demanded and the impacts caused by the subdivision.

Many courts have upheld requirements for such dedications. One city required that 4% of the land development area be set aside for parks and playgrounds. One court said this requirement was reasonable and upheld it because the new development created the need for the park and playground.<sup>20</sup> The limit on the amount of such dedications and exactions allowed by law in subdivision process differs greatly between states and between different jurisdictions in a state. In a recent case, the U.S. Supreme Court has ruled that a "rough proportionality" must exist between the exaction sought and the slated public purpose involved.<sup>21</sup>

Recent land development regulations address environmental concerns. These controls often require the developer to submit project plans to a government agency. This agency will grant approval if there is no indication of adverse impact on the environment. The State of Vermont has adopted a new land development law administered by the Environmental Control Board through seven environmental district commissions. Developers must obtain permits for proposed subdivisions from this board. This law is an ex-

ample of the tendency in land development controls to curb negative environmental impact.

The approval process for subdivisions will vary from one place to another, but the overall procedure is much the same.

1. Review and understand the local subdivision ordinance.
2. Survey the land to obtain an accurate description of the boundaries.
3. Proceed with surveys to locate abutting streets, determine topography and locate special features.
4. Locate any structures and easements existing on the site.
5. Determine any restrictions imposed on the property.

Water lines that serve the subject property should be located if they exist. State and local requirements for sanitation disposal and provision of potable water should be determined. Information on water table heights, soil conditions, and prevailing wind directions is important. This information may affect lot layouts.

The surveyor should check subdivision ordinances for restrictions. They often include lot size, setback lines, and other specified items. In addition, the surveyor should review the local zoning ordinance for other limitations. Subdivision regulations typically control street width and grades. The ordinance usually includes the procedure for presenting maps and plans to the controlling government body.

The developer should employ an engineer or surveyor to prepare preliminary plans for the proposed development. The local subdivision ordinance provides a guide for the preparation of such plans. These plans should show the approximate size and shape of the proposed lots. The preliminary plans include street locations, street grades, and the method of providing drainage for all areas within the development. These plans also indicate land use, utilities, and zoning.

The surveyor should locate public facilities such as schools, parks, and playgrounds and record distances from these facilities to the proposed development site. Available transportation and shopping facilities affect the needs assessment. Cost estimates determine the feasibility of the proposed development. Sound developments usually depend on investment moneys for home mortgages and development cost, including streets, utilities, and earthwork.

Various agencies require filing of proposed development plans. The local planning commission is the coordinating agency in most localities. After approval of the proper agencies, the development can proceed. Submission of a final set of plans to proper authorities usually precedes lot sales.

### Shopping Centers

Land use for commercial purposes covers a broad range. It includes the regional financial center through the small cen-

<sup>20</sup> *Aunt Hack Ridge Estates Inc. v. Planning Comm.*, 160 Conn. 109 (1971).

<sup>21</sup> *Dolan v. City of Figard*, 114 S. Ct. 2309, 2314 (1994)

tral business district down to the smallest commercial unit, called the neighborhood center. Each of these units must meet a specific need. Controlled designs of this type include provisions for traffic, parking, and floor space. The type of center determines the appropriate balance between these factors.

One can categorize centers by the population that they serve. Neighborhood centers serve between 7,500 and 20,000 people. Community centers provide more variety in service and reach a population in the 20,000 to 100,000 range. Regional centers are complete with major department stores and serve a population from 100,000 to 250,000. A thorough analysis should precede the building of any center to provide for needs assessment.

Most neighborhood centers require approximately 5 acres of land for the site. Sites for community centers are larger and often are 10 or more acres in area. Regional centers are around 30 acres. Consult local regulations for definite control of zoning and restrictions.

### Industrial Parks

These subdivisions are a very specialized type of development and require special knowledge of industrial requirements. Today these parks serve a community in much the same manner as the industrial districts of earlier times. Such parks are not suitable for all industry.

These parks offer benefits to industry and communities. Industrial parks can relieve industry of legal problems stemming from zoning. The site will offer utilities and other services, and therefore the industry will encounter no problems from local government in securing such services. Often there is a significant reduction in site development cost to the industry that locates within such a park. Other beneficial factors available to industries that locate near each other include security, eating facilities, and clubs for employees.

Industrial parks, as is true of most subdivisions, must have a set of protective restrictions. These occur in the form of covenants carried in deeds and leases. They help ensure compatibility among occupant plants and between the park and the local community. Standards must be sufficiently high to be acceptable to the nearby residential communities. They must not be so rigid that they become unacceptable to the very industries sought by the park.

Properly developed and administered restrictions can make land usage predictable and therefore protect property values in the park and the area surrounding it. Protective covenants in most industrial parks regulate both permitted and prohibited uses. They control emissions of smoke, noise, heat, industrial waste, light, odor, and other environmental factors. Included in typical restrictions are such items as site size, site coverage, building line setback, parking, building, and construction types.

The communities that house industrial parks reap benefits from the diversification of the local economy. Benefits that come with successful industrial parks include a broader

tax base, more income into the community, and a general stimulus to the area.

### City-County Planning and Zoning

City and county planning processes are as old as the governing bodies themselves, but zoning processes are of 20th century vintage. Past planning may or may not have been as good as that seen today. Modern urban planning and zoning processes leave these roots in the Standard City Planning Enabling Act of 1928. Tomorrow's generations will criticize today's planning in a different light because planning is forecasting and forecasting is most difficult. In the past, forecasts have tended to underestimate growth in our cities. This has created many of the problems that one sees today. It has also made solutions to others difficult and more costly to solve. At the root of city and county planning is the plan for an adequate transportation system, and as this system becomes more complex our planning process must become more comprehensive. Good planning will meet the demands and problems of the present.

One objective of the planning process includes increasing economic efficiency by coordinating the size and location of physical facilities with projected future needs. Planning should provide an overall design for urban expansion that will be aesthetically pleasing and retain the natural integrity of the land. The planning process also serves to allocate land to varied uses necessary for stable and healthy growth. Properly executed planning will enhance the relationship of various land uses.

A major factor in the planning process is control, and a U.S. Supreme Court ruling has qualified zoning laws as an essential part of those controls.<sup>22</sup> Planning and zoning commissions exert control over subdivisions and other land use areas surrounding cities. The land developer has the opportunity to produce well-planned land developments. However, he will need the help of a land planning team composed of engineers, surveyors, architects, and land planners.

Zoning is a plan or process that, as the term suggests, divides a city or county into zones. Each of the designated zones serves a particular use. Provisions for zoning ordinances come from the state's police powers. These are necessary to protect the health and safety of the public. Such zoning prevents overcrowding, establishes appropriate sanitary regulations, provides for a more efficient transportation system, and protects quiet residential areas. For example, zoning lessens the danger of fires by banning factories from residential districts.

Zoning ordinances were simplistic in the beginning. Division of a city into three zones, residential, commercial, and manufacturing, was usual. Trends in zoning are evolutionary, and modern ordinances create more classifications. Residential districts are no longer exclusive to single-family dwellings but provide for multifamily as well.

<sup>22</sup> *Warth v. Seldin*, 422 US 490 (1975).



Subdivision of commercial zones into retail and wholesale areas is usual. Subclassifications used in industrial zones are *heavy* and *light*.

Zoning is exclusionary by its nature, and the courts have ruled that zoning that excludes a class is unconstitutional. Examples of these exclusions are apartments, mobile homes, and even zoning exclusively for large lots. Some courts have held valid those ordinances that specify minimum lot sizes, but these requirements must be reasonable and must be for the health and safety of citizens. The courts have been consistent in ruling that the exercise of zoning powers are impermissible if the result renders land economically useless. Variances in ordinances allow flexibility and prevent hardship on landowners of particular tracts. This is true for tracts that can be shown unsuitable for a certain zone. Ordinances also provide for nonconforming uses. These are uses in existence at the time of conception of an ordinance that conflict with the uses established by the new ordinance for the affected property.

Zoning can be for aesthetic purposes. Bulk zoning regulates the size and shape of buildings and their location on a property. Zoning ordinances usually require property owners to obtain building permits before erecting any structure. Applications must accompany plans and specifications. It is through these permits that enforcement of zoning takes place.

## SUMMARY

It is the intent of this chapter to provide a brief overview of legal topics that often affect land development. The section on use of law libraries should provide a foundation for the reader to carry on specific research in those areas of individual interest. This chapter is not a substitute for legal counsel and should not be used by non-legal professionals as a basis for providing legal opinions or legal analysis, or as encouragement to engage in the unauthorized practice of law. Further, this chapter is a supplement to Chapter 1, "Boundary Surveys for Land Development."

Real property and the law controlling and protecting the individual's rights in that property are at the root of our democratic way of life. Under our system, classification of rights of ownership and defining the limits of that ownership as these limits relate to the classification provide order and structure to real property ownership. Instruments such as deeds provide proof of rights held in property. Additional rights that guarantee full usage of property include easements, water rights, and other items. These provide for greater enjoyment of land ownership. All these topics be-

come vitally important under our system of laws and holding of real property.

Orderly progress in land development rests on the system of land ownership provided for under the laws related to such ownership. However, some areas such as zoning and planning can on occasion impede development.

## REFERENCES

- Brown, C. M., and W. H. Eldridge. 1967. *Evidence and Procedures for Boundary Location*, John Wiley & Sons, New York.
- Brown, Curtis M., H. Frederick Landgraf, and Francois D. Uzes. 1969. *Boundary Control and Legal Principles*. 2d ed., John Wiley & Sons, New York.
- Cheshire, G. C. 1949. *The Modern Law of Real Property*, 6th ed., London, Butterworth.
- Gallion, Arthur B., and Simon Eisner. 1975. *The Urban Pattern*. 3d ed., D. Van Nostrand, New York.
- Powell, R., and P. Rohan. 1987. *Powell on Real Property*, rev. ed., Matthew Bender, New York.
- Skelton, Ray Hamilton. 1930. *The Legal Elements of Boundaries and Adjacent Properties*. Bobbs-Merrill, Indianapolis, In.

## ADDITIONAL READING

- Backman, James H., and David A. Thomas, *A Practical Guide to Disputes Between Adjoining Landowners—Easements*, Matthew Bender & Co., New York, 1989.
- Brown, Curtis M., Walter G. Robillard, and Donald A. Wilson, *Evidence and Procedures for Boundary Location*, 2d ed., Wiley-Interscience, New York, 1981.
- Burby, William E., *Handbook of the Law of Real Property*, 3d ed., West, St. Paul, Minn. 1965.
- Cribbet, John E., William F. Fritz, and Corwin W. Johnson, *Cases and Materials on Property*, 3d ed., The Foundation Press, Mineola, N.Y., 1972.
- Goodman, William I., and Eric C. Freund, *Principles and Practice of Urban Planning*, International City Managers' Association, Washington, D.C., 1968.
- Graves, Charles Alfred, *Notes on the Law of Real Property*, Bobbs-Merrill, Indianapolis, In., 1912.
- Grimes, John S., *A Treatise on the Law of Surveying and Boundaries*, 3d ed., Bobbs-Merrill, Indianapolis, In., 1959.
- Kratovil, Robert, *Real Estate Law*, 6th ed., Prentice-Hall, Englewood Cliffs, N.J.
- Mack, William, and Donald J. Kiser, *Corpus Juris Secundum*, vol. 2, West, St. Paul, Minn.
- Hoagland, Henry E., *Real Estate Principles*, 3d ed., McGraw-Hill, New York, 1955.
- Second Decennial Edition of the American Digest*, St. Paul, Minn., 1918 [a complete digest of all reported cases from 1906–1916].

# PART II.B SITE ANALYSIS AND SITE SELECTION

## CHAPTER 6

# ENGINEERING FEASIBILITY

William Fissel, P.E.

### INTRODUCTION

Development projects often begin with the developer or landowners approaching the land design team with a plat showing a piece of property, asking, "What can I do with it?" Others ask, "I would like to develop this for a specific use . . . can I?" The answers to those questions require more than a five-minute office consultation to check the property's zoning. The experienced consultant must recognize the unspoken parts of those questions that are perhaps more indicative of the level of investigation and commitment required. "Can I make a profit?" "Is the expense and effort worth the return?" "Can I complete the development in a reasonable amount of time?" "Will people buy what we have to offer in a predictable period of time?" "Can I make a valuable contribution to the community and my reputation?" These are the questions that a developer needs answered. To answer them requires extensive evaluation of all of the components, participants, and dynamics of land development that impact on the project and property.

Land development is highly regulated at all levels of government. The developer may spend considerable amounts of money to demonstrate that the development program and the design comply with those regulations. This must be done without any guarantees that construction will ever be authorized and profit will be realized. In addition, because the design, processing, and construction period can take several years, even a well-conceived project may find no buyers. Economic and demographic forces keep changing, while the development program remains essentially the same.

In a competitive market where vacant land is in short supply, decisions must be made in a relatively brief period

of time. Otherwise, the land may be lost to another buyer, usually a competitor. In periods of tight money supply and slow economic growth, the decision to purchase must be well reasoned. The developer must be assured that the investment will provide economic return. This is particularly true if rising land prices have not abated, despite weakness in the real estate market. The developer must base the commitment of resources to purchase land on a determination that the land will have future value and use.

The engineering feasibility study identifies problems likely to be encountered in planning, government review, and construction, as well as potential uses for the land.

Land development can be a risky business. It is this engineering feasibility study that serves to help a developer find the answers to basic questions to minimize the risk. Such questions include:

- What are the physical characteristics of the site? Are they conducive to the type of land development envisioned?
- What regulations apply?
- What are the costs involved to conform the infrastructure to the envisioned development?
- What is the timing of the design and approval processes?

An initial engineering feasibility study should be organized to assist the developer in obtaining answers to such land related issues *before* the land is purchased. This study can take place either prior before negotiations with a landowner or during a purchase contract option period. If the study suggests that an unfavorable price has been asked for

the property, the developer has an opportunity to reject the purchase or renegotiate the price. Meanwhile, the land is protected from purchase by other buyers. Failing to perform a thorough investigation can lead to costly mistakes and, in some cases, expensive lawsuits. At their first meeting, the consultant should advise the developer of the importance of performing an engineering feasibility study as part of their scope of service agreement.

To perform such studies, one must understand the utility, value, and potential use of a tract of land. These are based on a number of underlying principles concerning physical and economic characteristics common to all land. Physical characteristics of land include its *immobility*, that is, it has a fixed location and cannot be moved to avoid or take advantage of other locational factors. In addition, from the government's standpoint, its fixed location makes it easy to regulate, tax, and attach. Land is *unique*, with no two properties being identical in size, shape, elevation, view, or access to natural and man-made resources. Notwithstanding certain natural forces that cause erosion, inundation or landslide, land is *indestructible*. This reflects the stability of investment made in it. Economic characteristics include its *scarcity*, with no more land being created and with unused land in particular locations in increasingly short supply. The nature of *improvements* affects not only the value of the land, but that of adjoining land as well. Land represents a *permanence of investment*. While certain improvements can be destroyed, the public infrastructure usually remains that investment is characterized as long-term and stable. Perhaps most important are the public's *area preferences* to location, which result in similar structures being valued differently in alternative locations (Galaty et al., 1988, p. 21).

Physical constraints and opportunities of the site need to be identified and quantified to assist the developer in evaluation of the property. In addition to these inherent characteristics are those that affect its physical adaptability to certain uses. For example, consider two parcels of waterfront property, one flat, the other steep. The likelihood that the latter can be used for maritime purpose is limited. In addition, practical limitations on the mobility of humans, automobiles, machinery, and equipment affect convenience and ease of certain uses, especially when coupled with climatic differences. Roads, parking areas, and service drives must be designed to accommodate these limitations. For instance, if a road must be built on steeply sloped land, it may not be easily negotiated by an automobile. In climates where snow and ice are a factor, such a road may be rendered impassible for days or weeks at a time. The slope of sidewalks and trails must accommodate human capabilities. Long, steeply sloped walkways and stairways from streets and parking areas may not only be a hindrance to occupants, it may prove a market deterrent as well. In many instances, the law requires that access by people in wheelchairs must be considered. Graded slopes must safely accommodate maintenance equipment, such as mowers. While site grading during construction attempts to elimi-

nate practical conflicts, existing topography and local ordinances sometimes make extensive regrading economically infeasible. The purpose of the engineering feasibility study is to establish a framework for making such decisions.

Lastly, the value of land is affected by government regulations that control its development and use. These laws in many ways supersede and restrict the rights associated with land ownership while protecting the public's health, safety, and welfare. Development potential may be severely constrained by environmental factors. As a consequence, site development may not provide sufficient yield to return a profit. The cost and timing of securing development approval or complying with local, state, and federal regulations may be so severe as to detract from a tract's value as a development site. Changes in development regulations also affect parcel value.

Land developers who routinely operate in most communities already are very familiar with the land development process and expectations. The developer often is sensitive to the area's real estate market and can visualize project layout. Many are familiar with local attitudes and the political motivation of area leaders. Yet most developers engage consultants during the acquisition feasibility option phase because of the need to assemble information prepared by unbiased professionals. Even if it is feasible to employ a large full-time staff of development specialists, their judgment might be considered prejudicial in negotiations with property owners. In addition, the consultant brings the benefit of the experience gained in working with many clients and projects. This cumulative knowledge about the process is valuable to even the sophisticated developer.

In some instances, multiple sites are available to the developer. Each site might have some unique element, location, topography, zoning, access, or infrastructure that separates it from other sites. If there are several sites, a site selection study needs to be performed to evaluate the different aspects of the site to determine which site is the best for the developer and his program.

In cases where rezoning is inevitable, political and legal issues will have to be addressed. The developer should retain an attorney familiar with zoning for these purposes. In these instances, the consultant should refrain from offering legal advice for which he or she is not licensed. The consultant may act as technical advisor to the attorney and developer. However, unless the consultant is extremely experienced with local regulations, he or she rarely becomes involved in the details of the zoning laws. It is important to note, though, that the land design team will become involved with the interpretation and application of zoning ordinances and comprehensive plans. These interpretations are usually as important as the laws themselves and become central to evaluation of the development for approval.

This chapter describes the important studies performed for the developer, leading up to the conceptual planning and detailed engineering design required for the prepara-

tion of construction documents. It explains their purpose and utility to both the developer and government agencies and includes discussion of the steps involved and resources needed to perform the analysis. Not all of these studies are performed by the developer's principal consultant, although they must understand the information in the study and implications for planning and design. However, the land design team should be involved at the earliest possible time to coordinate the activities of professionals in other disciplines.

### ENGINEERING FEASIBILITY STUDY<sup>1</sup>

The engineering feasibility study is a critical step in the development process and ideally is performed before the actual purchase of land. The information produced by this study is needed in assessing a potential development site's constraints and opportunities. The results are used to evaluate physical, environmental, regulatory, or other constraints that must be overcome or accommodated in constructing the intended use. The results of this study often affect the purchase price of the property, which is frequently based on presumed development potential. Uneducated assumptions about development potential frequently prove to be in error because of physical, locational, or external characteristics not properly considered.

The study is also important in providing legal protection to the prospective owner or developer. The undetected presence of flood plains, underground utilities, and other constraints such as hazardous waste sites, contaminated soils or groundwater, or underground storage tanks could subject the owner to expensive cleanup operations or litigation under federal laws.

The developer will use information from these preliminary studies to procure loans and to present to other individuals engaged in making project-related decisions. For this reason, the acquisition/feasibility study should be completed before the actual purchase of the land. The developer should insist on having a study period established as a contingency clause in the purchase contract. The purchase of the land may hinge on the information in the study as well as other contingencies. Time and money are the major concerns of the developer. The study is performed with developer's funds. If the developer elects not to purchase the land, this is money that will not be recouped from the project. For this reason, it is to the developer's benefit to incur as little expense as possible at this early stage. Some purchases may be contingent on whether a rezoning or other application (such as special exception, variance, or subdivision) is approved. Others may hinge on whether a minimum number of buildable lots can be obtained. It should be emphasized that where there is a clear intent to rezone

a parcel, land purchase contracts under consideration by the developer should include zoning contingency clauses such that if approval is not granted by the governing body, the developer is released from the obligations of the contract.

Because the seller wants to both obtain maximum price for his land and sell the land in a reasonable amount of time, the study period specified in the contract may only be on the order of months. The buyer may be able to negotiate a longer time period within which to conduct the analysis. Normally, however, the buyer must be willing to compensate the seller for extraordinary periods of time. A higher purchase offering may be needed to extend the buyer's purchase option period. Because of the financial risk, the client needs accurate information in short period of time. The decision to exercise a purchase option will be based on the information in the analysis/assessment study.

Most experienced developers will know the lowest value of a cost per unit that renders the project uneconomical. In practice, the developer may study several alternative uses based on the information compiled during the assessment period. This helps determine what uses are economically feasible or whether the land development project can be profitable. Yield studies may also be performed in conjunction with each feasibility study. Depending on the skill of the people involved, a poor site may be made profitable. In some heavily developed areas, there may be few options among alternative tracts of land. Land often was bypassed during earlier stages of a community's urbanization because it had characteristics that made it less suitable for development. However, as years passed, surrounding properties and areas more distant from urban centers were developed. Many of the bypassed sites have increased in value because of their location and the presence of public infrastructure. Such sites may now be candidates for development if the higher development costs associated with environmental features can be compensated for by greater yield and prices now prevalent in its market.

### Scope and Process of the Study

The study must be well organized and usually supported by maps, photographs, and other graphics. It is often, but not always, presented in report form, although annotated base and topographic maps may suffice, depending on the complexity of the proposal. Any biases or opinions by the author should be included only at the request of the client and stated as such. The document will serve as evidence should any discrepancies arise or if lawsuits are filed as a result of claims for incorrect information.

The intent of the study is to identify development constraints along with options available to minimize those constraints. The study does not always specify preferred solutions. Those will be addressed later, if and when the developer considers the site to be suitable for development. Some site constraints, when recognized early in the process, can be accommodated by the plans and used to the advan-

<sup>1</sup> An example of an engineering feasibility study report appears at the end of this chapter. The reader is referred to this report to get an idea of the types of graphics and information discussed throughout this chapter.

tage of the site and developer. The consultant can devise ways to modify the constraints into amenities or to some other advantage.

In one project, a wetland area was initially perceived as a major constraint. However, the constraint was turned into an opportunity by incorporating the wetland area as part of a golf course where footbridges were used to navigate around the water hazard. Whereas the reviewing agencies were initially opposed to most developments that impinged on the wetland area, incorporating it as an amenity helped to convince the agencies to approve the overall project.

The study approach varies with the developer's intentions and preconceptions and circumstances related to the property. If the developer knows exactly what land use will be constructed on the tract of land and the zoning is compatible with that use, the feasibility study will analyze the site in accordance with that use and zoning. If the use is uncertain, the study will identify land use options based on the potential of the land. The potential of the land considers the ultimate density in the context of the property's existing and master-planned zoning, the existing infrastructure, and public facilities. The developer may wish to consider several options; the consultant may need to perform several feasibility studies if there is a significant variation in land use permitted by the possible zoning.

The results of the study will provide information to the developer as an aid in deciding the most appropriate course of action. It will also be used to determine whether the site warrants further analysis. If the problems associated with the site appear too difficult and expensive, the project will be abandoned or the price of the land will be adjusted. Frequently, the client will require a cost estimate for the construction of certain items in the study, such as utilities and road improvements. Additionally, unusual or extraordinary costs will be identified and estimated. This will help in assessing the economic evaluation of the site. While the study is not intended to offer solutions to site problems, the feasibility study will present the problems to be expected if the site is developed. A properly executed and documented study is an invaluable resource in preparing for subsequent phases. Items identified in the feasibility study will help in the layout of the site, including roads, developable areas, and open space areas. Off-site requirements must also be considered, such as easements or utility extensions that will affect site development. The decision to proceed will be based on information revealed in the engineering feasibility report.

### Required Information

The engineering feasibility study requires a comprehensive collection of all information that could affect the site and its development. This includes information about its legal condition, such as easements, land rights and other property encumbrances, its physical condition, such as topography, soils, utilities, drainage, external influences created by neighboring properties and uses, and regulatory con-

cerns. Such items have a significant impact on how the land is developed and how successful the project will be. For purposes of this section, the research and analysis associated with an engineering feasibility study are categorized into three types of information:

1. Legal condition of the site
2. Physical condition of the site
3. Regulatory concerns of the site

**Legal.** Title investigation must be performed by an attorney or land title insurance company for the developer to ensure that the landowner holds a fee estate in the subject property. The following title and other property information must be reviewed in order to determine constraints of the site:

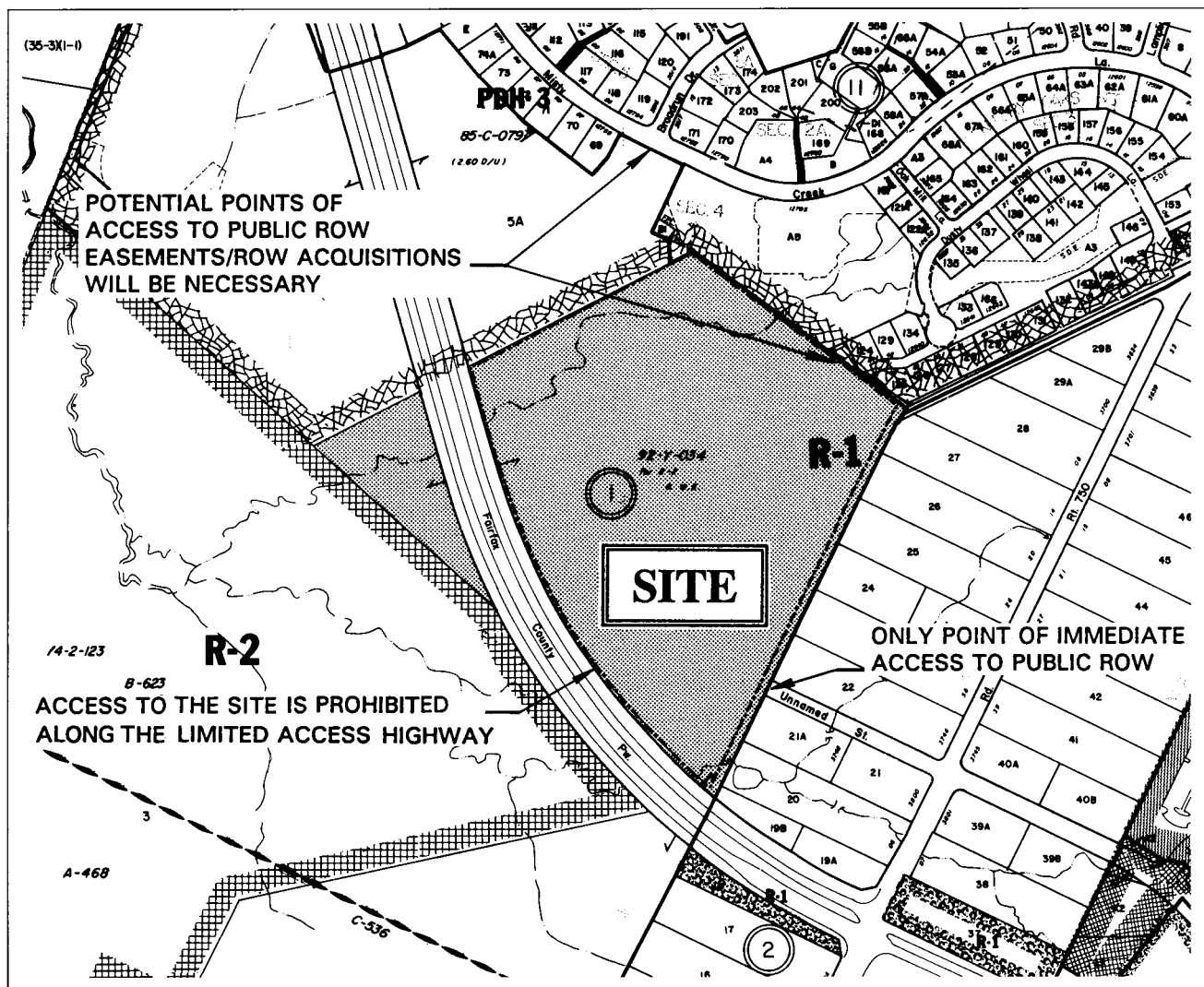
- Land ownership records, including property description (using metes and bounds or government survey method). *Do these match the scope of the project? Will land be excluded from the project and do subdivision regulations allow such an exclusion? Is the property contiguous or are there portions of the property separated by rights-of-way or other properties?*
- Chain of title traced back to creation of tract boundaries or adoption of local subdivision regulations, whichever is earlier. A title company usually performs a title search to ensure, and ultimately insure, that the chain of title has not been broken. However, the purpose of the land design team's analysis is to determine the applicability of local development regulations. *Has the subject parcel been legally subdivided from its parent tract (that is, with government approval if such approval was required at the time the parent tract was split)?*
- Deed conditions, restrictions, or covenants that could affect future use and enjoyment of the property. *Is the proposed use prohibited by deed? What private deed restrictions are imposed on the final land use, such as lot size, setback from property boundaries, architectural style, or building material? Is any portion of the tract, such as a lot around an existing dwelling, to be reserved for the existing owner?*
- Prior recorded plats, including government takings, boundary adjustments, and subdivision plats. *If there is an existing subdivision plat of record, what is its status? Can it be used to advantage; can the lots be developed and sold? What are developer's responsibilities concerning plat-tered public improvements, such as streets and storm drainage? What other requirements would apply? Identify local procedures for street or plat vacation.*
- Records of easements appurtenant (usually providing access to or through the property or adjoining properties) and in gross (usually energy or communica-

tions transmission lines), showing purpose and holder of the easement. *What rights are accorded the holder and what limitations are placed on the developer's use of the easement? Can easements be abandoned or relocated?*

**Physical.** The research and review of the site physical attributes and constraints include the site location, access, topography, drainage, vegetation, soils, and utilities. The following information must be assembled and reviewed to determine the physical constraints and opportunities of the site:

- Configuration and area from tax maps and tax records. *Is this mapping, site area, and ownership information consistent with the title information?*
- Existing structures, paved areas, fences, walls
- Adjacent properties, including information on ownership zoning, land uses, and their proximity to property boundaries

- Encroachments from structures on adjoining properties
- Presence of landlocked parcels and other properties adjoining the subject property. These may require extensions of roads and utilities as part of a development plan (see Figure 6.1).
- Public road frontage and property access information. *Who is responsible for maintenance and repairs of frontage and access roads? Is the property's frontage on a public road sufficient for gaining proper access to the future development? If there is no frontage, are access rights of way to public roads of sufficiently short length and adequate width to accommodate local street requirements. These standards include width, grade, drainage, and maximum cul-de-sac length. Is their sufficient room for construction equipment to maneuver? If not, can additional land or access be acquired? Will there be adequate sight distance at proposed entrances, as well as vertical and hor-*



**FIGURE 6.1** Landlocked parcel, due to property lines, that could necessitate extension of public infrastructure by the developer.

izontal road curves? Is there sufficient frontage to provide proper spacing between road entrances?

- Existing roads, both adjacent and across from subject property, and road conditions. Include right-of-way width, pavement width, site distance at hills and curves, sidewalks, curb and gutter, and drainage swale information. Indicate conditions that may hamper flexibility in site design. For example, requirements for minimum spacing between intersections on the abutting roadway may limit potential entry points to the development. Large trees at the edge of pavement may prevent widening or draw public opposition. *Will any existing streets or rights of way require abandonment or vacation?*

- Highway noise influences:

- *Will existing and proposed highways in the vicinity of the site generate highway noise that should be mitigated?*

- Site topography:

- In performing a topographic analysis, identify streams, swales, ridges, and similar landforms and features. Identify steep slopes, where grading may be expensive or prohibited (greater than 15% and 25%), excessively flat areas (less than 2%) where storm and sanitary sewer drainage may be difficult or expensive to achieve. Show incremental breakdown of intermediate slopes to assist in plan layout. (See report at end of chapter for topographic analysis.)

- Storm drainage:

- Drainage basin and watershed within which the property is located; unique restrictions or conditions applicable to development
- Description of on-site drainage patterns
- Location or plans for regional stormwater management facilities; timing of public improvements
- Existing floodplains from local jurisdiction, United States Geological Survey (USGS)/Federal Emergency Management Agency (FEMA) reports
- Potential for flood plain when stream is present. Small streams and swales not flowing continuously throughout the year may still need to be analyzed for their flood capacity.
- Probable locations and sizes of culvert and outfall improvements due to increased runoff from development
- Downstream problems with drainage; known complaints

- Requirements for detention and retention ponds for stormwater management

- Adequate outfall (easements) and ownership of existing structures

- Location, size, depth, and condition of existing pipes

- Requirements for non-point pollution control and best management practices (BMPs)

- Overland relief constraints from downstream properties and potential overland relief constraints of the subject site to upstream properties

- Location of wetlands and other sensitive environmental areas (from National Wetlands Inventory maps) or other mapping

- Vegetation:

- The location of large trees, areas of tree cover, including a review of the quality and type of existing trees, should be determined.

- Soils information, including types and characteristics:

- This information may include bearing strength, stability, shrink/swell potential, perched water table, presence of naturally occurring asbestos, radon potential, suitability with regard to building foundations, culverts, utility trenching, and erodibility. *Consider soil percolation characteristics; do soils indicate the need for extensive earth movement and placement of engineered foundations on compacted fills?* Where soils of questionable suitability are identified, the consultant should recommend that further investigation and testing be performed by a geologist or other expert.

- Sanitary sewer information, including:

- Agency with ownership and approval authority

- Sewershed in which the property lies; available capacity, projected demand, local restrictions concerning sewer allocation

- Location, size, depth of, and distance to existing lines

- The age and condition of existing lines in order to evaluate whether current materials are compatible at connection points and whether the type of pipe will be structurally adequate for proposed uses over the line

- If not on-site or adjacent, the distance to appropriate connections, means of access, need for easements

- Responsibility for extension and improvements and current timing of public improvements; potential for reimbursement from public or private funds, such as other developers
  - Gravity vs. pumped vs. on-site package treatment plant
  - Interference of system construction with other utilities/features
  - Pro-rate shares/assessment fees
  - The age and condition of existing lines in order to evaluate whether current materials are compatible at connection points and whether the type of pipe will be structurally adequate for proposed uses over the line
  - On-site disposal issues, including treatment method, soil suitability, drainfield and lot size restrictions, impact on project density/lot size, comprehensive plan and ordinance considerations, depth to water table. Spot percolation tests may be required.
- Water distribution for service and fire flows, including:
- Agency with ownership and approval authority
  - Size, location, depth of, and distance to existing water mains, means of access and need for easements. *Are off-site easements required to extend service?*
  - Water quality, quantity, pressure, and necessary corrective measures.
  - Responsibility for extensions and improvements, associated fees; timing of public improvements
  - Requirements for fire hydrants; water supply and distribution requirements for fire flow
  - The age and condition of existing lines in order to evaluate whether current materials are compatible at connection points and whether the type of pipe will be structurally adequate for proposed uses over the line
  - On-site well information, including depth to water table, groundwater quantity, water quality, plan, impact on project density, proximity to dwellings and septic systems, and other ordinance requirements. Test wells may be required.
- Information concerning other public or private utilities and services that are essential components of the development:
- Service options for energy and communications utilities, such as electric, gas (natural or liquid propane), cable, telephone, and fiberoptic. *Are there competing companies serving the same area? If so, are rate structures and builder incentives comparable?*
  - Current and projected levels of service. *Are improvements budgeted, is timing compatible with project? Can timing be advanced?*
  - Responsibility for design, upgrade, connection
  - Consideration for required easements. *Who will obtain?*
  - *Connection fees; when must they be paid?*
  - Electric, telephone, and cable undergrounding requirements, on-site, adjacent, and off-site responsibilities and contributions
  - Information regarding the provision of trash removal and street disposal, street cleaning, snow plowing, and similar services. *Are services public or private; are there alternative providers? If trash removal is public, is it available to condominiums and commercial operations? What is the availability of private contractors for these services?*
  - Location, proximity, and planned improvements of elementary and secondary schools, means of access (pedestrian, school bus, and public transit). *Do sidewalks or trails exist between site and schools; will interior sidewalks be required in the development? Are there school impact fees?*
  - Requirements and responsibility for installation and maintenance of streetlights
- Other aspects of the site such as the following should be investigated:
- Availability, proximity, and planned improvements of emergency services, such as police, fire, and rescue
  - Aircraft flight patterns and noise contours. (See section on Noise Abatement in Chapter 5, Environmental Considerations for aircraft noise contour map.)
  - Unusual on-site and adjacent features, such as cemeteries, railroads, and historic properties
  - Earthquake potential, prevailing weather patterns, and solar exposure that could affect project design
  - Research into previous uses, needed to determine possible underground structures or contaminating conditions.
- Regulatory.** Regulatory information must be included in an engineering feasibility analysis in order to identify the appropriate processes to allow the proposed development.



These processes will influence the timing, cost, and extent of community and public involvement in the development process. The following aspects of an engineering feasibility study will aid the developer in the assessment of the regulatory aspects of a proposed project.

■ Planning, zoning and related development information:

- Relevant comprehensive plan and zoning maps and texts. These include growth management ordinances, such as adequate public facilities ordinances, impact fees, and other construction limitations, such as annual building permit caps. Include information on miscellaneous fees, such as filing and processing, recreation, drainage, etc. *When must fees be paid, improvements installed?*
- Current property zoning, proposed zoning, including uses permitted by right and those requiring special exception. Describe the purpose of the zoning district. *Do zoning boundaries divide the subject property? What is the relationship of zoning district boundaries to the subject property?*
- Requirements for zoning overlay or special districts. Examples of such districts include highway corridor, historic, transit area, central business district, transfer of development rights, resource protection or conservation areas.
- Comprehensive plan recommendations peculiar to the site, e.g., density or allowable land use. *Does the current comprehensive plan show any roadway improvements that will impact the site?*
- Pertinent requirements of the zoning ordinance. These include maximum density or floor area ratio, minimum or average lot size, setbacks (from project boundaries, property lines, lot lines, rights of way, railroads, highways, waterfront, etc.), building height and bulk requirements, maximum lot coverage and open space requirements, off-street parking and loading, screening buffering and landscaping requirements. *Is right of way of future roadways used for density credit?*
- Proposed or pending changes in comprehensive plans or development regulations likely to be adopted within the project's life span. *What significant impacts will these changes on the property's use or yield? Will the change increase the time it takes to secure construction authorization or expose the project to additional public hearings or government agencies' review?*
- Grandfathered or vested rights that may be jeopardized by the proposed development. *Will the project be exposed to requirements far beyond those needed for the additional construction or expanded use?*

- Development history of the property, including records of previous submissions for rezoning, special exception, subdivision, or building permits
- The likelihood of citizen opposition and delays associated with the development process. The experience of the land design team is particularly valuable here. Familiarity with local issues and past encounters with civic groups can help the development team prepare for future conflicts.
- Subdivision and other ordinance regulations peculiar to the site, such as environmental protection and dedication, tree preservation, or planting. *Can private streets be utilized? Will there be any assessment by any local or state agency for road construction? Are there requirements for adjoining property owners to contribute to the cost of constructing streets to service their properties?*

The land design team prepares base maps of the subject property in a level of detail and accuracy commensurate with the time and budget. The most common map scale to use is 1" = 100' or 200', although for smaller properties 1" = 50' may be more suitable. The selected scale should be suitable to show sufficient detail. For many projects the 1" = 100' to 1" = 200' scale range is adequate to show the detail and yet limit the number of drawing sheets to only one or two. The base map is prepared on topographic maps and shows property boundaries with dimensions and intersecting property lines from adjoining parcels. Adjoining roadways and scaled locations of all other physical features identified in the information survey are also shown. These will be verified and information expanded in the next step of the assessment study, the site visit.

Based on the information just discussed, a checklist such as that shown in Figure 4.2 can be created to facilitate the feasible study.

### Sources of Information

The land design team must become familiar with information resources and the local sources of that information. These simplify the investigation of property conditions and local regulations and eliminate much of the need for original research and testing. Throughout the study phase, sources must be documented with particular care, whether the source is available public documents or conversations with public officials. This is especially important in preparing the final report document, where the consultant's opinion must be separated from others' opinions or established facts.

Of particular value are existing public records. These include published tax maps of the community, which depict property boundaries, land area, and landowners, along with references to recorded subdivision plats and deeds. An office of land records, court clerk, or similar agency maintains

PROJECT NAME: \_\_\_\_\_ DATE: \_\_\_\_\_

TAX MAP REF: \_\_\_\_\_ DATE DUE: \_\_\_\_\_

I. Administrative Checklist

A. Check files to determine existence of current or previous studies for subject or nearby sites.

B. Obtain pertinent plans for existing or proposed developments in vicinity of site. This also applies to proposed public improvement projects which may affect subject site's development.

C. Site Visit

1. Vegetation, tree coverage, and any unusual conditions found on site.
2. Onsite streams and adequacy of outfall channels both onsite and offsite.
3. Note culverts (type and size) along outfall.
4. Note existing utilities onsite and adjacent to site.
5. General topography and soil conditions, including rock outcrops.
6. Check site distance of road frontage.
7. Note existing structures onsite and proximate to the site.
8. Note size and condition of roadways.
9. Note indicators of potential wetland areas.
10. Note indicators of environmental contamination.
11. Note current construction activity on or near the site.

II. Written Feasibility Study Checklist

A. Legal

1. Land ownership records (tax records).
2. Title information and easements.
3. Deed conditions and covenants.
4. Subdivision history.

B. Physical

1. Location, type, and condition of existing structures, paved areas, fences, and walls.
2. Adjacent property ownership, land use, encroachments, access and other potential impacts.
3. Public road frontage, sight distance, access constraints, noise impacts.

**FIGURE 6.2** Checklist for acquisition/feasibility study.

copies of deeds, subdivision plats, and similar records relating to property ownership.

The local planning, public works, building, or transportation department often maintains aerial photos of the community taken at various intervals. These provide both historic reference and indications of recent or current use. These photographs often are printed in conjunction with

tax maps of the community. Many municipalities have incorporated geographic information systems (GIS). Consequently, property and infrastructure information may be accessible electronically with the proper (computer hardware) software systems.

Some communities provide topographic maps of the community based on aerial photography. Otherwise, these

4. Site topography, steep slopes, flat areas.
  5. Drainage basin and watershed impacts. Onsite drainage patterns, existing water bodies, floodplains, outfall conditions, downstream drainage problems, stormwater management requirements, overland relief constraints, potential wetland areas.
  6. Location, type, and condition of vegetation.
  7. Soil information from existing records and other pertinent studies.
  8. Sanitary sewer information including sewershed, location and size of existing lines, condition of existing facilities, capacity of existing facilities, extent of improvements necessary to connect to existing facilities, potential conflicts with other utilities, pro rata assessments, onsite disposal options.
  9. Water information including size & location of existing lines, pressure in existing lines, fire flow requirements, condition of existing lines, extent of improvements necessary to connect to existing facilities, potential conflicts with other utilities, pro rata assessments, onsite water supply options.
  10. Natural gas, electric, telephone, fiberoptic review including franchise, availability, and proximity to site.
  11. Other impacts such as trash removal, location of schools, street light requirements, emergency service proximity, aircraft noise impacts, onsite cemeteries, historic aspects, endangered species, prevailing winds and solar exposure, potential environmental contamination.
- Regulatory
1. Zoning requirements.
  2. Master plan/comprehensive plan recommendations.
  3. Proposed or pending changes to the ordinances.
  4. Grandfathering or vesting rights.
  5. Development history of the site.
  6. Local development issues and concerns.
  7. Subdivision regulations.
  8. Development process and timing.

**FIGURE 6.2** (Continued) Checklist for acquisition/feasibility study.

are available from private sources or can be commissioned for each project. In addition, the USGS quadrangle maps are useful for identifying site topography, natural and man-made features, perennial and intermittent streams, and other items of interest to the consultant. In many communities, the local soil conservation district or state natural resources agency publishes soil maps and related information.

Recent building and development plans, permits, and application materials often are kept on file in various agencies, either in their original files with all supporting documents or on microfilm or similar medium. Most are available for public inspection. These provide records of previous studies that might apply to the subject property. Along with public records of construction plans and as-built documents for public facilities, these records are useful for information relating to underground utilities. These files often provide useful information concerning the experiences of previous developers. The development consultant's own

in-house records of its previous and ongoing projects near the development may contain recent studies or more current information than available from other sources.

Copies of local plans, regulations, and ordinances are available from these agencies. The community's comprehensive plan may list local, state, and federal sources for information upon which the plan is based. The transportation agency may have recent studies of existing traffic counts, capacities, and level of service for the road network near the project.

A local historic preservation agency or society may have compiled a register of historic properties or sites, including archeological information. The local economic development authority or chamber of commerce provides useful market area information. The local utility companies will provide distribution maps of the service areas.

Many federal agencies provide maps of various utility to the consultant team. In addition to USGS and FEMA, these include U.S. Fish and Wildlife Service, Army Corps of En-

gineers, Environmental Protection Agency, and Soil Conservation Service.

The level of detail of the study depends on client requirements. Typically, a preliminary yield study will accompany the feasibility study in order to obtain an idea of the density that can be expected. The level of detail available on existing maps and records usually is sufficient for feasibility studies. These maps are generally available in a wide variety of scales, often ranging from 1" = 100' to 1" = 2000'. Contour maps with contour interval of 5 to 10 ft are readily available and acceptable, except perhaps in very flat areas, where a closer interval may be desirable.

### Site Inspection

One of the most important elements of the feasibility study is the site inspection. Much of the information contained in printed maps often is outdated, reflecting only property conditions at a specific date in the past. Physical conditions change constantly through the action of both man and nature, and maps may not accurately represent the actual field conditions. In addition, contours and other information normally shown on aerial topographic maps may be unclear. The ground may have been obscured by foliage or snow cover, depending on the season of the year, time of day, or other conditions at the time that the area was flown. A site shown as wooded may in fact have subsequently been cleared and graded. Illegal dumping or similar activity may have taken place. Adjacent properties may have undergone development since the maps were last produced. Certain features, such as wetlands and small streams, may not readily be evident. These often require field analysis to verify their location.

For these reasons, it is imperative that a field inspection be performed as part of the engineering feasibility study. The visit is necessary both to verify and to build upon information collected elsewhere. The person visiting the site is looking for obvious contradictions with recorded information, as well as evidence that implies conditions not previously known. Of particular importance are conditions that may render the land unusable or impose extreme costs in the development.

Prior to visiting the site, the field personnel assemble and prepare information in advance. A base map should be prepared as a composite of the information previously discussed, at a scale that can be conveniently used in the field. A clipboard, writing paper, tape measures, scales, waterproof pens and markers, and plastic bags for collecting specimens should be among the equipment brought to the sight. Field binoculars may also be useful; a camera, possibly a video camera, and tape recorder are particularly useful for recording observations.

It is imperative that the field visit team make arrangements with the developer, the owner, and any residents of the site prior to the site visit. Agents of the developer may not necessarily have right of entry if it is not spelled out in the purchase contract. Unexpected visits may be unwelcome by occupants, who may not even be aware that the

property is being sold. Ask that dogs and other animals on the site be penned, if possible, as a protective measure.

The consultant should be appropriately dressed for the visit, anticipating the weather, brambles and dense brush, mud, standing water, poison ivy, insects, snakes, and other wildlife. One of the purposes of the field inspection is to determine the site's response to rain and runoff. Therefore, it is advisable to schedule at least one visit to the site soon after storms or spring thaws. The field team can observe ponding, running water, and other surface conditions that otherwise may not be visible.

Upon arriving at the site, the consultant should drive the boundaries of the site, noting landmarks such as fences, hills, swales, and curves in the roadway that have previously been recorded on the base map. These will serve as points of reference while walking the site and make it easier to record information about the visit. A systematic walking tour of the site should be planned, taking care to include critical natural and man-made features that were previously noted in the office review of site information.

On the map, note and verify those areas that are important to the development of the site, as either problems or opportunities. Outline the apparent boundaries between different topographical and geological conditions, such as between improved and unimproved areas, stable and unstable slope areas, wetlands and dry ground, wooded areas and open fields. Visualize property boundaries, particularly in locations where topography or other circumstances appear to create difficulties during development. Depending on the relationship between the site boundaries and topography, construction in these areas may necessitate the acquisition of easements for drainage or equipment access across adjoining properties. Retaining walls may be necessary at property boundaries if significant grade changes are required.

Areas of interest that should be recorded on base maps during the field reconnaissance are described below:

- Streams, swales, washes, and evidence of confined running water and intermittent streams, such as unusual patterns of fallen leaves, vegetation and stones, soil erosion, uprooted or undercut trees, and areas cleared of leaves
- Ridges and obvious drainage divides with nearby running or ponding water at lower elevations, indicating high water table, springs, and springheads
- Floodplains, often evidenced by high water marks on shrubs, tree trunks, and low-hanging branches
- Ponds, lakes, and other impoundments, again, trying to identify the limit of impoundment
- Evidence of pollution or sedimentation in running and standing water, from on-site or off-site uses
- Condition of stream valleys, banks, and shorelines
- Marshes, swamps, wetlands, bogs, wet and soggy areas, noting types of vegetation, areas of matted

leaves, or unusual soil coloration that may suggest frequent or periodic inundation

- Areas and types of vegetation, boundaries of wooded areas, and stands of trees that might serve as buffers against adjoining properties, protected species, large trees and other specimen trees or mature ornamental landscape materials that may be preserved, either in place or transplanted for subsequent reuse in the development
- Presence of fish and wildlife and evidence of animal habitats, such as beaver dams and eagle aeries that must be considered and protected or that may even preclude development of the property
- Cliffs and other unusual landforms indigenous to certain areas of the country, such as coastal and Great Lakes dunes, sinkholes
- Areas of steep slopes, noting vegetative cover
- Evidence and sources of erosion and slope instability, such as leaning trees, poles, fences, broken pavement at top of slope, softness at toe of slope, sharp vertical drops suggesting landslides and other signs of previous slope movement, exposed soil colors
- Locations where, based on visual inspection and soil map data, additional subsurface explorations (e.g., auger boring, test pits) will be necessary
- Evidence of strong prevailing winds, such as distorted plant and tree growth
- Existing pilings and retaining walls
- On waterfront property, piers, moorings, and other marine uses, access points to the edge of water. Observe conditions and uses of adjoining and opposing shorelines, maritime activity, information concerning water quality, depth, and bottom configuration.
- Rock outcroppings, which may create problems in site excavation for roads, utility trenching, well and septic system suitability and foundations. Consider their possible use as aesthetic features and points of interest to enhance the development's market appeal. It may be possible to stockpile stone and rock for subsequent use in erosion and sediment control or for landscape material.
- Condition, size, and location of culverts, outfall channels, and any existing drainage pipes and swales
- Location, use, and structural condition of buildings, paved areas, abandoned wells and other man-made features on the site, whether or not they are to be preserved; evidence of flood damage, earth settlement and movement in walls and foundations
- Evidence of cemeteries, gravesites, burial grounds, archaeological and historic sites, battlefields, old found-

ations, and other unusual or unexpected existing or prior land uses on-site that could limit development potential, incite community opposition, or delay project approval

- Evidence of trespass and community use of the property, such as footpaths, dirt bike trails, picnic areas, and sports fields, which may be an indication of potential community opposition to the development
- Interesting views within and from the site, areas that might be cleared to enhance views, and views from adjoining properties onto and over the site
- Character, condition, and use of adjacent property and proximity of neighbors and site improvements. Record evidence of access easements, encroachment of fences and structures.
- Current construction activities on or near the site
- Evidence of noise, smoke, dust, odors, light intrusion, or other activities from sources within the site or nearby uses, such as from nearby industry, highways, railroad crossings, racetracks, hospitals, fire and rescue stations, schools, commercial areas, airports, landfills, sewer lines or sewage treatment plants. These could affect the site's value or market appeal. Prevailing winds should be considered in evaluating their impact. Noise walls, landscape barriers, or special construction techniques and materials may be employed to mitigate potential nuisance. Evidence of significant trash, debris, chemical or oil dumping, burial and storage. Evidence of unusual odors that may suggest natural decay, sensitive environmental features, or ground contamination.
- Sight distances at curves and hills adjacent to the property and probable entrances
- Traffic congestion on adjoining roads and nearby intersections
- Condition of surrounding roads and pavement, including paving and shoulder stability and widths, roadside swales, curbs and gutters, location of nearby and opposing driveways
- Locations of overhead utility and power transmission lines
- Manholes, standpipes, vent pipes, signs and other evidence of underground tanks, sewers, and transmission pipes. Isolated areas with poor vegetation in contrast to their surroundings may be indicative of subsurface materials.
- Possible locations of individual and clusters of buildings, internal roadways, and open-space areas, trying to visualize development of the site. Record existing and potential means of access to areas separated by deep swales and ravines.

Depending on size and scope of the project, several individuals may be required to walk the site. In addition, it

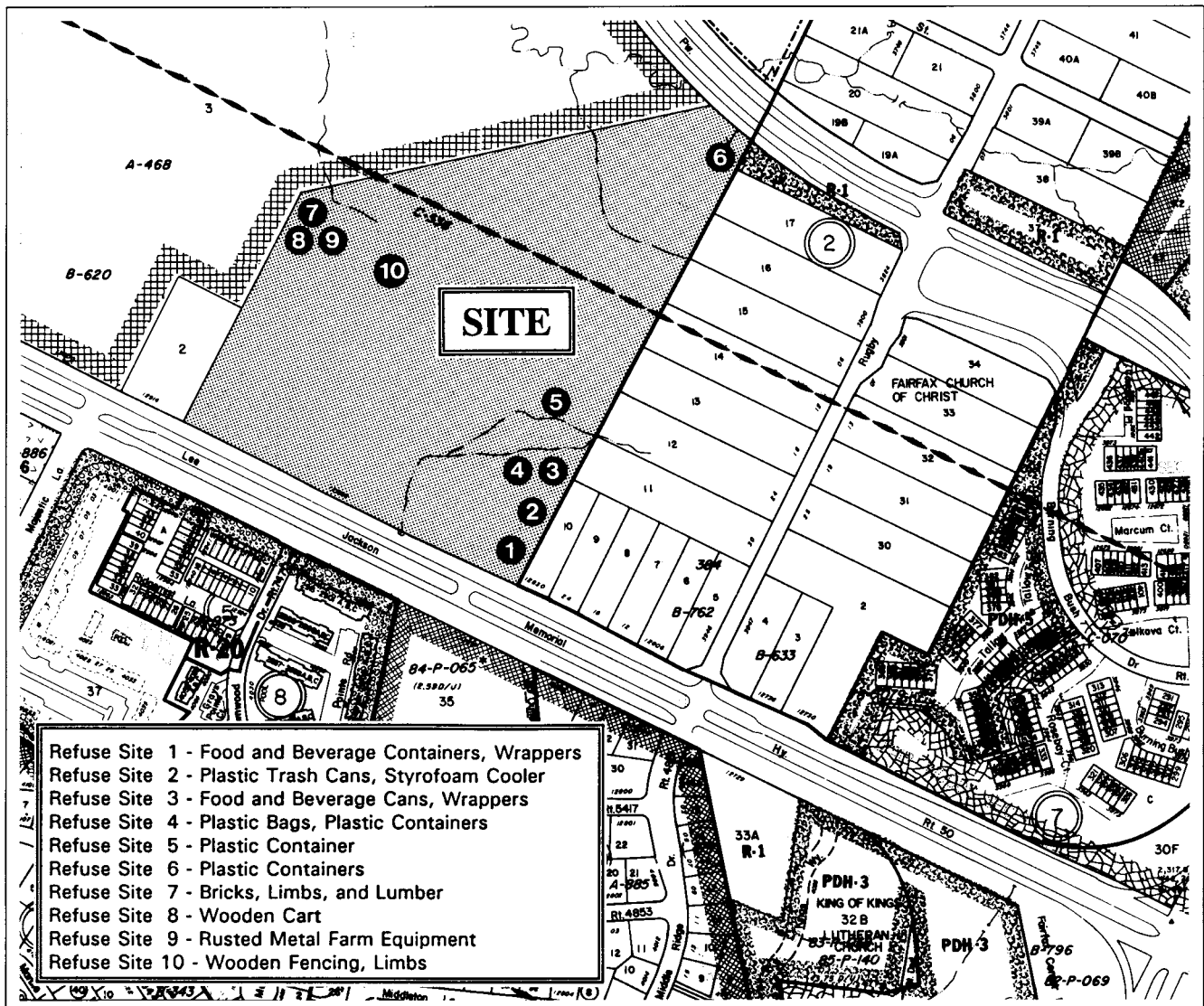
may be necessary to revisit the site with other professionals whose expertise is indicated by the findings or local requirements. Further analysis may be required where preliminary investigations show the presence of unusual soils or wetland areas. Additional site visits will be needed if the project moves beyond the feasibility study. Therefore, as thorough an inventory as possible at this phase will simplify later work. However, this must be balanced with efforts to control costs at this phase. These costs must be controlled because of the possibility of project abandonment.

During the site visit, extensive photographs should be taken. The location and direction of each photo should be noted on the base map for reference in the office. These should include photographs of important views and significant features. A series of panoramic shots taken from the property boundaries is useful for setting points of reference. Including people or other items of known size helps establish height, depth, and width of features being photo-

graphed. As in all photography, lighting and shadows are important to adding dimension. Photographs serve as valuable reminders when the site analysis is studied back in the office. In addition, photographs assist other members of the development team who were not present in the field. Videotaping the site visit also provides a useful reference, allowing for more interactive commentary about site features and provides a clear record of site conditions prior to development. This is useful for comparison to conditions during and after construction if legal or procedural conflicts arise. Figure 6.3 shows how site photographs can be referenced to the property.

### Site Analysis Mapping and Report

Data from the site visit must be compared to other recorded data. Any inconsistencies should be resolved to verify true field conditions. Of particular importance are discrepancies in property boundaries and topography. The consultant



**FIGURE 6.3** Site photographs cross-referenced with their locations on a map often help others assess potential constraints or liabilities.



a)



b)



c)

**FIGURE 6.3** (Continued) (a) Refuse site #5. (b) Refuse site #7. (c) Refuse site #9.

should transfer information to a clean base map, which will be used to report results of the study to the developer. In addition, the map becomes an important tool in performing yield studies and beginning the design process should the project advance to that phase.

The relationship to the site of existing roadways, utilities, and other facilities should be shown. Site constraints should be noted and labeled, using shading, heavy outline, and other techniques to identify areas of the site that are totally unusable or usable with significant corrections. These in-

clude floodplain areas, unstable or erodible soils or soils of poor bearing capacity, steep slopes, and wetlands and other environmentally sensitive areas that local state and federal regulations accord special treatment. Area measurements of these encumbrances are recorded both on the map and in tabular form in the report.

The map should indicate those facilities and improvements that will be necessary, because of function, industry practice or government regulation, to develop the site. These include probable vehicular access points, along with an assessment of improvements to public roads that will be needed to facilitate safe, convenient access. A consultant experienced with community policies will know whether site development will require turning lanes, acceleration or deceleration lanes, medians, or traffic control devices. Even if these are not required, the consultant should be familiar with expectations of consumers and community residents. The map should note dedications and reservations that the governing body will require in conjunction with development. Connections and required improvements to public utilities should also be indicated. The map should provide notations concerning site setback and buffer requirements along property boundaries.

Along with these site constraints, the map should reflect significant site opportunities. These should stress references to the recommendations and guidance of the community's comprehensive plan, an indication of the issues that the development program must resolve. The map should suggest potential amenities that will facilitate development and enhance project marketing. Important views, stands of trees at property boundaries, and similar characteristics should be highlighted.

Site data are also compiled in report form. The report must be well organized to present each category of information studied by the consultant. Sources of information should be clearly stated, including public officials who have been interviewed during the feasibility study. The consultant should attach copies of comprehensive plan map or zoning ordinance language that have unique and direct bearing on the project. The consultant should identify items for which administrative waivers appear appropriate and likely to be granted.

Limitations on the reliability and scope of the information should be noted. Any opinions of the consultant included in the report should be clearly identified as such. In addition, the report's findings should be qualified as being preliminary only, withholding any guarantees about potential problems that could not have been identified because of the scope or depth of study. The report will be used by the developer to determine future actions concerning the property.

The report should clearly identify qualifications to its conclusions. Sources and timeliness of information, limitations imposed by time and resources, and lack of applicability to subsequent projects should be firmly established. The report will likely serve as a reference document in further negotiations with the landowner and lender. If the de-

veloper concludes that development is feasible, the documentation is useful for those engaged to perform subsequent analysis and design. In addition, the report serves as documentation of the consultant's findings, useful in case of subsequent claims of error or omission.

### An Alternative Study Approach

Generally, it is not possible to conduct the depth and detail of analysis just described. Time constraints are frequently imposed by dynamics of the real estate marketplace and the competitive nature of the land development industry. Decisions to move ahead must be made quickly during short purchase option periods in order to take advantage of favorable land prices or other conditions. Monetary constraints are always a factor in land development, where financial gain is likely to be several years off and cost containment always an essential consideration. The studies described represent a considerable expenditure of funds, an up-front expense that may be incurred several times in the analysis of various sites and projects.

To provide the broadest range of services to the land developer, the land design team must learn to develop an abbreviated approach for a engineering feasibility study. Such an approach would typically include the essential factors that relate to a parcel of land. These could include:

- Identification of proper zoning or the ability to rezone
- Location and adequacy of utilities and essential services
- Access
- Topography, soils, environmental and sensitive features, defining the usable area
- Site visit

Throughout the analysis, the development team seeks to identify elements of concern, either physical or political, that suggest that the land is or is not unsuitable or that a project is or is not practical. The experienced consultant begins each new study with a prioritized list of items in the investigation. Through experience and frequent involvement with local land use processes and decision-makers, it becomes possible to determine early in each study when the investigation should be terminated. Each consultant arrives at a level of expertise, or comfort level, that allows relatively educated snap judgements to be made. Regardless of whether the developer then moves ahead with a particular project or abandons it, the analysis can be viewed as a justifiable expense.

### Yield Study

A yield study is often performed at this stage, upon request of the developer, to determine the probable intensity of development for the property. By knowing the total development potential that can be achieved on a site, the developer

can determine if per unit costs are within an acceptable range for local market conditions. The study will also be used as a guideline during the conceptual layout stage to determine if the land is being used to its greatest efficiency.

The simplest approach to a yield study is to apply maximum densities to the project area. This method first subtracts all uncountable land from the gross (total) land area, to arrive at the net site area suitable for density calculation. Note the distinction between uncountable and unbuildable land. The former accounts for acreage not eligible for density calculation as defined by the local zoning ordinance. The latter is land composed of certain physical characteristics that cannot be corrected and that make construction impossible or impractical. For instance, most local subdivision regulations provide that floodplain land may not be built upon; some require that it be dedicated to public use. However, zoning ordinances in many of these same communities may grant the developer density credit for a portion of the land in floodplain. The uncountable land therefore *may* comprise only a percentage of the acreage to be set aside. Again, depending on local ordinance, land required for abutting street right of way widening or land area with terrain in excess of certain gradients may be excluded as uncountable land in calculating the potential yield of given acreage. Unbuildable land may include portions of a site characterized by unsuitable soils, underlying bedrock conditions, or excessive slopes whereby simple construction economics warrant it as unbuildable yet the land may be eligible for density calculation from the standpoint of the local zoning ordinance.

For example, assuming a permitted density of four units per acre, a 10-acre property with 1 acre in required road right of way widening and 2.5 acres of floodplain may be projected to yield 26 lots in a jurisdiction that precludes right of way and floodplain from density calculations. In this case, the uncountable area comprises 3.5 acres of the 10-acre parcel. A like-size parcel with the same uncountable land area but also containing 3 acres of significant rock outcrops and slopes in excess of 30% (i.e., unbuildable area) while technically entitled to the same 26 lots, would, on average, require smaller lots or a different unit type to achieve a comparable yield.

In the case of residential land, the resulting net area is multiplied by the density allowed in the regulations for the proposed zoning district. (For nonresidential land, total building area is derived by multiplying the countable area by the floor area ratio permitted in the district.)

The number of units derived by this method usually establishes the high end of the range for probable yield. It does not account for open space, subdivision streets, storm water management facilities, or other components of a development that occupy land to the exclusion of other uses. Other factors affect the achievement of full development potential. For instance, this approach does not reflect the practical difficulties of accommodating units precluded from unbuildable land. Characteristics of parcel size, shape, and topography also limit the number of lots that can be



achieved. Small, irregularly shaped parcels are difficult to develop efficiently; hilly terrain also limits opportunities to maximize development potential without extensive, and expensive, clearing and grading.

A more accurate reflection of residential site yield results from determining total buildable area of a tract. This calculation is then adjusted to account for land that will be used for streets and other public dedication or lost to inherent inefficiencies in layout. An adjustment factor of 20% is reasonable. The resulting area is then divided by the lot size, either allowed in the zoning district (which may be specified as a minimum or minimum average) or required by the builder (in order to accommodate the product), to arrive at probable yield. Table 6.1 is useful as a rule of thumb for estimating site yield per acre for a single family detached development.

For greatest accuracy, and for non-single-family projects, a series of trial sketches prepared on an overlay of the base map is often the best approach to a yield study. This is especially important where cluster development is possible or desirable (cluster allows for reductions in lot size while maintaining the density permitted by the property's zoning). Quick calculation and rule-of-thumb methods assume that a conventional subdivision layout is proposed. If the local subdivision (or zoning) regulations have provisions for cluster development, a trial sketch is needed to estimate project yield. With cluster methods greater flexibility in street and lot layout can be achieved. This technique encourages the developer to avoid and preserve sensitive and other unbuildable areas while acknowledging that those areas are a

part of the parcel and project. With cluster provisions of the zoning or subdivision regulations, the site layout can often approach full yield. Cluster allows for the recapture of units that otherwise may be lost due to unbuildable land or unusual configuration.

Trial layouts are time-consuming, and the time available during the developer's feasibility option phase may be insufficient. Described fully in Chapter 7, preparation is an iterative process. However, by considering a site's physical, locational, economic and regulatory characteristics, trial layouts provide the developer with the most realistic assessment of a parcel's value and potential. The opportunity to consider several layouts offers an early indication of the type and form of development possible. When coupled with an early estimation of potential development costs and evaluation of the likelihood of project approval, inclusion of trial layouts in the feasibility study is an important tool.

Estimating project yield by any method requires skills and confidence, which evolve only through experience. Without that experience, both with project design and with local regulatory involvement, unrealistic estimations can be misleading to the developer. False estimations at either extreme can result in financial loss or lost opportunity. Therefore, the consultant must avoid the temptation to use shortcuts and conjecture. If the yield study is not within the scope of services agreement with the developer, one should not be performed. In addition, site yield varies with the ability of the development team, resources of the developer, and market demand for a product to accommodate difficult site conditions. Thus, any yield study that is performed should be reflected as a range of units, specifying conditions that must be overcome during project implementation.

Because of the important role that construction costs play in project feasibility, and the impact of environment and regulation on those costs, a land development consultant should have a general understanding of construction economics. Consideration must weigh the final price of units against the cost of acquisition, engineering, and construction. There are always several alternatives for developing the site, and certain tradeoffs will be available. The assessment of these tradeoffs requires knowledge of relative costs. In addition, certain characteristics of development projects affect market absorption, the rate at which units are sold or leased, and the period within which the developer recoups the investment. The developer will base the decision to continue or abandon the project on an estimation of costs to correct site problems. The cost analysis performed at this time will be general because of the nature of the information researched. However, it should be sufficiently detailed to enable the developer to make a well-educated decision.

The developer anticipates a certain intensity of development in order to recover costs and earn a profit. These costs include land acquisition, project engineering, hard construction costs for public facilities and the building

**TABLE 6.1 Estimating Single-Family Site Yield**

<b>CONVENTIONAL SUBDIVISION LAYOUT</b>	
<b>AVERAGE LOT SIZE (FT<sup>2</sup>)</b>	<b>AVERAGE YIELD (LOTS PER BUILDABLE ACRE)</b>
6,000 to 8,000	4.5
8,000 to 10,000	3.5
10,000 to 12,500	3.0
12,500 to 15,000	2.7
15,000 to 20,000	2.3
20,000 to 25,000	1.5
25,000 to 30,000	1.2
30,000 to 40,000	0.9
40,000 to 50,000	0.7

Reproduced with permission from J. Dechiara, and L. Koppelman, *Site Planning Standards*, McGraw-Hill, New York, 1978.

product, fees, and interest-carrying charges for the land throughout the project. Using proprietary formulas that correlate investment, demand, preference, and return, the developer can evaluate finished costs, project unit size and price range in order to recover these costs. If the projected selling price needed to recover cost is out of line with either market demand or affordability, then reevaluation of the project is needed.

If the developer concludes that the project is economically viable, he may elect to proceed with the development application process. The information assembled in the feasibility study is an important foundation upon which to build subsequent investigation. With issues identified for further analysis, it enables the consultant to assemble a team of development specialists whose expertise is specifically geared to solving problems essential to project design. This helps in the preparation of the project work program and budget, on which are based estimation of manpower and resources needed, the type of development proposed, issues that must be resolved, and time needed for approval.

### Site Selection Study

Occasionally there may be several sites available with the potential to fit the development program. In such cases, some process needs to be used to compare and contrast the physical and regulatory factors that affect the development program for each of the properties. This process, a site selection study, determines which property, if any, is best suited to accommodate the client's development program. In order to present the information in a fashion that allows the client to evaluate different sites in an unbiased manner, a standardized narrative and graphic format needs to be developed, with data compiled consistently for each site. The site selection study involves an engineering feasibility study for each property. However, the information contained in the site selection study must be obtained in a consistent manner in order to compare the properties equally.

To determine the development opportunities and constraints associated with each property, the following site-related issues need to be reviewed:

- Property location and ownership
- Land use and zoning and master plan
- Physiographic and environmental characteristics
- Order of magnitude development costs

Information can be obtained, where available, from published documents such as:

- County tax assessment maps
- County topographic, soils, and floodplain mapping
- Previous actions on the property, such as rezoning applications, special exceptions, subdivision plats, and site plans

- U.S. Department of Agriculture, Soil Conservation Service Soil Survey
- U.S. Fish and Wildlife Service National Wetlands Inventory (NWI) mapping
- U.S. Geological Survey (USGS) mapping
- Utility plans
- Comprehensive (master) plans
- Road network plans

As in any engineering feasibility study, but most importantly in the site selection study, the elements that are most important to the client's program need to be identified and prioritized. While many criteria are of equal importance, often the land use criteria are the first reviewed. Zoning is a major factor in whether the program can be located on the property. Consideration must be given to the time necessary to amend the land use map or rezone the property if the intended use is not permitted by right. This is documented and presented to the client as the first piece of information. The time to amend a comprehensive plan or rezone a piece of property, as well as the interaction with the governing body and coordination with the citizens, may eliminate a site for consideration early in the evaluating process.

After zoning and land use issues, many other factors can affect the selection of the site. In some instances, site characteristics can be overcome during the development of the site. If the soils can be used for the development of the site, then grading alone can eliminate slopes, hills, and valleys to produce a desirable and functional area. This must, however, be weighed against the vegetation on the site. Grading the entire site destroys stands of vegetation.

Environmental characteristics of the site also need to be determined and evaluated against the client's program. Wetlands, floodplains, and Level I environmental assessments (ESAs) determine the available area suitable for development. Again this information will be available from the engineering feasibility study, but it needs to be presented in a consistent manner for comparison.

Capacity of the infrastructure is often the most critical to the development potential of a site, and upgrading the infrastructure can be a significant cost to the client. The major components of the infrastructure are utilities and transportation. The availability of water, sewer, power, and telephone is usually determined in the site assessment study, but the unique aspects of the client's program for special utility requirements need to be assessed as a portion of the site selection study. Additional power capability for computers, dual feeds, telephone expansion, and security are all possible elements that might need to be considered as part of the matrix. Again, these factors are to be compared on a site-to-site basis.

The ability to provide adequate access to the site is also a consideration in any site selection study. Not only does

the existing and proposed road network need to be considered, but so do rail, bus, air and water, or any other means of transportation that might be critical to the client's program. Such access may be a part of the overall master transportation plan for the jurisdiction. While this may provide the improvements necessary for the development of the property, the timing of the improvements is crucial.

Not only are the factors that affect the development of the site important to the client, but so is the ability of the client's program to be flexible and adjust to the site. A rigid site layout or program could, by itself, eliminate a site from the selection process. The engineering feasibility study and associated yield study will present the development potential of the site. A comparison between the yield study and the client's program will determine the amount of adjustment and the ensuing flexibility required to merge the client's program to the different sites. The greater the flexibility in the program, the better the chance the program can fit to a site. Lastly, as stated earlier as part of the engineering feasibility study, the cost to develop the site is a critical element in the feasibility of the site. Not only are the development's construction costs important in a site selection study, but so are the relative construction costs associated with bringing the sites to a comparable level. If a few additional dollars need to be spent on a site (or on the infrastructure for a site) that does not require rezoning or any land use time, then the additional dollars may be less expensive than the time to go through the planning process. While the client often does not own the property until it is zoned and does not have a great carrying cost on the land loan, rezoning is still a time-consuming process that results in a lost (or delayed) opportunity to make money on the project. In some cases the site selection study may have a criterion for site availability. Is the site available now for development, or are there steps and time required before development can begin? In that case, the only way to make the site immediately available may be through spending some money to make it acceptable as an element of review. Money may have to be spent to improve the access or provide utilities and thus avoid having to go through the rezoning process.

The sites' characteristics and all of the information affecting the development program that was obtained in the engineering feasibility study need to be presented uniformly for each proposed site. This information should be evaluated based on how it affects the development program before you can determine the potential of the site. Once all of the data are collected, a value or rank is assigned for each factor. A matrix can then be used to collectively show the information in order to evaluate each property.

An example of the procedure is shown in Tables 6.2 and Table 6.3. Table 6.2 lists the criteria pertinent to one particular development program. The type of shading of the boxes indicates the basic differences of each item. Each potential tract of land is then evaluated based on the listed

criteria. Table 6.3 is a compilation of the criteria for each alternative site. Selecting the property best suited for the program is ultimately a business decision for the client. Each item does not necessarily have equal weight in terms of importance or priority, and only the client knows what the relative weight of importance or priority of the items are in terms of his best interests or situation. The land design consultant supplies the data for the client to analyze to make the selection.

### OTHER IMPACT STUDIES

Other studies may be required by the local governing body to determine the impact of the proposed development on the community, environment, or public infrastructure. These typically are performed after the proposed use and development plan has progressed towards its final stages. However, in many cases, the output from these studies is considered during development applications to determine the appropriateness of the use or intensity. Often, it is used to determine appropriate dedications or improvements, assess impact or similar fees, or set a timetable for the staging of development.

Land development specialists other than the design consultants often perform these studies. However, they are mentioned here because many of the data inputs collected for the feasibility study are used in subsequent analysis. In addition, the outcome of these studies could affect project feasibility. This is especially true where known constraints in public systems or issues of community priority may have direct bearing on the use and yield of development sites. These studies are discussed in greater detail in the context of project design in Chapter 7, since information from the plan is required to conduct the study.

### Traffic Study

Traffic studies often are required as part of the submittal package for a rezoning or special exception application. They serve two primary purposes. The first is to identify potential impact of the proposal on the transportation system in the general area of the development. Secondly, the study looks specifically at potential impact on roads and intersections in the immediate vicinity of the project. In communities that assess impact fees on development or operate under an adequate public facilities ordinance, a traffic study may be required for subdivision and building permit approval as well.

In addition to its review in consideration of project approval, the results of the traffic study serve to indicate system improvements to mitigate project impact. For instance, the need for additional right of way for road widening or turn lanes may be a consequence of the study. This may affect project yield by encumbering otherwise developable land. The local governing body also may use the study to support operating restrictions on certain uses to increase the efficiency of the existing system.

**TABLE 6.2 Criteria and Values for Elements of the Site Selection Study**

<b>PLANNING CRITERIA</b>	
Existing zone	<input type="checkbox"/> Zoned for intended use <input checked="" type="checkbox"/> Zoned for nonresidential use <input type="checkbox"/> Zoned for agricultural/residential
Adjacent land use	<input checked="" type="checkbox"/> Adjacent uses office/mixed use <input checked="" type="checkbox"/> Adjacent uses nonresidential <input type="checkbox"/> Adjacent uses agricultural/residential
Consistency with comprehensive plans	<input checked="" type="checkbox"/> Specific use consistent with comprehensive plan <input checked="" type="checkbox"/> General use consistent with comprehensive plan <input type="checkbox"/> Use not consistent with comprehensive plan
<b>SITE CHARACTERISTICS</b>	
Topography	<input checked="" type="checkbox"/> Relatively flat site <5% <input checked="" type="checkbox"/> Moderate slope constraints 5–15% <input type="checkbox"/> Significant slope constraints >15%
Drainage	<input checked="" type="checkbox"/> Single drainage shed <input checked="" type="checkbox"/> Several drainage sheds <input type="checkbox"/> Numerous drainage sheds
Soils/Substances	<input checked="" type="checkbox"/> Minimum grading/excavation problems anticipated <input checked="" type="checkbox"/> Moderate grading/excavation problems anticipated <input type="checkbox"/> Significant grading/excavation problems anticipated
Vegetation	<input checked="" type="checkbox"/> Significant native vegetation for landscape buffer/character <input checked="" type="checkbox"/> Moderate native vegetation for landscape buffer/character <input type="checkbox"/> No native vegetation for landscape buffer/character
Structures	<input checked="" type="checkbox"/> No existing on-site structures <input checked="" type="checkbox"/> Existing structures of marginal value/concern <input type="checkbox"/> Existing structures of significant value/concern
<b>ENVIRONMENTAL CHARACTERISTICS</b>	
Wetlands	<input checked="" type="checkbox"/> Minimum wetlands constraints ( <i>approx. less than 1 acre of care area</i> ) <input checked="" type="checkbox"/> Moderate wetlands constraints ( <i>approx. between 1 and 10 acre of care area</i> ) <input type="checkbox"/> Significant wetlands constraints ( <i>approx. more than 10 acre of care area</i> )
Floodplain	<input checked="" type="checkbox"/> No floodplain <input checked="" type="checkbox"/> Floodplain but not impact on care development area <input type="checkbox"/> Floodplain within care development area
<b>INFRASTRUCTURE—UTILITIES</b>	
Power Water supply Sanitary sewer Communications Natural gas	<input checked="" type="checkbox"/> Available capacity on-site or immediate proximity <input checked="" type="checkbox"/> Available in general vicinity <input type="checkbox"/> Capacity not available in general vicinity

**TABLE 6.2 Criteria and Values for Elements of the Site Selection Study (Continued)**

<b>PLANNING CRITERIA</b>	
<b>INFRASTRUCTURE—TRANSPORTATION</b>	
Existing roads	<input type="checkbox"/> Two or more existing roads available to access/egress site and major regional arterial/freeway in close proximity <input checked="" type="checkbox"/> Two or more existing roads available to access/egress site <input type="checkbox"/> One existing road available to access/egress site
Site access	<input checked="" type="checkbox"/> No encumbrances to two points of access/egress <input checked="" type="checkbox"/> Limited encumbrances to two points of access/egress <input type="checkbox"/> Both access/egress points significantly encumbered
Proposed roads	<input checked="" type="checkbox"/> Multiple master planned or existing roads adjacent core development area and regional arterial/freeway in case proximity <input checked="" type="checkbox"/> Two master planned roads or existing road adjacent core development area <input type="checkbox"/> One master planned road or existing road adjacent to core development area
Mass transit	<input checked="" type="checkbox"/> Rail and bus available <input checked="" type="checkbox"/> Bus available <input type="checkbox"/> No mass transit available
Rail	<input checked="" type="checkbox"/> Rail line immediately adjacent to site <input checked="" type="checkbox"/> Rail line in general vicinity <input type="checkbox"/> No rail line in general vicinity
Flight paths	<input checked="" type="checkbox"/> No flight path near site <input checked="" type="checkbox"/> Flight path near site but flight altitude <input type="checkbox"/> Flight path nearby and low altitude
<b>PROGRAM FIT</b>	
	<input checked="" type="checkbox"/> Program fits/additional acreage provides location flexibility <input checked="" type="checkbox"/> Program fits/limited location flexibility <input type="checkbox"/> Program does not fit
<b>R.O.M.* COSTS</b>	
	<input checked="" type="checkbox"/> Excessive or unusual cases <input checked="" type="checkbox"/> Ranking 1 through 9 low cost to high cost <input type="checkbox"/> N/A—non-applicable

The traffic study consists of four basic elements:

1. System capacity and level of service analysis
2. Background traffic assessment, an analysis of existing traffic that also considers traffic generated by pending projects and potential projects for which rezoning approval is not required
3. Projected traffic generation for the proposed development, including probable origins or destinations and modes of transportation
4. Analysis of adjoining roads and nearby intersections to determine need for right of way or pavement

widening, turn lane improvements, and traffic control signs or signalization

### **School/Pupil Generation**

In communities experiencing overcrowding in classrooms, the ability to develop a site may often be jeopardized where project approval depends on available capacity. The development application may include requirements for a pupil generation study to be performed by the development team. This examines the impact on the public schools system by the new development. Through the analysis, the developer may be able to demonstrate that because of characteristics peculiar to the project, demand for available capacity may

TABLE 6.3 Matrix Comparison of Properties

CRITERIA	ALTERNATE SITES								
	A	B	C	D	E	F	G	H	I
Planning criteria									
Existing zoning	☑	☑	☑	☑	☑	■	□	□	□
Adjacent land use	■	■	■	■	☑	☑	□	□	☑
Consistency with comprehensive plans	☑	■	☑	□	□	■	□	□	☑
Site characteristics									
Topography	■	☑	☑	☑	■	■	■	□	■
Drainage	☑	□	☑	☑	■	■	☑	☑	☑
Soils/subsurface	☑	☑	☑	□	☑	☑	■	□	■
Vegetation	□	■	☑	■	■	☑	☑	☑	☑
Structure	☑	□	□	☑	☑	☑	☑	☑	☑
Environmental characteristics									
Wetlands	☑	☑	☑	☑	■	☑	■	☑	■
Floodplain	☑	☑	□	☑	□	■	☑	□	☑
Infrastructure—utilities									
Power	☑	■	☑	□	☑	□	□	□	☑
Water supply	■	■	■	☑	☑	■	□	□	☑
Sanitary sewer	■	☑	☑	☑	☑	■	□	□	☑
Communications	☑	☑	☑	☑	☑	□	□	☑	☑
Natural gas	■	■	■	□	□	■	□	■	□
Infrastructure—transportation									
Existing roads	■	■	■	□	☑	■	□	☑	☑
Site access	■	■	■	■	■	■	☑	□	■
Proposed roads	■	■	■	☑	☑	■	□	□	□
Mass transit	□	□	□	□	□	□	□	□	■
Rail	□	□	□	□	□	□	□	□	■
Flight paths	☑	☑	□	☑	□	■	■	□	□
Program fit									
R.O.M.* costs (ranked; #1 = Lowest cost)	3	4	N/A	6	1	5	7	8	2

\*Rough order of magnitude.

be less than anticipated. Many communities recognize that factors such as housing type and bedroom count have direct correlation to the number of school-age students that will occupy a development. In addition, developments geared to older single-person households are likely to generate few students, even if age restriction is not a factor or prohibited

by law. Rather than accept standard local housing unit/pupil ratios, the developer may have the option of surveying similar developments in the community. With this study, it may be possible to demonstrate that the probable impact of the project on student population is lower than originally suggested.

**EXAMPLE OF A PRELIMINARY FEASIBILITY STUDY**

Preliminary Engineering Feasibility Study  
January 1992

**TRACT LOCATION**

The subject site consists of approximately 30 acres and is located in the Sully District of Fairfax County, Virginia. The site is identified on the Fairfax County Tax Map as 45-2 ((1)) part of parcel 1. The property is bordered by the Lee-Jackson Memorial Highway (Route 50) to the south, the Murray Farms Subdivision to the east, the Fairfax County Parkway (Route 7100) to the northeast, and undeveloped property to the north and west. The mailing address for the subject site is 12908 Lee-Jackson Memorial Highway, Fairfax, Virginia 22033.

**ZONING/COMPREHENSIVE PLAN**

The study area is in the UPS Lee-Jackson Planning Sector of the Upper Potomac Planning District in Area III of the Fairfax County Comprehensive Plan. The property is currently zoned R-1; low density residential use not to exceed one dwelling unit per acre. The Comprehensive Plan recommends low to medium residential use at two to three dwelling units per acre. The Plan further states that “. . . Complete consolidation of Parcels 1, 2, and 3 is a condition for consideration of the high end of the Plan range, three dwelling units per acre.” The portion of the Murray Farms subdivision located immediately east of the subject parcel and south of the Fairfax County Parkway is planned for a residential use at one to two dwelling units per acre. The Plan provides an optional density of four to five dwelling units per acre for this area if all the land is consolidated. Please refer to Attached 1, Zoning Map.

The Fairfax County Zoning Ordinance permits the establishment of golf driving ranges and commercial golf courses in the R-1 District with the approval of a Group 6 Special Permit by the Board of Zoning Appeals. Included as Appendix #1 is a copy of a portion of Part 6 of Article 8 of the Zoning Ordinance which specifies the standards for all Group 6 uses as well as the additional standards applicable to golf courses and golf driving ranges.

The process for a Special Permit application takes approximately 90 days from the date the application is accepted by Fairfax County. State statute requires action by the Board of Zoning Appeals within 90 days unless the applicant agrees to a deferral beyond the 90 days period. The timeframe for the actual preparation of the Special Permit Plat will be dependent upon the receipt of a conceptual design of the facility by your golf course consultant. Once that information is received, the Plat can be prepared in approximately ten (10) working days.

**UTILITY AVAILABILITY**

**Water:** The subject site is located in the Fairfax County Water Authority (FCWA) service area. According to Mr. Don Hume of FCWA, there are two water mains which could serve the subject site: (1) A 14-inch water main located along the south side of Route 50, and (2) A 12-inch water main located along the north side of Route 50 approximately 450 feet east of the site. Please refer to Attachment #2, Water Availability Map.

Connecting to the existing 12-inch water main would require boring under the roadway for approximately 200 feet. An eight-inch water main within a 20-inch casing would be required. The second alternative would require the extension of a 12-inch water main approximately 1,000 feet from the existing terminus to the subject site. Preliminary calculations include that either option would cost approximately \$50,000 to \$55,000.

It appears that no offsite easements would be required with either water service alternative since all connections and extensions would be within the Virginia Department of Transportation (VDOT) right-of-way.

Mr. Hume stated that further analysis by FCWA will be required in order to confirm exact line size requirements and to determine which alignment option may best benefit the developer and FCWA.

**Sanitary Sewer:** The subject site is located in Subshed T-2 of the Cub Run Sanitary Sewer service area. According to Mr. Jerry Jackson of the Fairfax County Department of Public Works (DPW), sanitary sewer service is not readily available to the site. Although sanitary sewer service is available for the townhouse and garden apartment development immediately south of Route 50, Mr. Jackson stated that a majority of the subject site is in a different drainage shed and cannot be sewered by gravity flow to this location. The closest available sewer by gravity flow is a 12-inch line approximately 2,100 feet northwest of the property. Please refer to Attachment #3, Sanitary Sewer Availability Map.

This existing sanitary sewer, which is within the International Town and Country Club property, was designed to accommodate flows from the subject site at a proposed density of two dwelling units per acre (current Comprehensive Plan density). Mr. Jack-

Reprinted with permission of Westland Golf Inc., Fairfax, VA.

son stated that approximately 12 units of the subject site, which natural drains south across Route 50, could be developed so that sewer would be diverted to the northwest. Given the topography of the site, the diverted sewer system should be able to cross the natural drainage divide at a depth less than the allowable maximum of 16 feet. Mr. Jackson stated that this would be the alignment preferred by DPW. A 10-foot-wide offsite sanitary sewer easement would be required from the owners of the adjacent Parcel 3 and the International Town and Country Club in order to construct the required eight-inch sanitary sewer line.

The Fairfax County Health Department was contacted for information regarding the septic system used for the residence on the subject property. The Health Department has not approved septic field plans or records for the property. Conversations with the owners have determined that the septic field is located in the southwest portion of the property behind the garage. The system was constructed approximately 25 to 30 years ago.

A preliminary soils study, including test borings and other field observations, was conducted by Soil Tech, Inc. Preliminary findings indicate that there are two areas suitable for conventional septic drain fields: (1) The southwest portion of the property immediately behind the onsite structures, and (2) the southeast portion of the property near Route 50. These areas appear to be capable of processing from 600 to 1,800 gallons of waste water per day. This would equate to a commercial/office establishment with 20 to 60 employees. Additional areas may be suitable for more progressive systems such as elevated sand mounds. It should be noted that additional studies and Fairfax County Health Department approval will be required before the drain field locations and types can be finalized.

### **TRANSPORTATION/ACCESS REQUIREMENTS**

The more logical location for access along Route 50 appears to be opposite the median break near the southeast corner of the subject site. Based upon Fairfax County topography, this location does not appear to present any horizontal or vertical sight distance problems; however, this location would still need to be approved by the Fairfax County Office of Transportation (OT) and the Virginia Department of Transportation (VDOT).

Route 50 is designated on the Comprehensive Plan as a six-lane divided facility. According to a spokesperson from OT, the following transportation improvements would be requested/required for the development of this property:

- Construction of a third through lane for westbound Route 50 or a monetary contribution for construction avoidance,
- Construction of a left-turn lane into the site eastbound Route 50,
- Construction of a right-turn lane into the site from westbound Route 50, and
- Construction of a service drive along the property's frontage unless a waiver can be obtained during the zoning process.

Additional development conditions may be imposed as the special permit application is reviewed by the appropriate State and County agencies.

It should be noted that Route 50 is currently being improved from Townwood Drive west to Plaza Lane. This construction is being funded by a developer in conjunction with the expansion of Greenbriar Shopping Center. According to a spokesperson from VDOT, this project involves the construction of dual left turn lanes for westbound Route 50 into Majestic Lane, Plaza Lane and the shopping center entrance. Service drive improvements are also being constructed.

### **SITE CHARACTERISTICS/TOPOGRAPHY**

The subject site contains five structures, all of which are located in the southwestern portion of the property. These structures include:

- A one-story wood-framed residence with basement and attic;
- A wood-framed two-car garage; and
- Three wood-framed out buildings, evidently used for the storage of farm equipment and tools.

The entire property remains relatively clear of mature vegetation; although cedar trees of three to six feet in height are beginning to overtake a large portion of the property. Overhead electric and telephone lines, which serve the onsite residence, extend along



the site's Route 50 frontage for approximately 400 feet. A concrete drainage ditch also extends along the site's Route 50 frontage for approximately 800 feet. This ditch drains into a 48-inch culvert which extends under Route 50 towards the Grays Point subdivision.

Onsite elevations vary from a high point of approximately 388.2 at the property's southwest corner to a low point of approximately 370.0 at the property's northwest corner where a swale exits the property. Onsite slopes are generally mild and primarily range from 1% to 10%. A small area of slopes in excess of 15% is located along the northern property line. Please refer to Attachment #4, Topography Map.

The property contains three pronounced swales which convey stormwater runoff through the subject site. These swales are not identified on the U.S. Geological Survey Quadrangle Map (Herndon) as either perennial or intermittent streams; therefore, minimal water flow is probably only apparent during storm events. However, during the field investigation conducted January 7, 1992, standing water was apparent in all three swales. For a more detailed discussion of stormwater runoff and onsite drainage patterns, please refer to the section entitled "Stormwater Management Requirements."

## SOILS

The Fairfax County Soils Survey was reviewed for preliminary geotechnical information. The soils survey indicates that the subject site is comprised of the following soil types:

<u>Soil Number</u>	<u>Soil Name</u>	<u>Percentage of Site</u>
1	Mixed Alluvial	1%±
14	Manassas	7%±
67	Penn FSL	41%±
72	Bucks (L)	< 15%±
76	Calverton (L)	16%±
80	Croton	30%±
273	Readington	4%±

Please refer to Attachment #5, Soils Map.

General ratings for development within these soil types can be summarized as follows:

<u>Soil Name</u>	<u>Subsurface Drainage</u>	<u>Foundation Support</u>	<u>Slope Stability</u>	<u>Problem Class</u>
Mixed Alluvial	Poor	Poor	Good	A
Manassas	Marginal	Fair	Good	B
Penn	Fair	Good	Good	C
Bucks	Good	Good	Good	C
Calverton	Marginal	Poor	Good	A
Croton	Poor	Poor	Good	A
Readington	Marginal	Good	Good	B

A geotechnical engineering report is mandatory for all construction and grading within Group A Soils, i.e. the Mixed Alluvial, Calverton and Croton soils. Some of the characteristics which are typical of soils within this problem class include:

- Flooding hazards following storm events,
- Low bearing values for foundation supports,
- High seasonal groundwater tables in drainage ways or low-lying areas,
- Moderate to high shrink-swell potential often having slow to very slow permeability rates, and
- Shallow depth to bedrock.

These soils comprise approximately 47% of the site.

A geotechnical engineering report may not be required for the Group B Soils if adequate provisions to circumvent soil-related

problems are incorporated into the site plan, or if the location of these soils will not impact the proposed development. Onsite soils within this problem class are the Manassas and Readington soils. Some of the characteristics which are typical of these soils include:

- Low bearing values for foundation supports,
- High seasonal groundwater tables in drainage ways or low-lying areas,
- Perched groundwater above restrictive soil or rock layers, and
- Shallow depth to bedrock.

These soils comprise approximately 11% of the subject site.

A geotechnical engineering report is usually not required for the Group C Soils as long as construction is in natural soils and not in controlled fills. Onsite soils within this problem class are the Penn and Bucks soils. Some of the characteristics which are typical of these soils include:

- Shallow depth to bedrock, and
- Slow permeability rates.

These soils comprise approximately 42% of the subject site.

In conclusion, based upon the information reviewed, a geotechnical investigation and report will be required for the development of the subject site.

### **WETLANDS/OFFICE INVESTIGATION**

An office investigation was conducted to determine the possible existence of wetlands on the subject site. The U.S. Fish & Wildlife Service National Wetland Inventory (NWI) mapping for this area (Herndon, Virginia-USGS Quad Sheet) was reviewed and it shows an area of Palustrine Forested Wetlands along Oxlick ranch northwest of the property. Please refer to Attachment #6, NWI Map. Additional areas of Palustrine Forested Wetlands and Palustrine Open Water Wetlands are shown on adjacent properties to the north and west of the subject site.

While the NWI Mapping is generally a reliable source for identifying the most obvious areas of wetlands, it is important to note that the mapping is compiled from aerial photography and omissions of existing wetlands typically occur. For this reason, it is essential that other sources of information be utilized to determine the possible existence of additional wetlands.

As an additional source of information, the Fairfax County Soils Survey was reviewed to determine if any of the onsite soils exhibit characteristics commonly associated with nontidal wetlands. The presence of these soils, commonly referred to as Hydric Soils, indicates a high probability of wetlands occurring within a particular area. In this regard, the Fairfax County Soils Scientist has assigned a Wetlands Probability Index to every soil type that occurs within the County. Hydric Soils, with the highest probability of supporting wetlands, were assigned an index of one (1). As the index increases to a maximum value of five (5), the likelihood of wetlands occurring decreases. The Wetland Probability Index associated with each of the onsite soils can be summarized as follows:

Soil Number	Soil Name	Wetlands Probability Index	Hydric	Percentage of Site
1	Mixed Alluvial	1	Yes	1%±
14	Manassas	4	No	7%±
67	Penn	5	No	41%±
72	Bucks (L)	5	No	<1%±
76	Calverton (L)	4	No	16%±
80	Croton	1	Yes	30%±
273	Readington	4	No	4%±

Please refer to Attachment #7, Hydric Soils Map.

Based upon a countywide average, a Wetlands Probability Index of one (1) suggests that from 80% to 100% of the area containing Mixed Alluvial and Croton soils may contain wetlands. An index of four (4) suggests that from 5% to 20% of the area con-

taining Manassas, Calverton and Readington soils may contain wetlands. An index of five (5) suggests that from 0% to 5% of the area containing the Penn and Bucks soils may contain wetlands. Based upon these probabilities, the total acreage of the subject site which may contain wetlands can be summarized as follows:

<u>Soil Name</u>	<u>Acreage</u>	<u>Wetlands Probability</u>	<u>Possible Acreage of Wetlands</u>
Mixed Alluvial and Croton	9.3±	80%–100%	7.4–9.3
Manassas, Calverton, and Readington	8.1±	5%–20%	0.4–1.6
Penn and Bucks	12.6±	0%–5%	0.0–0.6
<b>TOTALS</b>			<b>7.8–11.5</b>

In conclusion, based upon the information reviewed, the presence of wetlands may have a significant effect on the development of the subject site. For this reason, a field investigation is recommended to better define the acreage which may fall under the jurisdiction of Section 404 of the Clean Water Act.

### **WETLANDS/FIELD INVESTIGATION**

On January 10, 1992, a preliminary wetlands assessment was conducted on the subject site. Wetlands were identified following the procedures set forth in the *Corps of Engineers Wetlands Delineation Manual* (1987). The field investigation involved the collection of data documenting hydrologic conditions, soil characteristics and dominant vegetation in order to establish an approximate wetlands boundary.

Soil saturation and inundation was observed in three areas. Soil color indicative of hydric soil conditions was also present. Dominant vegetation common to all wetlands on the site include soft rush, woolgrass, redtop bentgrass, marsh fern and several sedges. Buttonbush, red maple and sweet gum were the dominant shrubs in the southern and northwestern wetlands areas. The total amount of jurisdictional areas delineated on the site was approximately 3.7 acres. Please refer to Attachment #6, National Wetlands Inventory Map.

### **FLOODPLAIN**

The Federal Emergency Management Agency (FEMA) and United States Geological Survey (USGS) maps were reviewed for the possible existence of floodplains in the vicinity of the subject site. Both sources indicate that the closest floodplain is located offsite approximately 3,000 feet northwest of the subject site along Oxlick Branch. Please refer to Attachment #8, FEMA Map.

Oxlick Branch passes within approximately 80 feet of the northeast corner of the property. This is just downstream of the triple box culvert which carries the stream under the Fairfax County Parkway.

The drainage area contributing to Oxlick Branch at this point is greater than 70 acres and by Fairfax County definition will require the delineation of a floodplain. A review of Fairfax County Parkway storm computations and available topography indicate that the estimated 100-year flood elevation (approximately 4 feet above the stream bed) will not encroach on the subject site.

### **STORMWATER MANAGEMENT REQUIREMENTS**

A ridge line bisects the site into two major drainage divides. The northern portion ( $\pm 18$  acres) drains into Oxlick Branch, which is a tributary of Flatlick Branch. The southern portion ( $\pm 12$  acres) drains such across Route 50, eventually entering Little Rocky Run. The runoff draining from the northern portion of the site exits at two low points; approximately 12 acres exit at the northwest corner of the site. There are approximately 14 acres, of which approximately eight areas are from offsite, exiting at the northeast corner.

The subject site is located in the Cub Run drainage shed. This shed flows into the Occoquan Reservoir which is the water supply source for a large portion of Fairfax County. Subsequently, development of this property is subject to the implementation of specially designed water quality control measures for stormwater runoff, referred to as Best Management Practices (BMPs). Depending on the proposed development program of the subject site, stormwater management and BMPs could be accommodated

in the form of two or three detention ponds. One of these ponds could be located at the southern low point of the site near the culvert crossing Route 50. The other pond(s) could be located at one or both of the low points along the northern boundary of the site. Please refer to Attachment #9, Drainage Map

Preliminary calculations indicate that approximately 85,000 cubic feet of total storage volume may be necessary to satisfy detention and BMP requirements for the full development of the site. Fairfax County's "Policy and Requirements for Adequate Drainage" require that a developing site convey stormwater runoff into a natural watercourse or storm drainage pipe of sufficient capacity without causing adverse impacts on the watercourse or downstream properties. A preliminary field investigation indicated that the two outfalls to the north may be inadequate to accommodate concentrated stormwater runoff. The drainageway outfalling northwest of the site is of particular concern. As a result, offsite watercourse improvements and/or storm sewer facilities and associated offsite storm drainage easements may be necessary to remedy any inadequacies. A more detailed analysis of this situation will be required as development of the subject site progresses.

### **CHESAPEAKE BAY PRESERVATION ORDINANCE IMPACTS**

A preliminary investigation was conducted to determine the potential impact of the Chesapeake Bay Preservation Ordinance on the development of the subject site. In this regard, the following Resource Protection Area (RPA) features were investigated:

- Tidal Wetlands and Tidal Shores—Since this area of the County is not subject to the influence of tides, these features are not applicable.
- Tributary Streams—As mentioned previously, none of the onsite swales are depicted as tributary or perennial streams on the USGS Quad Sheet; therefore, this feature is not applicable.
- Water Courses With Drainage Areas Greater Than 70 Acres—As mentioned previously, Oxlick Branch, located northeast of the subject site, drains an area greater than 70 acres; therefore, this stream is by definition a RPA feature.
- Nontidal Wetlands Connected By Surface Flow and Contiguous to a Tidal Wetlands or Tributary Stream—As previously mentioned, there are no tidal wetlands or tributary streams within the immediate vicinity of this property; therefore, the nontidal wetlands as discussed in the section entitled "Wetlands/Field Investigation" are not considered RPA features because they are not contiguous to tidal wetlands or tributary streams.

Based upon this phase of the investigation, Oxlick Branch is the only RPA feature in the vicinity of the subject site.

The provisions of the ordinance require the establishment of a buffer area to protect other components of the RPA from significant degradation due to land disturbing activities. In this regard, the following components, which comprise a buffer area, were investigated:

- Any land within a floodplain—As mentioned previously, the estimated floodplain for Oxlick Branch does not impact the subject site; therefore, this component is not applicable.
- Any nontidal wetland that is continuously connected to a watercourse—None of the onsite wetlands are continuously connected to Oxlick Branch; therefore, this component is not applicable.
- Any land within 100 feet of a RPA feature—As mentioned previously, Oxlick Branch passes within 100 feet of the subject site, therefore, this 100 feet buffer encroaches upon the subject site.
- Any land with a slope greater than or equal to fifteen (15) percent where such slope begins within fifty (50) feet of a floodplain—As mentioned previously, a small area of slopes in excess of 15 percent are found on the subject site, however, these slopes are not within 50 feet of the Oxlick Branch flood plain; therefore, this component is not applicable.

Oxlick Branch, as a RPA feature, is combined with the 100-foot setback, which acts as a buffer area, to form the Resource Protection Area (RPA) as shown on Attachment #10, RPA Map. The remainder of the site is designated a Resource Management Area (RMA).

It should be noted that this investigation is based upon the Chesapeake Bay Preservation Ordinance as endorsed by the Fairfax County Board of Supervisors (BOS) on May 20, 1991. As a condition of approval, this ordinance is currently being reviewed by the Chesapeake Bay Local Assistance Board (CBLAB). A date for official adoption of the Ordinance by the BOS has not yet been established.

In the event that the Chesapeake Bay Preservation Ordinance has not been officially adopted at the time the subject site moves forward into the zoning process, the County will impose their Environmental Quality Corridor (EQC) policy as a condition of development. Since the components of an EQC are essentially the same as those for a RPA, the impacted area would remain unchanged.

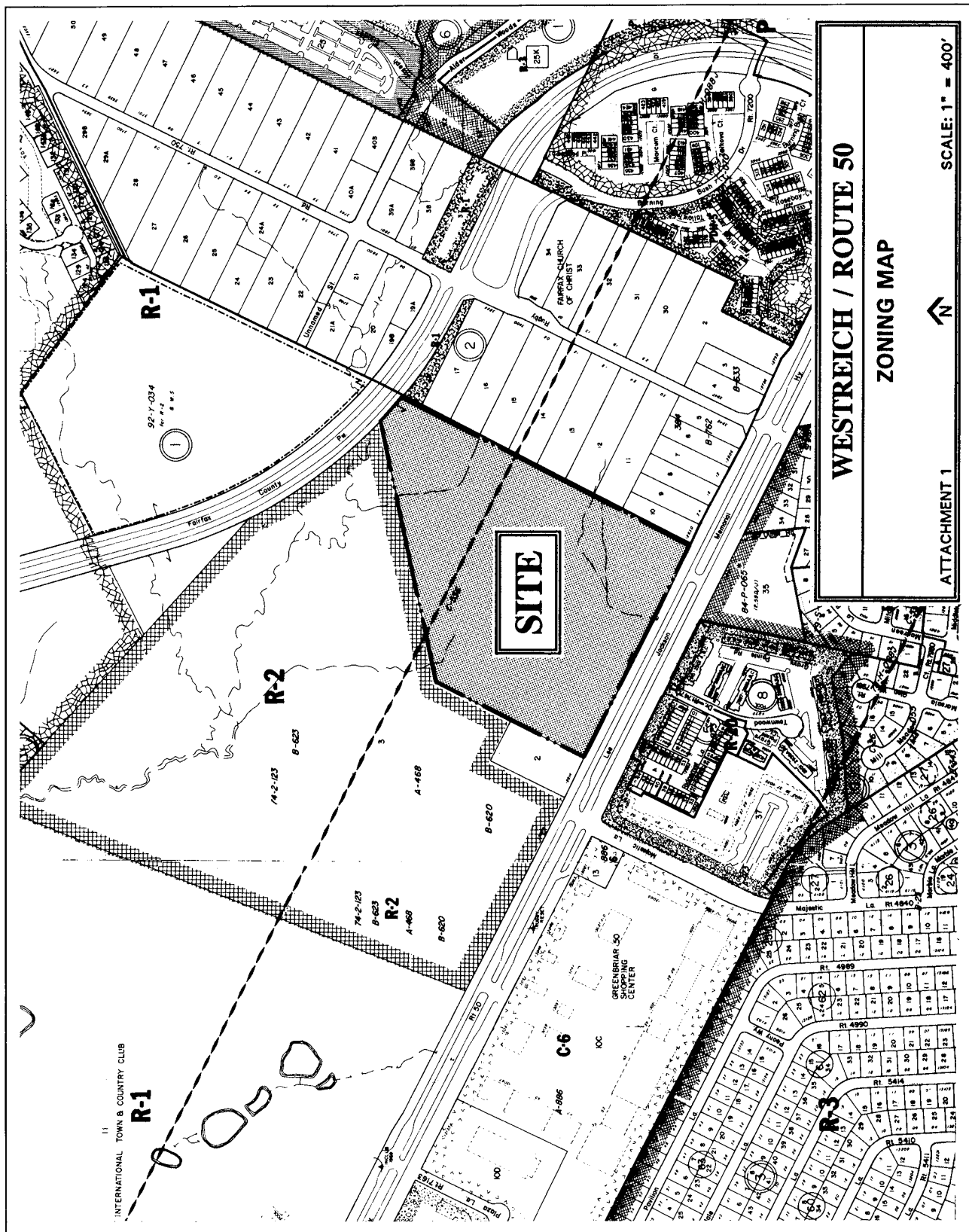
### **QUALIFIER**

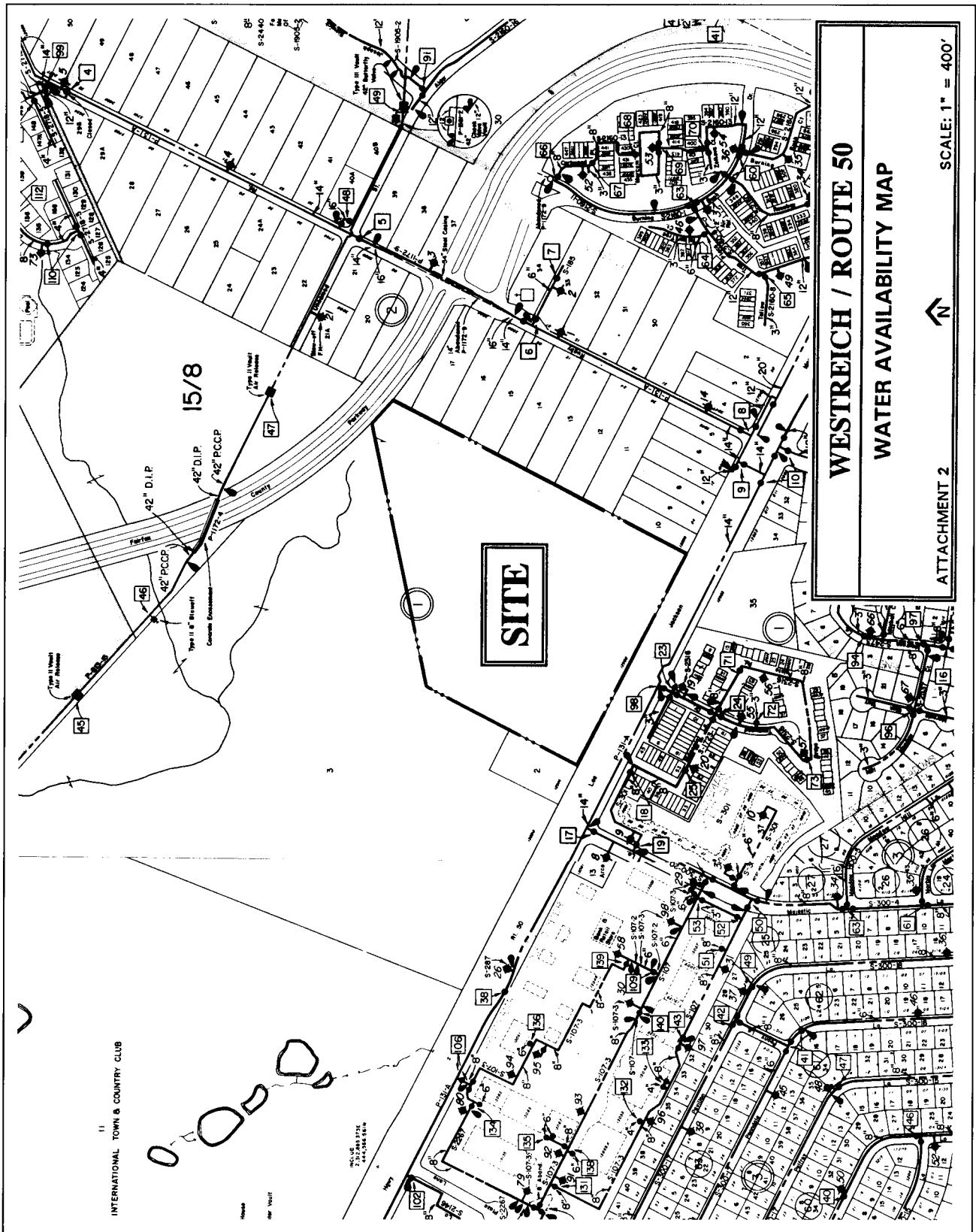
This report is a *preliminary* engineering analysis of the subject site's development potential. Issues addressed were limited to:

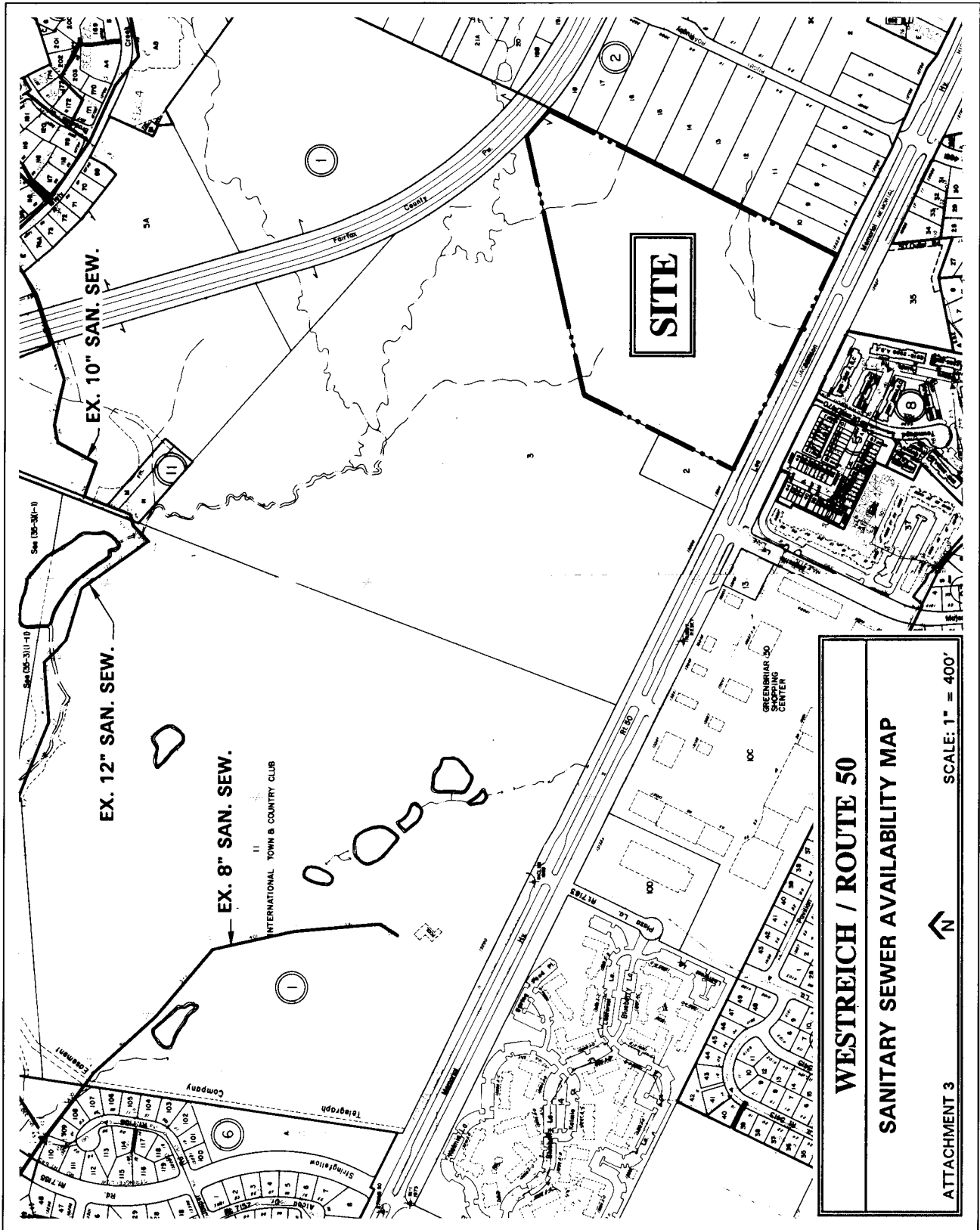
- Tract location
- Zoning/Comprehensive Plan
- Utility Availability (Water and Sanitary Sewer)
- Transportation/Access Requirements
- Site Characteristics/Topography
- Soils
- Wetlands (Office and Field Investigations)
- Floodplain
- Stormwater Management Requirements, and
- Chesapeake Bay Preservation Ordinance Impacts

This report is a *preliminary* engineering analysis of the subject site's development potential. Issues addressed were limited to: relate to subjects beyond the scope of this study.

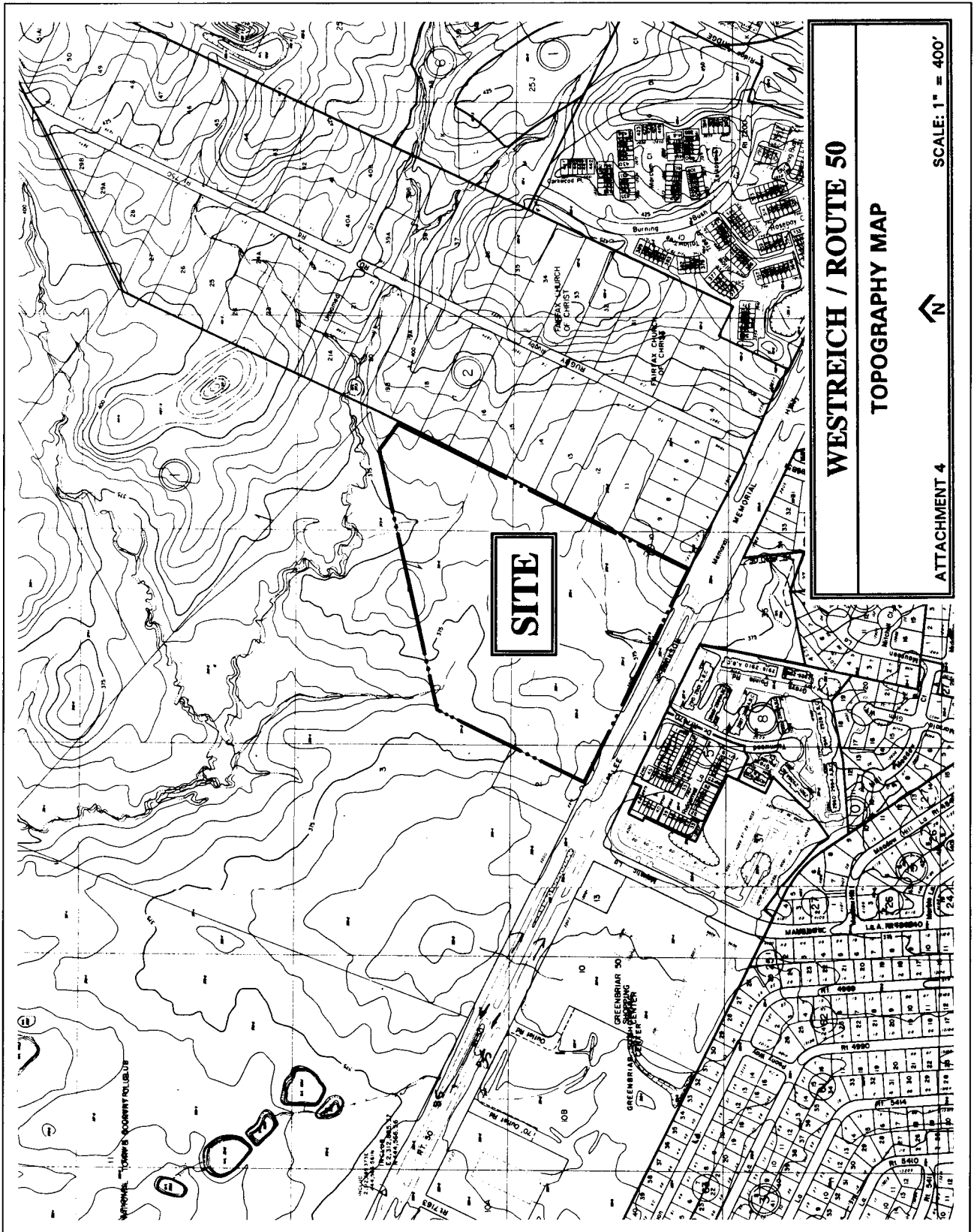
Much of the information presented herein was obtained from public officials whose opinions are generally reliable and sufficient for preliminary planning purposes.

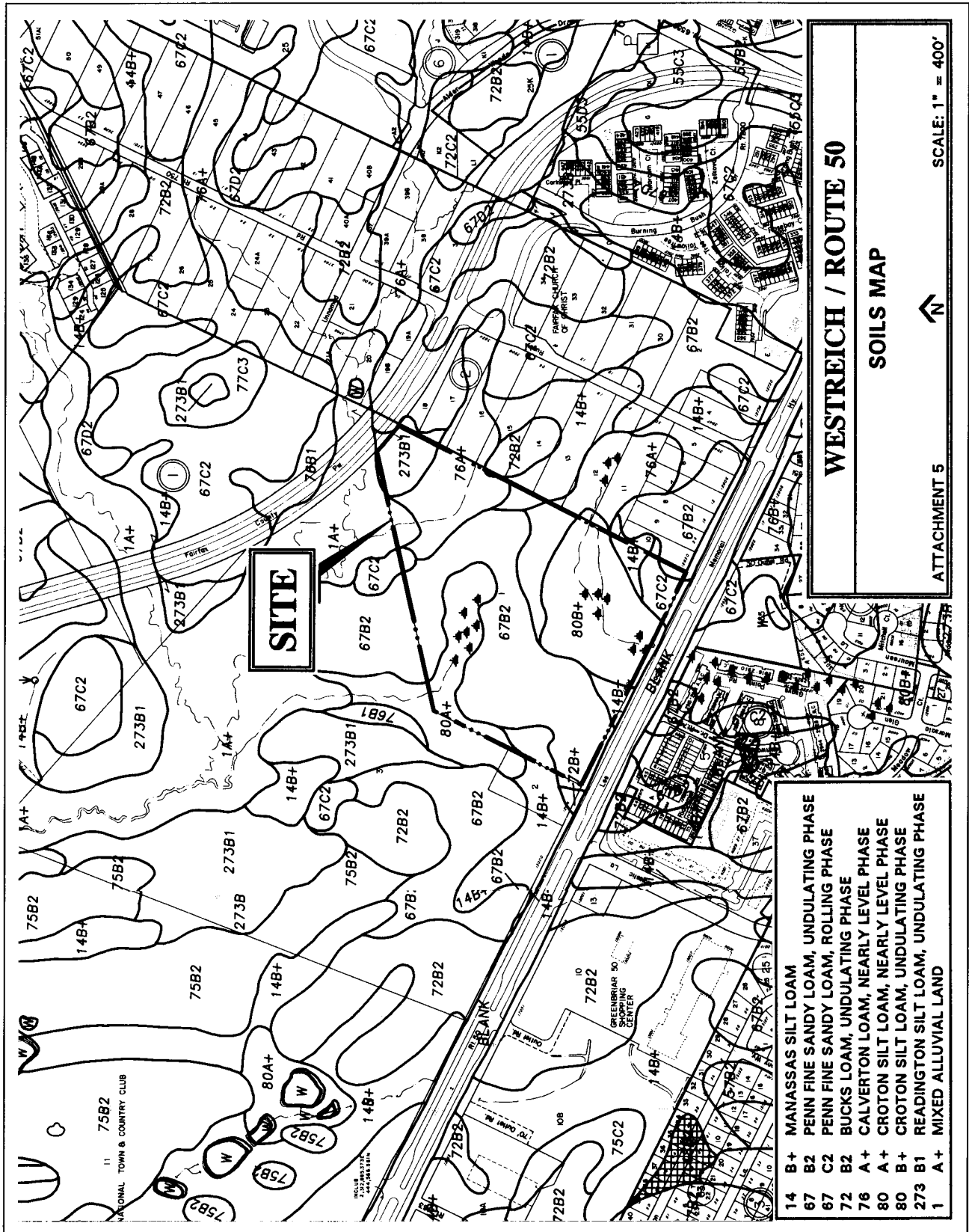


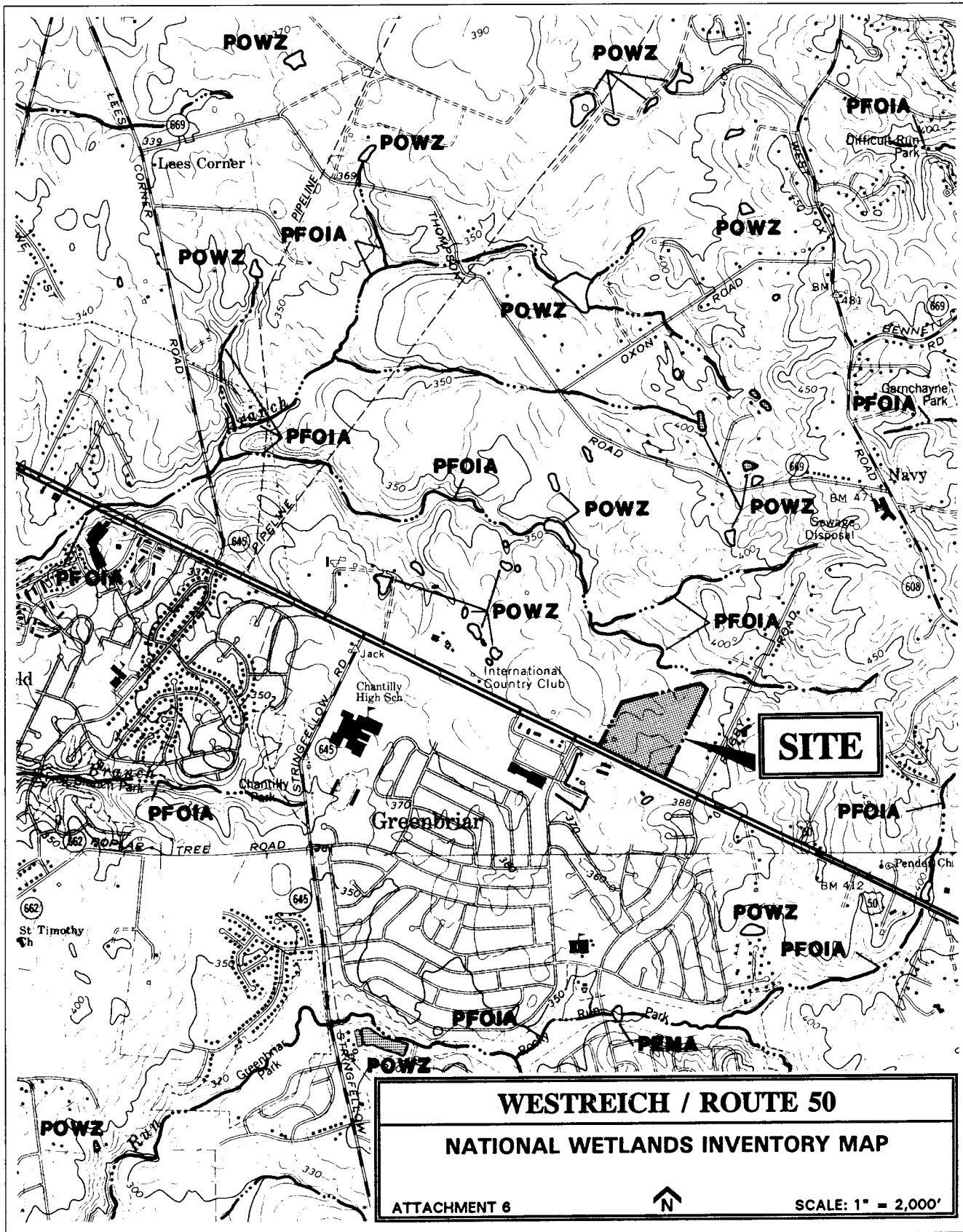


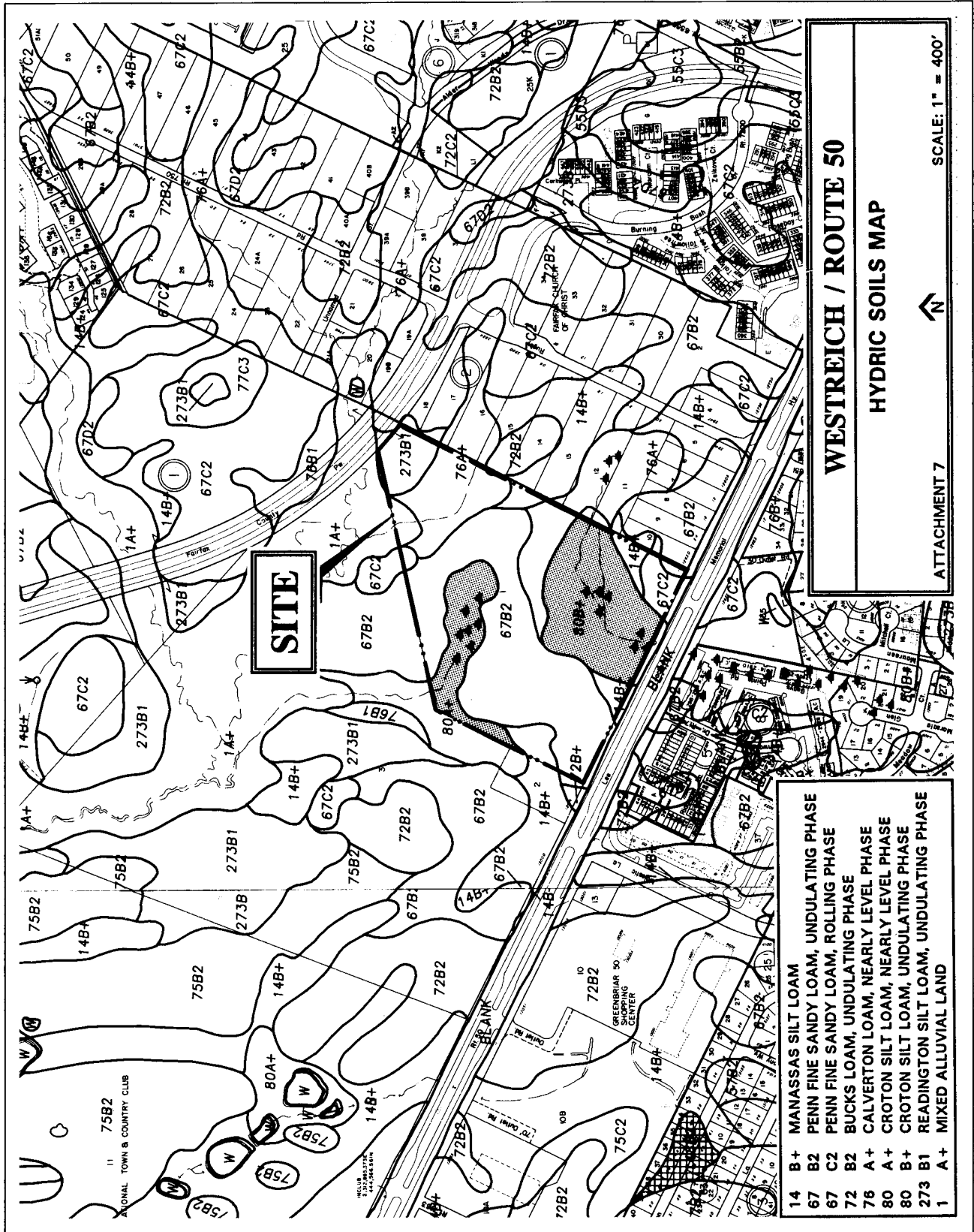


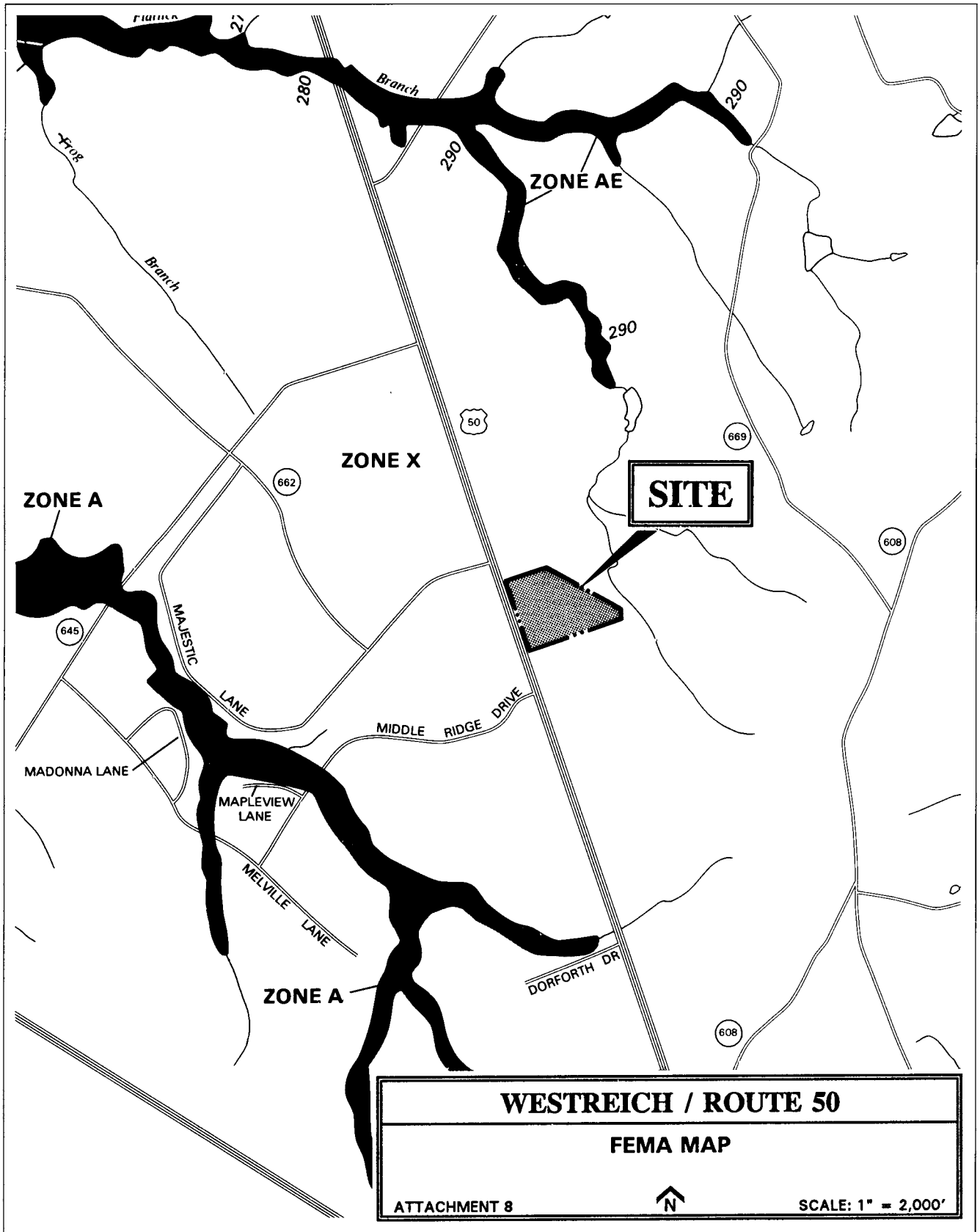


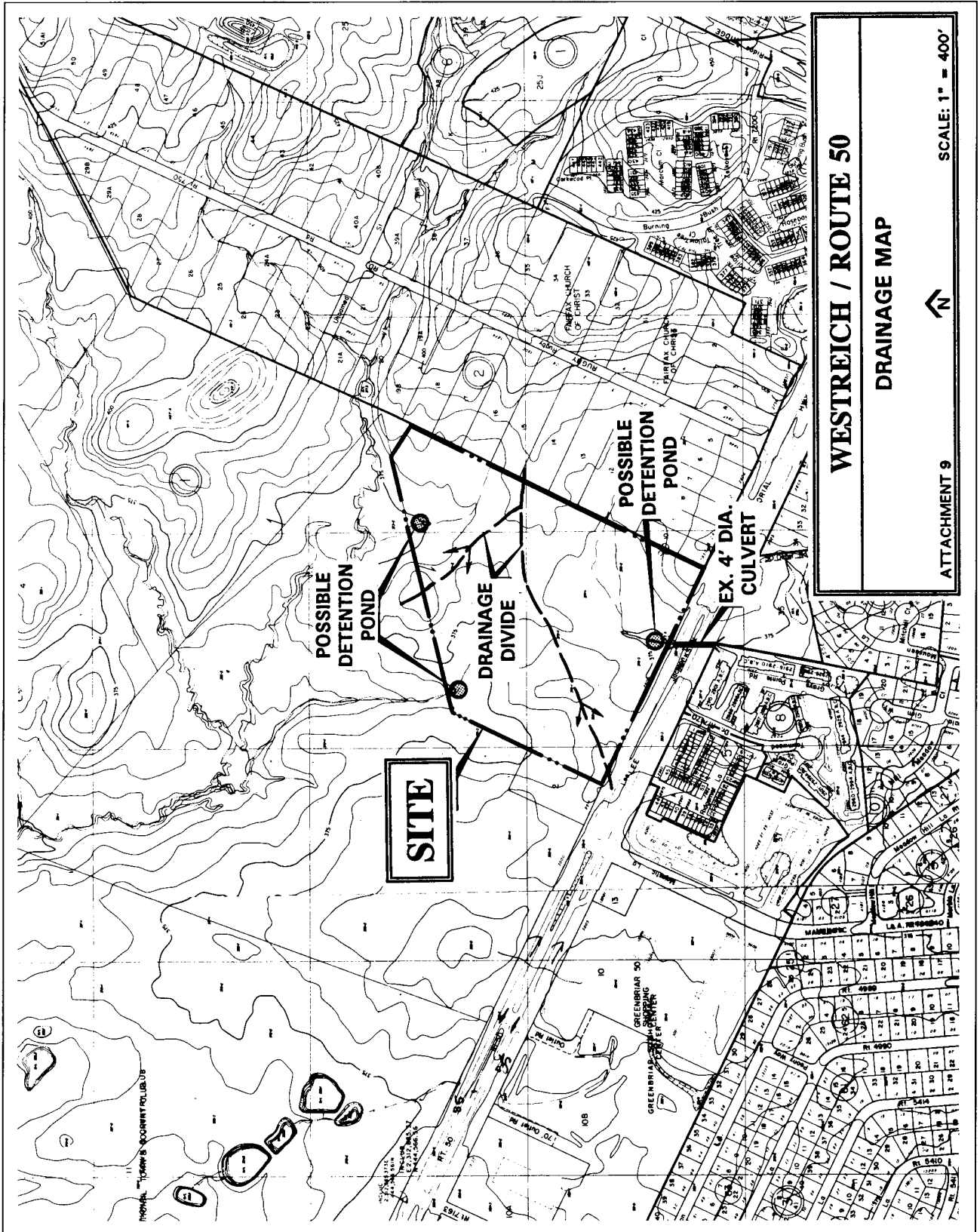


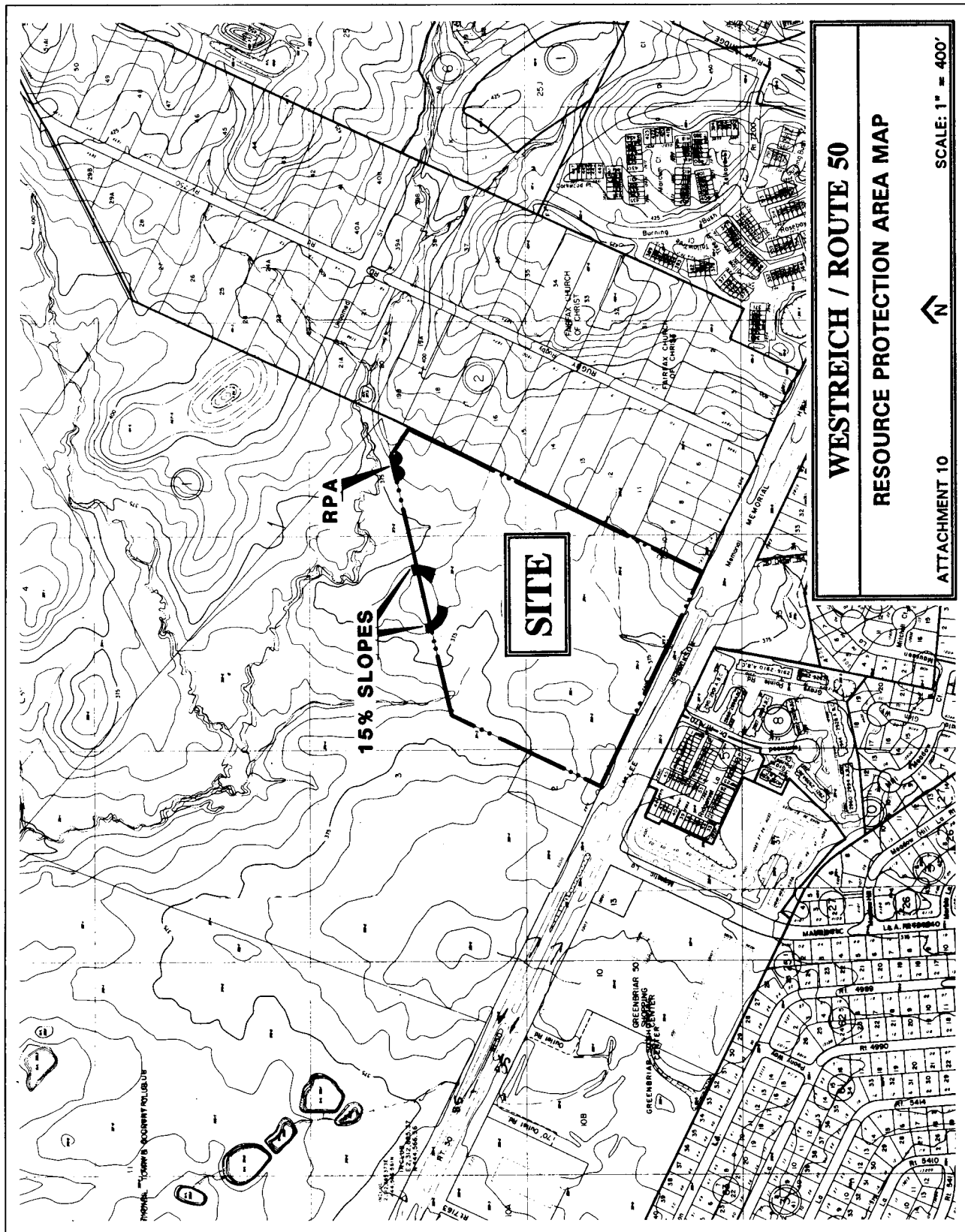












**REFERENCES**

Galaty, Fillmore W., Wellington J. Allaway, and Robert C. Kyle. 1988. *Modern Real Estate Practice*, Real Estate Education Company, Chicago.

**ADDITIONAL READING**

Brown, Thomas L., *Site Engineering for Developers and Builders*, National Association of Home Builders, Washington, D.C., 1988.

De Chiara, Joseph, and Lee E. Koppelman, *Site Planning Standards*, McGraw-Hill, New York, 1978.

National Association of Home Builders (NAHB), *Land Development*, NAHB, Washington, D.C., 1987.

National Association of Home Builders, (NAHB), *Land Development Manual*, NAHB, Washington, D.C., 1974.

Northern Virginia Builders Association, *The Basics of Land Acquisition*, Northern Virginia Builders Association, Fairfax, Va., 1985.

Northern Virginia Building Industry Association and Virginia Bankers Association, *Guidelines for Environmental Site Assessment*, Fairfax, Va., 1990.

Urban Land Institute (ULI), *Residential Development Handbook*, ULI, Washington, D.C., 1978.





## CHAPTER 7

# ENVIRONMENTAL REGULATIONS

Jeffrey M. Moran, P.E.

### INTRODUCTION

Land development is faced not only with the issue of competing with the demands of human needs but also with that of meeting the requirements of environmental restoration and preservation. Our past actions as a society have resulted in significant impacts to the environment due to human consumption and the mismanagement of our natural resources and hazardous waste. To prevent continued degradation of the environment, environmental regulations were established to ensure that everyone acts responsibly toward the protection and maintenance of the environment. From the perspective of the developer and the engineer, these regulations impact real estate transactions, demolition, restoration, and development. Therefore, understanding the basis and objectives of environmental issues can prevent unanticipated delays and conflicts with future development. The old adages that “What you don’t know won’t hurt you” and “Ignorance is bliss” definitely do *not* apply to environmental issues and the land development process. Environmental issues must be proactively addressed and dealt with, often with agencies, procedures, and individuals outside the traditional development process. For the most part, the earlier in the development process that these issues are identified, the better, since many of them can involve liability or even affect the purchase of property, not to mention the development of it.

The intent of this chapter is to provide the reader with an overview of how regulations are codified and what environmental regulations currently exist, describe their purpose, and briefly address their relationship with other regulations. Some regulatory requirements are inexpensive to comply with and take less than a month to perform,

while other requirements can stop development for years and take millions of dollars to complete. This chapter briefly describes the various regulations that a developer should be aware of in developing a property. The information provided in this chapter is not intended to be an exhaustive or all-inclusive reference of environmental regulations. However, note that entire volumes have been written on many of these topics and specialists in various fields have found a niche in the industry to deal with these issues from the identification, mitigation, or regulatory perspectives. While many of the environmental regulations are under federal law, these and many others are addressed by a myriad of state and local regulations and agencies. The reader must bear this in mind and become familiar with the agencies and regulations at all levels that may affect development in a particular locale.

### Federal Laws and Rulemaking

Most laws passed by Congress are statements of policy broadly outlining what the law prohibits and allows, as well as the penalties for noncompliance. These laws are found in the *United States Code* (U.S.C.), Public Law (P.L.), and *Code of Federal Regulations* (CFR). The Executive Branch department or agency assigned responsibility by the Congress for enforcement of the law develops rules or regulations designed to implement the legislation. The rules or regulations describe the allowed and prohibited activities, detail procedures for permitting, and document the consequences of violation. Initially, the rule is offered as a proposed rule and published in the *Federal Register* (FR). The FR is published daily by the federal government and also includes announcements and notices of federal government activities. Usually, 60 days are provided as a comment pe-

riod following publication of the proposed rule. In addition, a public hearing is held upon the request of five or more people. In considering the comments received by the public, minor changes can be made to the rule before it is signed by a department head and published in the FR as a final rule (i.e., promulgation). A promulgated final rule can be changed through published amendments, technical amendments and clarification notices.

The basic format of a rule consists of the:

- *Heading*, which gives descriptive information such as who published the rule, the regulations affected and a descriptive title of the program addressed
- *Preamble*, which explains the purpose and intent of the regulation, who is affected, and why the rule is being written as it is and additionally includes effective dates, addresses, and contacts for additional information
- *Text*, which details the regulations

All final regulations of the federal government are published in the CFR. The CFR is organized into Titles. The Titles are divided into Chapters, the Chapters into Parts, and the Parts into Subparts. An example is the regulations the U.S. Environmental Protection Agency (EPA) has promulgated. They are under Title 40, Protection of Environment, Chapter I, Parts 1–799. The CFR, published yearly, and the *Federal Register* are available at most large libraries and on the Internet by simply searching for the home page of the responsible Agency and then searching for “Regulations,” typically linked to the agency’s homepage.

## ENVIRONMENTAL REGULATIONS

Environmental regulations are the mechanism for implementing the intent of the various acts and statutes developed to protect human health and the environment. One of the challenges for developers is that these regulations are not only extremely diverse, but they are also continually being updated and revised. Therefore, a developer must take the necessary steps to ensure compliance in order to minimize liability and avoid unanticipated costs and delays associated with the purchase and/or development of a property.

A brief summary of environmental regulations over the last three decades can help put into perspective the progression of environmental regulations and their future focus. The U.S. EPA was established in 1970 to facilitate coordinated and effective governmental action designed to ensure the protection of the environment by abating and controlling pollution on a systematic basis. Consequently, the 1970s were a decade of extensive new federal legislation covering all aspects of environmental issues. The 1980s emphasized refining existing legislation and enforcement policies. The 1990s emphasized balancing economic and environmental costs. The future of environmental regulations is unknown.

There are numerous issues facing the current regulations, and it is reasonable to expect changes to be made. For example, there is speculation by some of the policy makers that effective environmental regulations need to focus on good engineering and science to achieve the desired objectives versus cleanup standards and laboratory detection limits. There is also concern within certain industries that increased restrictions on the discharge of pollutants may have significant economic impacts while achieving little benefit to human health and the environment.

Developers are faced with issues such as smart growth versus urban sprawl. One of the challenges is that both have environmental impacts. Urban sprawl threatens to reduce the limited amount of undisturbed/undeveloped land impacting our natural resources and the increased traffic volume results in more pollution. Conversely, smart growth has the potential to increase noise pollution and cause higher pollutant loads in a more concentrated area. These factors, and many more, are issues that a developer has to deal with in order to strike a balance between the demands of human needs and environmental regulations.

Many of the environmental statutes that work together to address hazardous waste problems and the protection of our natural resources are outlined in Table 7.1, which provides a chronological listing of the regulations. The environmental regulations most likely to have financial implications and project impacts to a developer are the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) which focuses on response to existing environmental conditions, the Resource Conservation and Recovery Act (RCRA), which regulates the management of hazardous waste, and the Toxic Substance Control Act (TSCA) which regulates the manufacturing and use of toxic waste. Table 7.1 cites references for the U.S.C. and *Public Law*. Actual references to specific CFRs can be found in specific sections describing these regulations later in this book. These Acts can be further researched by accessing the Congressional Research Service through a website maintained by the National Council for Science and the Environment: <http://www.cnie.org>.

### Fish and Wildlife Coordination Act

The objective of this act is to provide wildlife with equal consideration as other water resource development programs. It establishes the need to coordinate activities of federal, state, and private agencies in the development, protection, and stocking of wildlife resources and their habitat. This act authorizes the preparation of plans to protect wildlife resources. Any report recommending authorization of a new project must contain an estimate of wildlife benefits and losses and the costs and amount of reimbursement required.

### Federal Insecticide, Fungicide, and Rodenticide Act (FIFRA)

FIFRA regulates the use and safety of toxic pesticides that are produced in order to reduce the amount of waste that

TABLE 7.1 Examples of Environmental Statutes

NAME	YEAR	MAJOR AMENDMENTS	REGULATORY REFERENCE	
			U.S.C.	PL
Fish and Wildlife Coordination Act	1934	1946, 1958, 1965	16 U.S.C. 661 <i>et seq.</i>	P.L.
Federal Insecticide, Fungicide, and Rodenticide Act (FIFRA)	1947	1972, 1978, 1988, 1991	7 U.S.C. 136 <i>et seq.</i>	P.L. 80-104
Clean Water Act (CWA)	1948	1956, 1972, 1977, 1987	33 U.S.C. 1251 <i>et seq.</i>	P.L. 80-845
National Historic Preservation Act (NHPA)	1966	1973, 1976, 1980, 1992	16 U.S.C. 470-470t	P.L.
Wild and Scenic Rivers Act	1968	1974	16 U.S.C. 1271 <i>et seq.</i>	P.L.
National Environmental Policy Act (NEPA)	1969 /70	—	42 U.S.C. 4341 <i>et seq.</i>	P.L.
Clean Air Act (CAA)	1970	1970, 1977, 1990	42 U.S.C. 7401 <i>et seq.</i>	P.L. 84-159
Coastal Zone Management Act (CZMA)	1972	1976, 1980, 1985, 1990, 1996	16 U.S.C. 141 <i>et seq.</i>	P.L.
Endangered Species Act	1973	1978, 1982, 1988	16 U.S.C. 1 531-1 542	P.L.
Safe Drinking Water Act (SDWA)	1974	1977, 1986, 1996	42 U.S.C. 300f <i>et seq.</i>	P.L. 93-523
Hazardous Material Transportation Act	1990		49 U.S.C.1801–1819 5 U.S.C. 553	P.L.
Marine Protection, Research, and Sanctuaries Act	1972	1984, 1988	33 U.S.C. 1401 <i>et seq.</i>	P.L.
Toxic Substances Control Act (TSCA)	1976	1986, 1988	15 U.S.C. 2601 <i>et seq.</i>	P.L. 94-469
Resource Conservation and Recovery Act (RCRA)	1976	1984, 1992	42 U.S.C. 6901 <i>et seq.</i>	P.L. 89-272, title II
Noise Control Act	1972	1976, 1978	42 U.S.C. 4901 <i>et seq.</i>	P.L.
Comprehensive Environmental Compensation and Liability Act (CERCLA)	1980	1986 (SARA)	42 U.S.C. 9601 <i>et seq.</i>	P.L.-96-510
Superfund Amendment Reauthorization Act (SARA)	1986	—	42 U.S.C. 11001 <i>et seq.</i>	P.L. 96-510
Pollution Prevention Act	1990	—	42 U.S.C. 13101–13109	P.L. PL 101-508 Title VI

needs to be managed. The objective of FIFRA is to ensure that the impacts to the environment resulting from the use of pesticides do not outweigh the benefits.

### Clean Water Act (CWA)

The objective of the CWA is to restore and maintain the chemical, physical, and biological integrity of our nation's

water. One of the major focuses for the national water program is to control point discharge of pollutants to water through total maximum daily loads (TMDLs), which regulate the amount of pollutants permitted to be discharged to waters of the United States. Water supply and water pollution issues also include nonpoint controls and effective use of water resources management practices.



**FIGURE 7.1** Airplane spraying crops with pesticides.

Sludge resulting from wastewater treatment and pretreatment under CWA must be handled at a RCRA facility if it is hazardous. Discharges from a RCRA-permitted facility must be pursuant to a National Pollutant Discharge Elimination System (NPDES) permit. This means that either the facility has obtained a NPDES permit or the waste meets CWA pretreatment standards and has been transported to a privately owned treatment works.

### **National Historic Preservation Act (NHPA)**

The NHPA establishes a national policy of preserving, restoring, and maintaining cultural resources. The national register of historic places protects historic properties. Cultural assessments under the National Environmental Policy Act are now part of the environmental assessment. New projects can be affected on the grounds of the potential adverse affects in terms of character, scale, or style of the historic district.

### **Wild and Scenic Rivers Act**

This act designates and protects rivers for their wild and scenic values from activities that may adversely impact those values. It describes procedures and limitations for control of lands in federally administered components of the system and for dealing with deposition of lands and

minerals under federal ownership. Rivers are classified as wild, scenic, or recreational, and hunting and fishing are permitted in components of the system under applicable federal and state laws.

### **National Environmental Policy Act (NEPA)**

NEPA is a national policy that takes into consideration the preservation of the natural environment and human needs caused by population increases, high-density urbanization, and industrial expansion by taking into account economical and technical considerations. This act was one of the first laws that established the broad national framework for protecting our environment. The NEPA process is intended to help public officials make decisions that are based on understanding the environmental consequences associated with a proposed action by a federal agency. NEPA requirements are invoked when airports, buildings, military complexes, highways, parkland purchases, and other federal activities are proposed.

The objective of NEPA is to have a systematic approach to ensure that appropriate actions are taken to protect, restore, and enhance the environment. It is the planning and decision-making process of federal agencies. NEPA requires federal agencies to assess the environmental impact of implementing their major programs and actions early in the

planning process. It provides a systematic means of dealing with environmental concerns and the associated costs. One of the unique components of NEPA is that it provides a mechanism for public participation.

Prior to a federal agency implementing a proposed action, it must perform an environmental assessment to identify and address:

- Any environmental impacts of the proposed action
- Any adverse environmental affects that cannot be avoided
- Alternatives to the proposed action
- The relationship between local short-term uses of man's environment and the maintenance and enhancement of long-term productivity
- Any irreversible and ir retrievable commitments of resources that would be involved in the proposed action should it be implemented

### **Clean Air Act (CAA)**

Some of the major factors that contribute to air pollution are urbanization, industrial development, and the increasing use of motor vehicles. Air pollution poses a danger to public health and welfare, including our agricultural crops and livestock. The CAA focuses on the reduction or elimination of the amount of pollutants produced or created. Air quality is an international issue because polluted air can migrate across national borders, impacting the environment and human health throughout the world. Therefore, air quality requires participation from all countries. Given the highly diverse economic and political climates of all the countries throughout the world, this is an extremely challenging issue to resolve.

The law recognizes that states should take the lead in carrying out the CAA because pollution control problems often require special understanding of local industries, geography, housing patterns, and other factors. Therefore, states are required to develop state implementation plans (SIPs) that explain how they will carry out the CAA. An SIP is a collection of the regulations a state will use to clean up polluted areas. The states must involve the public, through hearings and opportunities to comment, in the development of each SIP.

Legislation and regulations will continue to evolve to address air pollution, such as global warming, ozone depletion, acid rain, and indoor air quality. The EPA proposed new National Ambient Air Quality Standards (NAAQS). Some of the issues surrounding air quality are impacts to human health and the effects on the cost to the industries. Air emissions from incinerators and other types of transfer storage and disposal facilities regulated under RCRA must comply with applicable ambient standard and/or emission limitations of the CAA. Extraction of pollutants from air emissions under CAA controls can create hazardous sludges

containing such wastes. Disposal of these materials must comply with RCRA.

### **Coastal Zone Management Act (CZMA)**

The CZMA is a federal program that establishes standards to protect the coastal resources that are determined to be of national significance. These resources include coastal areas, wetlands, floodplains, estuaries, beaches, dunes, barrier islands, coral reefs, and fish and wildlife and their habitats. The CZMA also protects these coastal areas by managing development.

### **Endangered Species Act**

The objective of this act is to conserve threatened and endangered species. The Fish and Wildlife Service of the U.S. Department of the Interior promulgates a list of threatened and endangered species and their critical habitat. Federal agencies are required to carry out programs for the conservation of the threatened and endangered species and must take actions to ensure that projects they authorize, fund, or carry out are not likely to jeopardize the existence of the listed species or result in the destruction or modification of their habitat that is declared to be critical.

### **Safe Drinking Water Act (SDWA)**

An estimated 50% of the U.S. population uses groundwater as a source of drinking water. The SDWA protects the nation's drinking water supply by setting drinking water standards, known as Maximum Contaminant Levels (MCLs), and regulating underground injection wells. Both the MCLs and the Underground Injection Control (UIC) program are especially relevant to RCRA. The MCLs are sometimes used by RCRA as cleanup standards for corrective actions.

### **Marine Protection, Research, and Sanctuaries Act**

This act regulates dumping of all types of material into the ocean and designates certain areas of the ocean waters as sanctuaries. The EPA is responsible for issuing permits for the dumping of materials in ocean waters except for dredged material, which is regulated by the Corps of Engineers. Additionally, the EPA will not issue a permit for the dumping of any radiological, chemical, biological warfare agents, or high-level radioactive waste.

### **Toxic Substances Control Act (TSCA)**

TSCA requires manufactures to conduct tests on chemicals to determine whether they have adverse affects on human health or the environment. The risks to human health include persistence, acute toxicity, and carcinogenic effects. This act authorizes the tracking of more than 75,000 industrial chemicals currently produced or imported into the United States. It controls the disposal methods of certain chemicals such as polychlorinated phenols and the inspection and removal of asbestos products from public schools. It also studies the public health danger posed by asbestos in public and commercial buildings.



**FIGURE 7.2** Industry is a source of air pollution.

This act places the burden of testing chemical substances as related to human health and the environment on the chemical manufacturers.

### **Resource Conservation and Recovery Act (RCRA)**

The primary purpose of RCRA is to protect human health and the environment from dangers of hazardous waste. RCRA regulates the management of hazardous waste, provides technical and financial assistance for the development of management plans, provides facilities for the recovery of energy and other resources from discarded materials, and provides for the safe disposal of discarded materials. RCRA focuses on the management of wastes from the time it is generated until it is disposed of, cradle to grave. Unlike CERCLA, RCRA does not provide for an innocent landowner exemption from liability.

Hazardous waste is defined by its characteristic or origin. Waste that exhibits the characteristics of being ignitable, reactive, corrosive, toxic, or radioactive are considered hazardous. In addition, chemical products involved in a particular waste stream are also considered hazardous. The basis for listing the types or classes of wastes is indicated by the use of one or more of the hazard codes. The hazard codes include ignitable, corrosive, reactive, toxicity characteristic waste, acute hazardous waste, and toxic waste. The chemicals involved in these processes, referred to as

listed wastes, are categorized into four groups, based on the specific process by which they are derived: hazardous waste from nonspecific sources; hazardous waste from specific sources; discarded commercial chemical products, off-specification species, or manufacturing chemical intermediates identified as acute hazardous waste; and discarded commercial chemical products, off-specification species, or manufacturing chemical intermediates identified as hazardous wastes.

### **Noise Control Act**

Noise pollution is a result of human activity and is one of the most pervasive environmental problems. It is not only a public health hazard causing hearing impairments, but a nuisance causing psychological stress. The major sources of community noise are construction and transportation. As development increases, exposure to these noises increases. The objectives of the Noise Control Act are to establish noise emission standards for new products, utilize noise abatement controls, and label products to increase awareness and protection against individual exposure.

### **Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA)**

The primary purpose of CERCLA is to protect human health and the environment from the dangers of hazardous

waste. CERCLA focuses on response to existing environmental conditions of a property and the liability of the responsible parties. CERCLA created a revolving fund, commonly referred to as Superfund, that can be utilized by EPA, state, and local governments to investigate and remediate hazardous waste sites that have been listed by the EPA on the National Priorities List. One of the unique features of CERCLA is that governmental and private parties who are not responsible for the investigation and remediation of a property can perform the work using the Superfund and seek reimbursement from the responsible parties. CERCLA covers all environmental media—air, surface water, groundwater, and soil—and can apply to any type of facility. If cleanup is conducted on-site under the CERCLA program, no federal, state, or local permit is required.

A CERCLA hazardous substance includes any substance that the EPA has designated for special considerations under RCRA, CAA, CWA, and TSCA and any substance that presents a substantial danger to human health and the environment. Exclusions from CERCLA's hazardous substance list include petroleum and natural gas and synthetic gas used for fuel. Due diligence and liability fall under CERCLA and will likely be the most applicable regulation to land development.

Old industrial sites that are contaminated but have the potential for redevelopment are defined as brownfields. Re-

development of these sites falls under CERCLA regulations. There is an interest in the redevelopment of these contaminated sites from an environmental and public policy perspective. To encourage developers to purchase and redevelop such properties, which they otherwise would not consider because of their liability under CERCLA, the government for some cases may limit liability and provide financial incentives.

### **Superfund Amendments and Reauthorization Act (SARA)**

This act is an amendment to CERCLA and provides additional authorization and amendments to CERCLA. SARA establishes requirements for reporting on-site hazardous/toxic materials and/or substances under Title III, the Community-Right-to-Know Act. This act establishes requirements for notification of uncontrolled releases.

### **Pollution Prevention Act**

The Pollution Prevention Act established a national policy for waste management. This includes prevention, recycling, treatment, and disposal of wastes. The objective of this act is to prevent or reduce pollution at the source, wherever feasible. Where pollution cannot be prevented, it should be recycled in an environmentally safe manner. In the absence of feasible prevention and recycling, pollution should be



**FIGURE 7.3** Recycling is a part of pollution prevention.



treated to applicable standards prior to release or transfers. Only as a last resort are wastes to be disposed of.

### **Occupational Safety and Health Act**

As important as the above acts are in protecting health and the environment, it is equally important to protect the workers who are working to mitigate the environmental problems. The Occupational Safety and Health Act of 1970 (29 U.S.C. 651 *et seq.*) contains provisions that protect workers performing environmental cleanups. The federal agency, the Occupational Safety and Health Administration (OSHA), is the governing agency for worker protection.

While the developer is not directly affected, it is important that the developer understand the consequences of not adhering to the regulations or performing illegal cleanups using unprotected workers who have not received the proper training and certifications. All contractors performing work for the developer should be thoroughly evaluated to ensure that they, as a business course, always are in full compliance with all environmental and OSHA regulations.

### **ADDITIONAL READING**

Corbitt, Robert A, *Standard Handbook of Environmental Engineering*, 2d ed., McGraw-Hill, New York.

*Virginia Administrative Code*, 9 VAC 20-60-100

## CHAPTER 8

# ENVIRONMENTAL SITE ASSESSMENTS

Jeffrey M. Moran, P.E.

### INTRODUCTION

The terms *environmental site assessment* (ESA) and *environmental assessment* (EA) are commonly used to describe the due diligence surveys performed under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) or for the studies performed under the National Environmental Policy Act (NEPA). Despite their similarity in title, the objective and purpose of the two studies are quite different. In simple terms, environmental site assessments, performed for due diligence surveys under CERCLA, focus on existing or prior impacts to a property prior to a real estate transaction to document potential liability associated with the presence (or likely presence) of any hazardous substances or petroleum products under conditions that indicate either an existing release, a past release, or a material threat of release of any hazardous substances or petroleum products into structures on the property or into the ground, groundwater, or surface water of the property. Whereas the environmental assessment called for within NEPA involves the review of a proposed action and alternatives to define and evaluate the possible effects of the proposed project on the environment. Specifically, the environmental assessment performed under NEPA must consider the environmental impact(s) of a proposed action, while also considering any adverse environmental effects that cannot be avoided should the action be implemented. Additionally, it must identify alternatives to the proposed action, including a “no-action” alternative. The assessment will also evaluate the relationship between short- and long-term benefits and consequences.

Another difference between the Phase I ESA and the EA is that EAs and associated documents are made available to

the public for review and comment. This is an important part of the process because of the issues that can be raised by the public and the potential delays associated with the comment and response period.

Developers could be required to perform an environmental study under either CERCLA or NEPA. Although they are more likely to be faced with the requirements of a Phase I ESA as a due diligence study under CERCLA, both types of environmental assessments are described in this chapter.

### ENVIRONMENTAL ASSESSMENT UNDER CERCLA—DUE DILIGENCE

In the field of environmental due diligence, three generally distinct, yet interrelated evaluations are currently utilized to determine the potential for past, present, and future contamination on a property: environmental assessments, environmental compliance audits, and environmental risk assessment surveys. Although this chapter focuses on environmental assessments, all three types are briefly described below:

- The environmental assessment is generally performed to provide the property owner, purchaser, or lender with documentation relative to the “Innocent Buyer (Landowner)” provision of CERCLA.
- The environmental compliance Audit, which deals with a company’s policies, procedures and operations, is conducted to determine sources of potential environmental liabilities and areas of noncompliance.
- The environmental risk assessment survey is utilized to reduce future environmental liabilities that

could result from a company's operations and materials management.

The environmental assessment is an integral part of site feasibility and is critical in minimizing a client's environmental liability exposure. Since both the environmental compliance audit and the risk assessment survey are associated with management and operation of ongoing business activities, more specific discussions will not be considered in this chapter.

Due to the impacts on the environment, federal and state governments have enacted legislation requiring the cleanup of conditions that pose a risk, or threat of risk, to human health or the environment. These laws have been promulgated to prosecute any party deemed to be a potentially responsible party (PRP). This has far-reaching implications because the definition of a PRP is not limited to those parties responsible for the actual contamination, but also includes owners: past, present, or subsequent; lessors, managers; lienholders, transporters; and other parties having ownership or management responsibility. Furthermore, there is no standard rule of assigning responsibility for a liability.

### PHASE I ESA

Other federal, state, and local environmental laws may impose environmental assessment obligations that are beyond the scope of a Phase I ESA. In addition, there are likely to be other legal obligations concerning hazardous substances or petroleum products discovered on property that are not addressed in this practice and may pose risks of civil and/or criminal sanctions for noncompliance. There are many assessments that are called ESAs with different scopes of work. Currently, the standard of care in the industry is the American Society for Testing and Materials (ASTM) Standard E-1527-2000, Standard Practice for Environmental Site Assessments: Phase I Environmental Site Assessment Process.

The objective of a Phase I environmental site assessment study is to identify, to the extent feasible, recognized environmental conditions in connection with the property. "Recognized environmental conditions," as defined by ASTM, means "the presence or likely presence of any hazardous substances or petroleum products on a property under conditions that indicate an existing release, a past release, or a material threat of release of any hazardous substances or petroleum products into structures on the property, or into the ground, groundwater, or surface water of the property" (ASTM E-1527-2000). The term is not intended to include conditions that generally do not present a material risk of harm to human health or the environment and would not be the subject of an enforcement action if they were brought to the attention of the appropriate governmental agencies.

Phase I ESAs, which are performed as part of commercial real estate transactions, are very common. These site assessments are prepared to demonstrate due diligence in evaluating the property using good commercial and customary practice for environmental site assessments for com-

mercial real estate. This practice is intended to allow an owner to satisfy one of the requirements in order to qualify for the innocent landowner defense to CERCLA liability. An environmental site assessment should be performed very early in the development planning stage to avoid stoppage of work or delays associated with environmental conditions at the property. Environmental professionals with the proper training and experience should perform these assessments. ASTM provides guidance to assist users in the preparation for, and selection of, an environmental professional to conduct a Phase I ESA. The technical competence of the environmental professional and the firm's quality assurance/quality control program, internal risk management program, experience, standard terms and conditions, and errors and omissions liability insurance policy should be evaluated prior to retaining the services.

Once a property is identified for which obtaining a Phase I ESA is being considered, certain information should be collected, if available, prior to the selection of an environmental professional. This information should be provided to the environmental professional selected to conduct the Phase I ESA.

- Why the Phase I ESA is required
- Type of property
- Type of transaction
- Complete address
- Activity and use limitations (liens, deed restrictions, etc.)
- Property value relative to comparable properties (reduction due to environmental conditions)
- Desired scope of services (standard Phase I ESA or services beyond requirements in ASTM Practice E 1527)
- Parties relying on Phase I ESA report
- Site contact
- Special terms and conditions
- Knowledge of or experience with the property pertinent to environmental conditions

A Phase I ESA has four components: records review, site reconnaissance, interviews, and report. The record review is performed to obtain and review records that will help identify recognized environmental conditions in connection with the property. This includes review of records for hazardous substances and petroleum products for areas outside the property that are likely to migrate onto the property. The information for the record research is typically obtained through a variety of public and other database sources. The site reconnaissance is performed to observe visually and physically the site and adjoining properties for evidence of recognized environmental conditions. If the property has

<u>TABLE OF CONTENTS</u>	
I.	<u>Introduction</u>
	A. Purpose
	B. Detailed Scope-of-Services
	C. Significant Assumptions
	D. Limitations and Exceptions
	E. Special Terms and Conditions
	F. User Reliance
II.	<u>Site Description</u>
	A. Location and Legal Description
	B. Site and Vicinity General Characteristics
	C. Current Use of the Property
	D. Descriptions of Structures, Roads, Other Improvements on Site (including heating/cooling system, sewage disposal, source of potable water)
	E. Current Uses of the Adjoining Properties
III.	<u>User Provided Information</u>
	A. Title Records
	B. Environmental Liens or Activity Use Limitations
	C. Specialized Knowledge
	D. Valuation Reduction for Environmental Issues
	E. Reason for Performing Phase I ESA
	F. Other
IV.	<u>Records Review</u>
	A. Standard Environmental Record Sources
	B. Additional Environmental Record Sources
	C. Physical Setting Source(s)
	D. Historical Use Information on the Property
	E. Historical Use Information on Adjoining Properties
V.	<u>Site Reconnaissance</u>
	A. Methodology and Limiting Conditions
	B. General Site Setting
	C. Exterior Observations
	D. Interior Observations
VI.	<u>Interviews</u>
	A. Interview with Owner
	B. Interview with Site Manager
	C. Interview with Occupants
	D. Interview with Local Government Officials
	E. Interview with Others
VII.	<u>Findings</u>
VIII.	<u>Opinion</u>
IX.	<u>Conclusions</u>
X.	<u>Deviations</u>
XI.	<u>Additional Services</u>
XII.	<u>References</u>
XIII.	<u>Signature(s) of Environmental Professional(s)</u>
XIV.	<u>Qualifications(s) of Environmental Professional(s)</u>
XV.	<u>Appendices</u>
	A. Site (Vicinity) Map
	B. Site Plan
	C. Site Photographs
	D. Historical Research Documentation (aerial photographs, fire insurance maps, historical topographical maps, etc ...)
	E. Regulatory Record Documentation
	F. Interview Documentation
	G. Special Contractual Conditions between User and Environmental Professional

**FIGURE 8.1** Recommended outline for a typical Phase I ESA.

buildings or structures, the interiors of the buildings are also observed. The interviews conducted as part of the assessment are performed to gather information about the property from those who are most familiar with the current and past uses of the property. Finally, the Phase I ESA report is prepared to document the work performed, findings, opinions, and conclusions. The report format recommended by ASTM is provided in Figure 8.1. An example of a site plan identifying key environmental features is presented in Figure 8.2.

The typical Phase I ESA usually does not include performing environmental testing or evaluations for the following considerations unless specifically requested by the organization paying for the assessment:

- Asbestos
- Lead-based paint
- Lead in drinking water
- Radon
- Wetlands

Because the laws regarding legal responsibility for environmental contamination have had insufficient time for legal precedents to be established, limited safety is available to a buyer, lender, or insurer. In addition, there are no standards, beyond the guidelines outlined by ASTM E-1527, or absolutes for environmental inspections (assessments). Technical guidelines exist on how to perform certain investigative actions; however, no approved or standard level of investigation has been required or adopted in a legal code sense. Subsequently, there is no numerical definition of what is legally considered an acceptable level of environmental risk, only comparative standards (i.e., drinking water standards, risk-based concentration calculations, etc.).

The general cost of typical Phase I ESAs ranges from \$1500 to \$3500, depending on size, location, and type of property. Industrial properties and large parcels of land with numerous recognized environmental conditions could cost significantly more than the “typical” ESA. A third party, composed of quality professionals, should perform these assessments. Oftentimes, lending institutions have lists of approved professionals who perform this work. Regulators may also maintain a list of qualified professionals. Although an environmental site assessment can be technical, a purchaser, developer, and/or lender should be familiar with the evaluation’s findings and conclusions. The ESA report is the proof that a due diligence study was performed to determine the status of the property and therefore should be kept on record.

If environmental concerns are identified in the initial ESA, they are likely to require further investigation, typically referred to as a Phase II ESA. Various environmental testing methods exist that can be used to analyze for the wide variety of contaminants, as required under specific

regulations. Since each potentially contaminated property is unique, a detailed environmental sampling and analysis plan should be developed for each property. For more information on the ASTM environmental site assessment process, visit their website at [www.astm.org](http://www.astm.org).

## PHASE II ESA

A Phase II ESA is performed to investigate further the absence or presence of a suspected contaminant or whether a known release of a contaminant or regulated substance occurred on the property. Additionally, a Phase II ESA may be required in situations other than real estate transactions. For example, if an existing property owner has a known release of a contaminant or regulated substance, that owner is responsible for the release and must take corrective action to assess and mitigate the situation, as necessary.

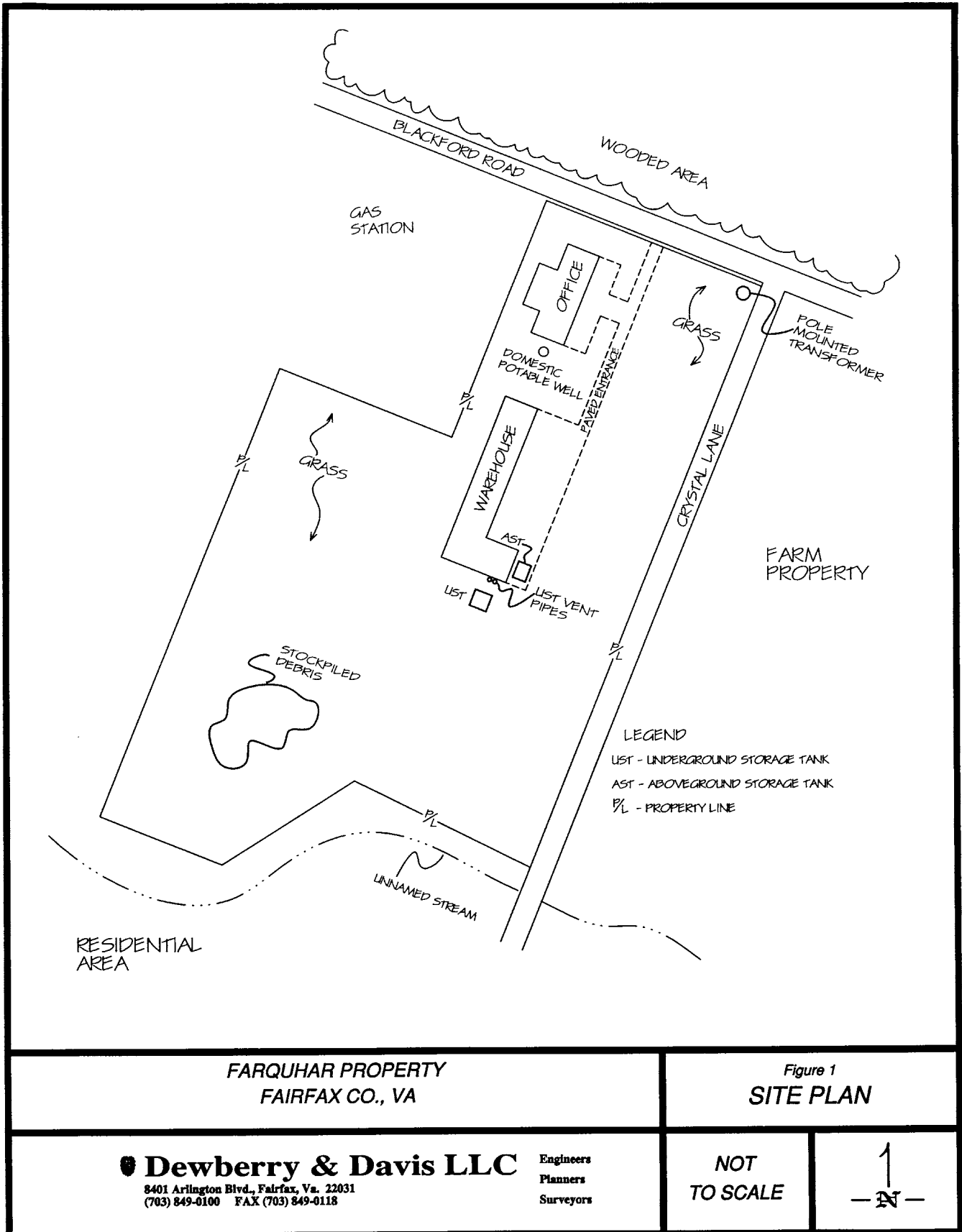
In contrast to the defined scope and relatively low cost of a Phase I ESA, the scope for a Phase II ESA is highly unpredictable, based on the variety of scenarios associated with the release of a contaminant. For example, a Phase II ESA may only require a few samples to be collected from the soil surface to determine if a petroleum product was spilled or released at a specific location. On the other hand, a hazardous waste may have been released with potential impact on the groundwater, air, or surface water, requiring extensive sampling protocol, including the installation of groundwater monitoring wells.

The purpose of a Phase II ESA is to determine the absence or presence of a contaminant. This generally requires environmental sampling. The protocol for collecting and analyzing environmental samples is based on the suspect contaminant, the media, data quality objectives, and the regulatory program requiring the site assessment. The Phase II ESA report addresses the additional research performed, provides a description of the field investigation (including sampling procedures and methodologies and sample locations), and reports the analytical results. If the presence of the suspect contaminant(s) is confirmed, the report provides recommendations for any further actions.

The time frame and cost of Phase II ESAs vary considerably, from a few thousand dollars and a one-month turnaround time to tens of thousands of dollars and a time frame of more than a year. Extreme cases can cost millions of dollars to investigate over a period of several years.

If an environmental testing program is warranted, there are limitations and risks associated with environmental testing that all involved parties must be aware of. In fact, if testing done as part of a Phase II ESA detects contamination, the results of that test have to be reported to federal, state, and/or local agencies. On the other hand, inadequate studies and testing can expose all of the involved parties to legal liability due to negligence.

The main objective of a Phase II ESA is to determine if the suspect contaminant is present on the property. The type(s) and amount of contaminant(s), as well as the property location/site characteristics, all play a role in determin-



**FIGURE 8.2** Typical site plan showing key site features and recognized environmental conditions at and adjacent to site.



**FIGURE 8.3** Removal of leaking underground storage tank.

ing the scope of work for the Phase II ESA. The following list of plans is typically prepared prior to performing a Phase II environmental site assessment for regulatory compliance, worker safety, and quality assurance.

### **Work Plan**

This plan specifies the performance standards, chemicals of concern, sampling and analytical methods required, field procedures, reporting requirements, and schedule. Developing a work plan requires a thorough understanding of the purpose and objective of the assessment, the regulatory requirements, the contaminants of concern, and the sampling methodologies and laboratory analysis appropriate for the assessment.

### **Health and Safety Plan**

In order to identify the hazards and appropriate safety and health controls, the potential hazards on a property must be evaluated prior to entry onto the property. A health and safety plan is required to identify chemical, physical, biological, radiation, flammable/explosive, and confined space hazards to the personnel engaged in operations relating to the site investigation. The plan should be prepared in accordance with applicable Occupational Safety and Health Administration (OSHA) Standard guidelines. In addition,

the personnel performing the work should be trained, as required by OSHA, for certification under 29 CFR 1910.120, Hazardous Waste Operations and Emergency Response Training.

### **Quality Assurance Project Plan**

The integrity of the sampling and analysis performed as part of the ESA is critical to the proper characterization of the contaminants on the property. Since the analytical data are used to determine corrective action alternatives, handling and disposal requirements, and health and safety procedures outlined in the health and safety plan, the analysis must be performed properly and in a timely manner. The quality assurance project plan addresses the specific sampling protocol and analytical procedures required to meet the data quality objectives for a specific project.

### **PHASE III ESA**

Typically, a Phase III ESA is the most expensive phase of environmental site assessments. Once a contaminant is confirmed (Phase I) and suspected to be significant enough to require corrective action (Phase II), a Phase III ESA is performed to determine the extent of contamination and, if required, remedial alternatives. In cases where remediation is required, the Phase II and Phase III ESAs are blended



**FIGURE 8.4** Environmental sampling of subsurface soil.

together. Therefore, the plans as previously prepared for the Phase II ESA will also apply for the Phase III ESA.

The cost and time frame to remediate a site can range from tens of thousands of dollars over several months to several million dollars over several years. The factors that influence cost and time include the type of contaminant, quantity of the release, the media that is impacted, and the location of the release. Because of the high variability of scenarios for Phase III ESAs, they are typically handled on a case by case basis.

As discussed previously, the primary purpose of environmental regulations is to protect human health and the environment. Therefore, the potential exposure scenarios play an important role in the requirements that have been established for corrective action and remediation. Regulators may require periodic monitoring to ensure the conditions do not worsen and present a threat to potential receptors. In other scenarios, regulators may require immediate and thorough remediation to clean up impacted media to remove the immediate threat to receptors. For example, a regulatory agency may require groundwater remediation where less than 50 gallons of gasoline were released on one site and require only groundwater monitoring at another site where over 1000 gallons of gasoline were released. This scenario may apply where the site with the 50-gallon release impacted the groundwater and the 1000-gallon release had no impact to the groundwater and there was no recognized threat to human health or the environment.

There are many different remediation methods and treatment technologies available today, ranging all the way from physical removal and transportation of the contaminant to a treatment facility so that it can undergo incineration (or

chemical treatment) to the *in-situ* (in-place) biological treatment of the contaminant. Whatever method is determined to be the most suitable, it is wise to identify several methods in case your main option does not produce the desired results. Similarly, pilot tests should be conducted for each method before a commitment is made to one particular method. This will likely require the services of an expert in site remediation. Additionally, it is recommended that you confer with the appropriate regulatory entities and get their buy-off on the remedial efforts. Do not forget that contaminated soil may be recyclable or that some facilities may even buy contaminated waste. Hazardous waste site remediation is a very expensive effort. Weigh your options. On-site treatment is rapidly becoming more popular as *in situ* treatment technologies become more available and cost effective.

### NEPA ENVIRONMENTAL ASSESSMENT

NEPA requires an environmental evaluation of major federal actions that could significantly affect environmental quality. While this is most commonly encountered with federally sponsored projects, privately funded or other non-federally sponsored projects that require some form of federal involvement can also be required to undergo the NEPA evaluation process. For example, the granting of a wetland permit by the U.S. Army Corps of Engineers could be a federal action that has a significant effect on environmental quality and thus trigger the NEPA process. Another example is a project that, while privately sponsored, utilizes federal funding in some way, such as U.S. Department of Housing and Urban Development funds to provide affordable dwelling units.



In addition, many states and local agencies have adopted regulations that require an environmental assessment as part of the project authorization or local approval process. Often, these assessments are patterned after the NEPA requirements and involve an evaluation of the existing environmental conditions, potential project impacts, alternatives considered, and ways to mitigate unavoidable project impacts.

Section 15.02(d) of the NEPA regulations requires agencies to provide for the early application of NEPA to cases where actions are planned by private applications or non-federal entities and are, at some stage, subject to federal approval of permits, loans, loan guarantees, insurance or other actions. This is intended to ensure that environmental factors are considered at an early stage in the planning process and to avoid the situation where the applicant for a federal permit or approval has completed planning and eliminated all alternatives to the proposed action by the time the EIS process commences or before the EIS process has been completed. The National Environmental Policy Act of 1969 (NEPA), 42 U.S.C. 4321 et seq., requires that Federal agencies include in their decision-making processes appropriate and careful consideration of all environmental effects of proposed actions and their alternatives for public understanding and scrutiny, avoid or minimize adverse effects of proposed actions, and restore and enhance environmental quality as much as possible.

The initial step in this process is preparation of an Environmental Assessment (EA). The EA is prepared to provide sufficient data and analysis to determine whether there is a Finding of No Significant Impact (FONSI) or that an Environmental Impact Statement (EIS) is required. The EA is a concise public document which has three defined functions:

- Briefly provide sufficient evidence and analysis for determining whether to prepare an EIS,
- Help identify better alternatives and mitigation measures when an EIS is not required, and
- Facilitate the preparation of an EIS when one is required.

The EA typically contains a brief discussion of the need for the proposed action, alternative actions and the environmental impacts of the proposed actions and alternatives. An example format for a typical EA under NEPA is provided in Figure 8.5. An environmental review process is implemented to assure NEPA considerations are taken into account. This review is the process that determines whether a proposed action may have a significant impact on the environment, requiring the preparation of an EIS. An EIS is not required when the EA determines that the proposed action results in a FONSI. An EA or EIS may not be required if the proposed action is “categorically excluded.”

The objective of the EIS is to present the alternatives, evaluate them and determine the preferred alternative. The

## TABLE OF CONTENTS

1.0	INTRODUCTION.....	1
1.1	Purpose and Need.....	1
1.2	Information Sources.....	2
1.3	Limitations.....	3
2.0	PROPOSED ACTION AND ALTERNATIVES.....	4
2.1	No-Action Alternative (Alternative 1).....	5
2.2	Proposed Action (Alternative 2).....	5
2.3	Alternative 3.....	6
2.4	Alternative 4.....	7
3.0	EXISTING SITE CONDITIONS.....	9
3.1	Population Demographics.....	9
3.2	Community Services and Infrastructure.....	10
3.3	Surrounding Residential Communities.....	10
3.4	Environmental Quality.....	11
3.5	Natural Systems.....	11
3.6	Historical and Cultural Resources.....	12
4.0	POTENTIAL ADVERSE ENVIRONMENTAL IMPACTS.....	14
4.1	Impacts on Population Demographics.....	14
4.2	Impacts on Community Services and Infrastructure.....	15
4.3	Impacts on Surrounding Residential Communities.....	15
4.4	Impacts on Environmental Quality.....	15
4.5	Impacts on Natural Systems.....	16
4.6	Impacts on Historical and Cultural Resources.....	17
5.0	ALTERNATIVES ANALYSIS.....	17
5.1	“No Action” Alternative.....	18
5.2	Review of Alternate Building Sites.....	18
5.3	Continue Project as Proposed.....	18
6.0	FINDINGS AND RECOMMENDATION.....	19

**FIGURE 8.5** Typical format for an environmental assessment under NEPA.

preferred alternative is the alternative that fulfills its statutory mission and responsibilities, giving consideration to economic, environmental, technical and other factors. The proposed alternative may or may not be determined to be the preferred alternative upon completion of the EIS.

The EIS requires the evaluation of all reasonable alternatives to determine the impacts the actions would have on the human environment. These alternatives must be rigorously explored and objectively evaluated. Where reasonable alternatives are infinite, a reasonable number of alternatives are required to cover the full spectrum of alternatives. In addition, a “No action” alternative is required to provide a benchmark, enabling decisionmakers to compare the magnitude of environmental affects of the action alternatives. This can be a critical element in deciding the preferred alternative because there are impacts with the No action alternative. For example, if the proposed action was to construct mass transit because of the increased traffic in the specified area, the impacts associated with the no action alternative is the increase in traffic.

## REFERENCES

- Corbitt, Robert A. *Standard Handbook of Environmental Engineering*, 2d ed. McGraw-Hill, New York, 1999.
- Environmental Protection Agency (EPA), *Musts for USTs: A Summary of Federal Regulations for Underground Storage Tank Systems*, EPA 510-K-95-002, 1995.
- Environmental Protection Agency (EPA), *Straight Talk on Tanks: Leak Detection Methods for Petroleum Underground Storage Tanks and Piping*, EPA 510-B-97-007, 1997.
- Environmental Protection Agency (EPA), *Don't Wait until 1998: Spill, Overfill, and Corrosion Protection for Underground Storage Tanks*, EPA 510-B-94-002, 1994.
- Virginia Department of Environmental Quality, *Procedure for Environmental Impact Review of Major State Facilities*, 1998.

## CHAPTER 9

# HISTORIC AND ARCHAEOLOGIC ASSESSMENT

*Elizabeth Couture, MURP, MARCH*

### GETTING STARTED

Preservation is viewed through buildings, artifacts and societal remnants saved in part or whole. It is a process that interacts with land development in two ways: (1) government ordinances, permits, zoning, and taxation; and (2) community awareness, involvement, and reaction. The extent of interaction varies with the historic or archeological resource perceived, discovered, or clearly evident on the land parcel. Understanding how preservation relates to the ownership and use of land is the context of this section.

Part of owning land and shaping its use centers on a knowledge of and respect for its past. That past may present clear and obvious historic or archeological attributes, or it may encompass a process of study and evaluation that leads to the identification or elimination of preservation resources. The discovery of such preservation resources, how best to define alternative strategies for their inclusion with the planning and development of land, and why their inclusion in the process of land use grows increasingly more important requires an understanding of:

- The Preservation movement
- Initial resource assessment
- Public preservation tools
- Strategies for resource inclusion

Historic preservation and archeology are often thought of together as they clarify the past. Images of “old” things displayed in museums or on tables tell a story of our past. Much of what we display is valued as historic, not for any national or monumental reason, but merely because it has survived time. These images and how changes to them affect our everyday lives have dramatically altered the rela-

tionship of both professions to land development. We all hold such images—the old courthouse, cemeteries, battlefields, monuments, sites cordoned off for the excavation of ruins and artifacts, and more finite examples of framed architectural drawings, aged photographs, and dusty stacks of deed or land books on library shelves—all of which highlight the poignant importance they play in shaping the definition of a historic resource.

Preservation of historic and archeological efforts identify “material” resources significant to the individual and community as a whole. By its very nature, the evolution of preservation is a natural off-shoot of land changes and development. It is that part of growth that requires a respect and acknowledgement of cultural precedent with careful investigation and documentation to assure that sufficient “images” remain amid the rapid and ever changing fabric of our landscape.

Keeping intact parts or all of a particular material image underscores the increasing role of historic preservation and archeological efforts by citizens, public officials, and land development professionals. Setting aside regulatory and legal concerns within the industry, one should recognize that the precedent use of any given piece of real estate embodies a history of land use itself.

Land development by its nature is the creation of “places,” each of which has the opportunity to establish history. With this understanding, each “developer” (owner, planner, and/or representative) bears the responsibility to respect the past and to plan for the future. Our future building efforts stand on the foundation of land development accomplishments for thousands of years.

Preserving the artifacts of our history is a reasonable response to a culture that aches for a sense of continuity while



**FIGURE 9.1** C&O Canal. (Courtesy of D.C. Insider Tours)

living amid constant physical change. Many people and communities feel a decline in continuity as they witness the physical landscape change daily. Steady and rapid change, be it building construction, increased vehicular traffic, and/or cultural integration, contribute to the psychological need people feel to live with a “sense of place.”

There is a basic human need for “permanence” and respect for “society” as perceived; this need weaves a thread of stability within “places” experiencing physical change. Rapid social and physical changes without regard to these individual and collective needs have fostered community outrage on more than one occasion. Heightened sensitivity to preservation concerns within the land development process can quiet such outrage and yield positive outcomes.

The information in this section is organized and chosen to broaden the understanding of preservation and archeological considerations within the framework of on-going land development and societal change. The old adage, “what goes around, comes around,” holds true with regard to land development. Consideration of preservation issues at the initial stage of developing a parcel of land will result in a variety of long-term benefits, which typically include:

- A reduction in real or perceived community apprehension, the loss of history
- Cost savings as a result of not having to backtrack at a later date to resolve issues not addressed
- Expedient and more cooperative public review
- The product result of a built form, which has cultural merit as well as a strong design integrity

### Historic Context

For thousands of years people have worked to establish their “mark” on society and/or time. Monuments, tombs, buildings, billboards, and even the first McDonald’s define accomplishment by a person or group of people. Such ac-

complishments define our society, the people, their routines and their daily lives.

While many builders, citizens and municipalities have demonstrated a regard for historic preservation and worked vigorously to save unique structures and land features, many have not. Numerous instances may be cited where a revered building or site has been destroyed, resulting in outrage. Community reaction to real or perceived threats to resources of our nation’s history have significantly influenced the language and enforcement of federal, state, and municipal legislative laws, permitting procedures, penalties, taxes and zoning ordinances during the 20th century. This increasingly stringent regulatory environment, coupled with selective financial incentives and tax credits, has spurred a major increase in rehabilitation, reuse, reconstruction and documentation efforts.

Archeology parallels historic preservation, interacting within the process of land development in a similar and related way. Archeology embodies the identification of history through the study of remnants, bits and pieces of materials buried and left by civilizations and persons of the past. Each artifact uncovered at a location brings factual importance to the investigation of a particular event or place. In many cases, like earthquakes, crashes, fire, etc., life and death casualty reports and cause-related evidence can only be understood by the archeological findings.

Archeological studies are time-consuming and expensive. Consequently, the process of archeology, unlike historic preservation, is undertaken generally after careful examination of existing physical forms to determine the importance and likely outcome of excavation.

Initial studies of on-site landforms, building remains and former land surveys offer excellent indications of whether further investigation is warranted. The trained eye and skilled hand of a professional archeologist can often gather sufficient information in a partial or sample excavation to determine if a particular land parcel, or isolated location within the parcel, should be contained for further investigation.

The U.S. Department of the Interior has categorized land elements, available in publications, as key indicators that an archeological resource probably exists; they list land mounds, walls, wells, pits/dumps, ruins/relics, foundations, vessels/ships/boats, depressions, caves, cemeteries, and/or unusual vegetation formations. Where encountered, stop and take time to permit on-site examination with possible sub-surface testing. The importance to and association with land development efforts is clear. Early involvement and site examination could curtail and resolve questions of archeological resources raised later in the development process under the guise of environmental assessment.

### Historic Overview

The movements of historic preservation and archeology started with the concerns and actions of private individuals.



**FIGURE 9.2** Jamestown archaeological dig site. (Courtesy of Jamestown National Historical Archives)

It has subsequently been institutionalized and has evolved into professional opportunities for a diverse range of disciplines, including architects, historians, landscape architects, lawyers, engineers, photographers, archaeologists, land surveyors, and educators. Several factors have nurtured the increasing professional pursuit of historic and archeological interests within the purview of land development activity. They include:

- Patriotism: our communal and individual need to protect and acknowledge the accomplishments of our national heritage
- Due Process—Representation: legislative actions at all levels of government have legitimized a public mandate for preservation activity
- “Roots”: our personal need as individuals and professionals to recognize and save artifacts commemorating our heritage
- Media: attention to lost cultural and historic artifacts has fostered community awareness and education of preservation
- Profit: The pragmatism of development that a saved or eliminated historic resource will generate profit

The home of the historic preservation movement is reputed to be Fairfax County, Virginia. It began in 1858, when the Mount Vernon Ladies Association succeeded in saving Mount Vernon, the plantation home of President George Washington (1754–1799). Their efforts were enhanced when the State of Virginia passed legislation empowering the association with the duty to manage, maintain and restore the property as a place of national importance for public education and enjoyment.

Comparable efforts can be matched by works of many other organized and individual endeavors nationwide. Restoration began in Sturbridge Village, Massachusetts (1859), in the historic district of El Pueblo de Los Angeles (1920),

Monticello—the home of President Thomas Jefferson (1923), the historic core of Charleston, South Carolina (1930), Colonial Williamsburg, Virginia (1930), and in New Orleans, Louisiana (1937). These are but a few examples.

Key legislative actions by the United States Congress have fostered the growth and importance of historic preservation and archeology. They include the Antiquities Act of 1906, which concentrated national attention on the protection of specific building and archeological sites and was largely spurred by park and monument building efforts starting after the Civil War. The intent of this act was expanded in 1935 with the Works Progress Administration (WPA) of the National Recovery Act during the Great Depression.

Out of these programs grew the Historic American Building Survey, a work program calling for the measurement and photographic documentation of particular historic structures. In 1935 Congress passed the Historic Sites Act, which authorized the Secretary of the Interior to identify and acknowledge properties of value to national history and archeology. At this time the National Historic Landmarks Registry and the Historic American Building Survey were permanently established within the U.S. Department of the Interior.

In 1949, Congress chartered the establishment of the National Trust For Historic Preservation, a quasi-public organization created to bridge the gap between private citizen efforts and government objectives to identify and preserve qualifying properties and sites of historic significance. In its creation, the Trust served to recognize officially and unify over 100 years of grass-root citizen and government efforts to preserve significant sites and buildings.

In step with this commitment, Congress passed the “Housing Act,” 1961, requiring the Secretary and Department of Housing and Urban Development (HUD) to identify, assess, and aid in the protection of historic properties within the guidelines of urban renewal activities. Congress further charged the Secretary and Department of Transportation in 1966, under the Transportation Act for Highway Expansion, with the responsibility of identifying potential archeological sites for exploration and/or candidate historic structures as a component of highway planning, design, and construction. Congress next passed the National Historic Preservation Act (1966), further expanding the responsibilities of preservation administration within the U.S. Department of the Interior and the National Park Service.

President Lyndon Johnson’s Housing Act of 1964, America Beautiful campaign, and the Housing and Urban Development Act of 1968 propelled the private sector preservation movement in negative reaction to federally funded urban removal and reconstruction projects in numerous older city neighborhoods nationwide (Waterbury, Connecticut, being one example).

Another powerful event that strengthened the preservation movement and challenged development efforts is the

1978 U.S. Supreme Court's ruling to halt demolition of Grand Central Station in New York City. The Court ruled the train station to be of cultural, historical, and architectural significance to citizens in New York City. The Court ruled that preservation of the building would not subject the land developer to an unjust economic burden. The decision expanded the base of preservation.

Legislative acts established a precedent for future land development ventures in several ways. For the first time, all building types, including but no longer limited to those associated with the "Founding Fathers" and colonists, were now candidates for preservation as a function of their location and association with the heritage of a "place."

Second, preservation forces could effectively petition the courts to enjoin developers from demolishing or jeopardizing the historic integrity of a structure. The extent of such forces served not only to stop but also to re-direct the design and development of a program envisioned. Third, the decision challenged the intent of zoning. Whereas previously a landowner viewed zoning as the definition of existing or potential development for a particular land parcel, this ruling established that a land use zoning classification does not necessarily guarantee a right or expectation of development. Last, but perhaps most important, this ruling also established a new concept for "fair" economic returns within the context of the private land development financial interest. It upheld that a profit is reasonable and expected by all private development efforts. However, the extent of that profit needs to be weighed against the costs that might otherwise be incurred with the loss of a historically significant structure or site. The ruling has been challenged many times and continuously upheld. When historic resources are in potential jeopardy, the privilege of private land ownership and development rights are placed under scrutiny.

### Preservation Resource Identification

Preservation and archeological exploration has come a long way since the 1978 Supreme Court ruling. As a movement and philosophy, it has grown far beyond the traditions of rebuilding and saving antiquities to the appreciation of built (buildings, land forms, and "things") elements common to everyday man that have merely survived time.

Today, 72,000 buildings and sites are listed on the National Register; only 77% of these are structures, including all building types and tunnels, roads, dams, bridges, etc. (See: [www.cr.nps.gov/buildings.htm@aol.com](http://www.cr.nps.gov/buildings.htm@aol.com).)

Public policy and incentives to work with land development endeavors have also expanded, allowing broader and more incremental flexibility to the extent and goals of preservation.

The National Historic Preservation Act of 1966 was expanded in 1992, Section 110, to broaden Federal agency involvement. Today, preservation staff and cultural resource policies exist within the following agencies (those of major significance to land development are underlined):

- Department of Agriculture
- Farm and rural services
- Forest and Natural Resource Conservation services
- Department of Commerce
- Economic and Trade (National and International)
- Ocean and Atmospheric Trade
- Department of Defense
- All sectors have rescues and sites to maintain
- Department of Education
- Department of Energy
- Department of Health and Human Services
- Department of Housing and Urban Development
- Department of the Interior
- Bureau's of Indian, Land, and Reclamation
- National Park Service
- U.S. Geological Survey
- Mining, Fish and Wildlife Services
- Department of Justice
- Department of Labor
- Department of State
- Department of Transportation
- All sectors: rail, road, aviation, coastal, and transit
- Department of the Treasury
- Department of Veteran Affairs
- Environmental Protection Agency
- Federal Communications Commission
- Federal Deposit Insurance Corporation
- Federal Housing Finance Board
- Federal Emergency Management Agency
- General Services Administration
- Library of Congress
- Metropolitan Washington Airports Authority
- National Aeronautics and Space Administration
- National Capital Planning Commission
- National Endowment for the Humanities
- National Science Foundation
- Nuclear Regulatory Commission
- Presidio Trust
- Small Business Administration
- Smithsonian Institution
- Tennessee Valley Authority
- United States Postal Authority

Note: each Federal agency is further expanded at State and Local government levels with comparable policy and regulatory programs. See: [www.achp.gov/federal.html@aol.com](http://www.achp.gov/federal.html@aol.com).

Controversial land development efforts where preservation is concerned will also fall under Section 106. This section created the Advisory Council on Historic Preservation as an independent Federal agency that provides a public and private forum for the resolution of many interests. See: [www.achp.gov/about\\_council@aol.com](http://www.achp.gov/about_council@aol.com).

Time coupled with increased land development nationwide reveals the ever-growing power of preservation. Resources, physically present or perceived (like battlefields,

mines, and other subsurface archeological evidence) are ever more important to people. The land developer, now more than ever, needs to be knowledgeable of the movement. What determines whether a resource is worthy of preservation or archaeological consideration must be understood within the context of different information categories:

- Resources of national importance are defined by the criteria for landmark and National Register nomination:
  - Association with significant events
  - Persons who have contributed greatly to our nation
  - Distinctive architecture based on style, materials, age (50 years or older in most cases), construction technique, or that represent the work of a master
  - The potential to yield information pertinent to national history or prehistory
- Resources of state and local significance based on the interpretation of national criteria applied at a more vernacular level
- Resources by their location within, adjacent to, or near a registered historic structure, site, or district
- Resources as a function of environmental assessment findings
- Potential resources based on perceived historic value

It is this very aspect of growing flexibility in land-use that draws every property or building under development in scrutiny with regard to potential historic value. In most instances, the opportunities and benefits emerge early in the research and design phases of the land development process. At the earliest possible stage of land acquisition and development, planning an assessment of potential preservation resources should be taken.

Acquisition typically includes the legal research of land records and the civil engineering of property boundaries. These transactions and others frequently produce information important to future development planning. At the initial stage of land acquisition and development planning, look at deed and property plats for property characteristics that merit further study. Ask real estate, legal and title personnel during the process of land acquisition to watch for and highlight any signs of a historic or archeological resource. Secure copies of deeds searched in support of land acquisition and clear title. Analyze each document for references to historic events, buildings, people, or cemeteries. Likewise, property surveys and engineering building studies are often routinely performed during the acquisition process; ask each professional to look for evidence of historicity—this can not be overemphasized.

Williams Center, a mixed-use development project in Manassas, Virginia, was stopped by the reaction of one citizen who effectively organized a national movement to save

a civil war battlefield based on its archeological importance. Years of community outrage matched by extensive efforts on the part of the developer to study alternative designs and legal avenues for project realization saw the project end in a condemnation law-suit in 1990. The court ruled that the property be purchased and preserved as a national park; this decision cost taxpayers millions of dollars for land acquisition and legal procedures. Perhaps early archeological investigation would have curtailed such an outcome? At least, in this case, early study would have afforded the developer more alternatives for land use or sale.

This case further demonstrates the diverse range of professional, historians, land planners, landscape architects, and civil with geotechnical engineers to a broad spectrum of legal specialties included in the process of historic resource protection today. Other people employed or actively volunteering in the fields of preservation and archaeology are also equally valued in the process. The list of specializations is lengthy, and in cases overwhelming. This case clearly demonstrates that the job of resource definition and resolution is complex.

In an attempt to simplify this maze, a two-step approach is offered. Start at the local government level, asking who or what office handles preservation and archaeology concerns. With a clear understanding of local information and terminology, branch to county, region and state government levels. In most cases, a telephone call will open the opportunity for further research via publications, internet, or library. Second, as your knowledge grows, so will the depth of your questions. Similarly, you will gain a better understanding of professional resources at hand and willing to help your effort.

One of the best ways to initiate resource identification is to obtain site and policy information from municipal representatives employed with the job of coordinating and addressing issues of historic preservation, archeology and land planning. You may find that their help saves time and money during the early planning stages.

## LEGAL ISSUES PERTAINING TO PRESERVATION

There is abundant legal precedent affording protection to historic and archeological resources at all levels: local, state and national. The law clearly sets forth limits regarding the disturbance of land and the improvements thereon. Land ownership and development carry the responsibility of being knowledgeable of such limits, especially with regard to the demolition of site improvements, the disturbance of cemetery plots, and the excavation of earth. While it is often difficult for federal, state, and local administrators to police every resource, penalties, fees, and criminal charges can result where deliberate action has resulted in the loss of property or artifacts that may be of historic or archaeological importance.

It is prudent to secure legal advice at the earliest stage of land acquisition and program planning to gain a clear understanding of the rights and responsibilities affiliated

with a development effort. Advice pertinent to preservation issues within the endeavor should include:

1. Ask that title reports and deeds searched for land acquisition be reproduced for file records, and that appropriate notations be made regarding ownership and property improvements over time.
2. Request legal interpretation of references and notations on deed copies provided.
3. Secure copies of property plats, often referenced in deeds and required for financing.
4. If a new plat is required, ask the surveyor to re-search older, former plats, and to provide copies.
5. In the event that deeds or plats reference improvements, relics, former events, sub-surface conditions (wells, waste, burial), or information implying historic or archaeological resources, ask for a definition.
6. Last, maintain records and communication. With good records and clear questions, a great deal can be resolved. Most attorneys involved in land development have prior experiences that may offer guidance to your situation. Likewise, most public agents working in land or historic planning have knowledge of other projects that may be important to your planning effort.

## CONCLUSIONS

The role of historic preservation and archaeological resource assessment within the land development process is continually growing and expanding. The traditional concerns of a property being historic in terms of age and national importance have branched into a wider arena, where contemporary issues of culture, heritage, community, environment, material use, and archaeology are of significant importance. Most land development endeavors recognize and plan for inclusion of these concerns. The emphasis of this presentation has been to increase the level of understanding of and appreciation for the role of historic preservation and archaeology as a function of land change.

The cost to overlook resource assessment is substantial. Every legal case since Mount Vernon heightens this reality. The natural process of change spurred by economic supply or demand, profit, or the needs of people must be viewed in context with the real and perceived value history holds. The rate and degree of change serves to increase the importance of history, in part or in whole, to people. This section broadens both awareness to and appreciation for preservation and archaeology interests as they play a direct role in development.

## REFERENCES

Couture, Elizabeth, for the Batman Company and U.S. Department of the Interior, McNair Farms. Herndon, Virginia, 1988.

- Darr, Lonnie, *A Technical Manual for Woodland Conservation with Development in Prince Georges County*, Maryland-National Capital Park and Planning Commission, Upper Marlboro, Md., 1990.
- DeChiara, Joseph, and Lee E. Koppelman, *Time-Saver Standards For Site Planning*, McGraw-Hill, New York, 1984.
- Fairfax County, *Comprehensive Plan: Fairfax County, Virginia Area III*, 1986.
- Fairfax County, 1987 Standard Report, 1987.
- Fitch, James Marston, *Historic Preservation*, McGraw-Hill, New York, 1982.
- Gillette, Jane Brown, "Elvis Lives," *Historic Preservation Magazine*, May 1992.
- Macie, Edward A., and Gary Moll, *Shading Our Cities: A Resource Guide for Urban and Community Forests*, Gary Moll and Sara Ebenreck, eds. American Forestry Association, Island Press, Washington, D.C., 1989.
- Marois, Denise. "Farming: 'A Way of Life under Seige,'" *The Gazette*, Vienna, Va., November 10, 1988.
- Mary Morris Planning Advisory Service, *Innovative Tools for Historic Preservation*, Report #438, 1992.
- Miller, Robert W., *Urban Forestry: Planning and Managing Urban Greenspaces*, Prentice Hall: Englewood Cliffs, New Jersey, 1988.
- Moll, Gary, and Sara Ebenreck, eds., *Shading Our Cities, A Resource Guide for Urban and Community Forests*, American Forestry Association, Island Press, 1989.
- Moloney, Noval, *Archaeology*, Oxford University Press, England and New York, 1996.
- The Mount Vernon Ladies Association of the Union, *Mount Vernon: A Handbook*, Mount Vernon, Va., 1985
- National Association of Home Builders of the United States, *Land Development*, National Association of Home Builders of the United States, Washington, D.C., 1987.
- Natural Resources Defense Council Inc. v. EPA*, Nos. 90-70671 and 91-70200, slip op. at 6217 9th Cir. June 4, 1992.
- Netherton, Ross, and Nan Netherton, *Fairfax County in Virginia*, Donning Company, Norfolk, Va., 1986.
- Robinette, Gary, *Local Landscape Ordinances*, Agora Communications, Plano, Texas, 1992.
- Smardon, Richard C., and James P. Karp, *The Legal Landscape*, Van Nostrand Reinhold, New York, New York, 1993.
- Travers, Naomi S., "Last Fairfax Dairy Farmer Holds Out for Love of the Land," *Washington Post*, Sept. 6, 1988.
- U.S. Department of the Interior, National Park Service, *National Register Bulletin: How to Apply the National Register Criteria for Evaluation*, Publication #15, Washington, D.C.
- U.S. Department of the Interior, National Park Service, *Guidelines for Preservation*, Washington, D.C.
- Upton, Dell, and John Michael Vlach, *Common Places: Readings in American Vernacular Architecture*, University of Georgia Press, Athens, Ga., 1986.
- Weinberg, Nathan, *Preservation in American Towns and Cities*, Westview Press, Boulder, Colorado, 1979.
- Wienczek, Henry, *The Smithsonian Guide to Historic America*, Stewart, Tabori & Chang, New York, 1989.
- Wood, Joseph S., "Shaping The Present: Geographical Essays on Historic Preservation Policy," *North American Culture, Society for the North American Cultural Survey*, Vol. 4 No. 2, 1988.

## CHAPTER 10

# MARKET ANALYSIS AND ECONOMIC FEASIBILITY

*JoAnna Pillis Alexis*

### INTRODUCTION

In determining the feasibility of developing a parcel of land, one cannot overlook the importance of the economic feasibility of the project. Previous chapters within this book have dealt with the environmental and engineering-related issues that need to be evaluated during a comprehensive feasibility study. This chapter will discuss the components of economic feasibility and its importance in developing a successful project.

In its most elemental form, economic feasibility analysis is nothing more than stacking up the most likely scenario of income and expense and answering the question “Is the anticipated return enough to justify spending the money?” While the market analysis identifies the product features that will lead to the probable income profile, cost estimating identifies the expense part of the formulas. Additionally, the required investment return of the developer/builder gives the parameters within which yields are acceptable, while the experience factor often shapes the decision in ways that are not obvious to any student of the science of analysis.

The objective of the market study and the marketability analysis is to

1. Define the specific product and features.
2. Set the price range.
3. Project the expected pace of sales or leasing.
4. Identify the most probable changes that will occur in the economic and development climate between initiation of the project and completion of the program of sales or leasing.

Decisions based on this information will dictate the cost structure, densities sought, timing of delivery of product,

and the target purchasers or tenants. However, regardless of the quality and quantity of statistical information available, the feasibility study also requires a thorough understanding of elements such as community acceptance and local political environment. This information can be obtained only from personal interaction with regulatory staff in the applicable jurisdiction, elected officials, and community leaders.

### MARKET ANALYSIS

The market analysis is performed to define the economic environment. Features analyzed include demographic data, transportation patterns, and regional social and economic trends that will affect the supply and demand for a specific type of real property. The analysis provides detailed information about commuting or travel trends, saturation and absorption rates for certain types of uses, and consumer preferences for price, land use, or density variations, and it is also used to identify those items that meet with political or consumer resistance.

The economic feasibility study looks at those factors that determine the needs of the community for housing, employment, and retail areas and other land uses. It examines the growth rate of employment in the community, manufacturing output, retail sales, and other elements that may reflect job creation and demand for commercial space and a variety of appropriately priced housing. The analysis looks at demographic factors such as number of households, household income, and age distribution. These factors determine the community's affluence and characteristics of the available labor force. In addition, those who perform market analyses look to the interrelationships between uses. Certain land uses create demands for other types of land



use that service them, such as warehousing facilities near manufacturing centers. Some land uses, usually public facilities or regional service centers, have features that make it attractive for related land uses to locate near them. An airport is an example of such a facility, which will attract hotels, shipping and warehousing facilities, and offices. The broad approach may be used by employers in other cities or by developers looking for specific properties in locations that fit in with company operations.

However, the market analysis that is performed in conjunction with all but the most complex projects focuses on a narrower geographic area, which is usually the area in the general vicinity of land under consideration for development. The market analysis is performed to identify elements that will help the developer determine what land use is most likely to succeed in a given location, what specific form and density it should take, what price range is reasonable, and what features and amenities should be incorporated into the project.

Although the analysis always reflects a point in time, decisions based on it must recognize that in many cases, land development is a multiyear process. Conditions often change significantly during the life of a development; therefore, comprehensive studies must take into account other competitive or complementary projects that may be under consideration by other developers or may already be in the review process. Such projects may either absorb some of the demand that is assumed to exist for the client's project or create new demand that will enhance the prospective project. By conducting market research before undertaking a development project, the developer can lessen the risks associated with the venture. The market analysis prepared for the land developer enables the consulting team to narrow their range of project alternatives with a plan supported by its conclusions. While the design consultants rarely conduct the market research for the developer, their early participation in the analysis is valuable.

Additionally, the market study inventories the range of products that are currently available. It categorizes them by their size, price range, and competitive features and the dynamics and change in consumer demand and preferences. Among these factors are the characteristics of the resale market for existing homes. Strength in the resale market, signified by homes on the market for short duration, suggests that demand could be strong for move-up homes, those in the next-higher price range. A survey of the competition should categorize projects by location, price, unit and lot size, features, and sales or leasing rate. This information is important to the design consultants engaged in studying project feasibility, as well as for those projects already in development review.

The demographic analysis examines the characteristics of the local population. Overall population growth and rates of household formation affect gross housing needs. In addition, the researcher specifically looks for patterns that are likely to affect demand for certain types and qualities of

land use. Data on size of households can be interpreted as suggesting demand for variations in housing, lot size, and number of bedrooms. A predominance of high household income could create demand for larger, more luxurious units as well as units requiring low yard maintenance. An age cohort analysis indicating a sizeable middle-aged or elderly population, so-called empty nesters, could also suggest demand for smaller, luxury units. In some communities, this demand could suggest housing types that require less maintenance, such as apartments or townhouses in condominium or cooperative ownership. Have certain segments of the population been overlooked? Could there be demand for senior citizen communities, more affordable housing, or projects that emphasize leisure-time activities? Information on commuting patterns, such as direction of peak hour flows and residents' tolerance and expectations for distance and time, is also important. Along with proximity to certain employment centers, this information could indicate who the potential buyers are likely to be.

Examination of the economy concerns itself with employment conditions and job growth. Two important elements are conditions that encourage job retention and local efforts aggressively attracting new businesses. Quality of life issues are important factors in the local economy. These include housing affordability, availability of an appropriately skilled labor force, cost of living, tax burden on individuals and corporations, interregional transportation facilities, incidence of crime, community identity, and cultural amenities. Of particular interest is the expansion of companies already located in the community and the movement of new employers into the area.

### Information Sources

Much of the information contained in the community's comprehensive plan offers a strong base upon which to build data about the local market. As with all printed information, however, the timeliness of the information is critical because of rapid change caused by local, regional, and national forces. Local market research firms often maintain relatively current survey data that are useful to the development team. Such information often is very detailed and includes construction and sales rates data, along with a description of important features in all active projects in the area. In addition, builders often conduct surveys of consumer preference based on responses from those who visit their sales offices in different projects. These provide useful information about which features to emphasize or eliminate in subsequent projects. Local chambers of commerce, economic development authorities, utility companies, the U.S. Census Bureau, local building or vital statistics departments, and similar institutions are other valuable sources of market, economic, and demographic indicators. Organizations such as the National Association of Home Builders, the National Association of Industrial and Office Parks, and the Urban Land Institute periodically publish information concerning consumer preferences.

## MARKETABILITY STUDY

Marketability analysis deals with the acceptability of a specific property and product type to be absorbed successfully in a given market condition. Recognizing that most projects, even small ones, require years of preparation, development, and construction prior to delivery of the product to the purchaser or renter, the marketability analysis will often be done several times during the life of the project. During the initial feasibility study, the developer/builder is often dealing with the broadest form of the product—for example, townhouses versus single-family dwellings, how large, and what overall product mix. As the engineering and design process progresses from schematic to conceptual, the product definition takes further shape—what exterior materials, major components of landscaping, patios, and parking areas. Just before commencement of construction, larger builders are often still testing the marketability of their specific product types—what has sold most among other new homes in the region? How large are the bedrooms? How many fireplaces are attractive? What features are popular in the kitchen? What modifications are indicated in feedback from the model home? The most common sources of information at this final stage are the purchasers themselves and the local brokerage community most familiar with the product type and sales or leasing, with many larger brokerage firms having acquired professional market research firms that will do gross statistical analysis for their clients.

## COST PROJECTIONS

In order to determine the potential for profitability, it is required that all of the elements of development be identified and cost estimates be made, with increasing levels of specificity as the project moves toward start of construction. Some of the broad elements of costs are listed below. All should be estimated with the greatest possible accuracy, given the information at hand. Many of these elements will be defined through the engineering consultants' reports. These reports will identify certain physical features of the property, from which initial cost estimates can be obtained from the engineers and then corroborated with construction professionals. The following list is not intended to be all-inclusive, but rather to demonstrate the range of costs involved in most development projects. Because this book deals with land development, the list is most detailed on such development issues and mentions only briefly construction costs of the buildings themselves.

### Major Cost Categories

- Rezoning, special use permits, special exceptions, and other amendments to land use, including costs of engineering and legal and community outreach to obtain approvals
- Land cost (and its relationship to yield) and structure of payment for land

- Feasibility studies, market analysis
- Contracting and purchase costs, including legal, title work and title insurance, closing costs, appraisals, loan fees, and lender's required studies
- Engineering and design
- Environmental studies
- Boundary survey
- Topography
- Flood plain studies
- Soils tests and bearing capacity
- Site plan, subdivision plan, plans and profiles
- Easements
- Land development costs
- Clearing, demolition, rough grading, and earth balance
- Soils/rock—soil import/export
- Hazardous materials removal and restoration
- Siltation control, maintenance during construction
- Stormwater management, best management practices
- Wetlands protection and management
- Retaining walls
- Streets and access, including temporary construction access
- Curb and gutter, sidewalks, driveways, medians
- Trails and bike paths
- Utilities: easements, infrastructure, permits and access fees
- Sanitary sewer
- Water
- Gas
- Electric
- Telephone, Internet, cable, specialized access
- Street and lot lighting
- Landscaping, trees
- Fences and decorative walls
- Offsite costs (e.g., downstream mitigation, easement acquisition)
- Covenants, conditions and restrictions, formation of owners' associations, applications and qualifications

for regulatory approval (e.g., FHA approval of subdivision), condominium or cooperative documents

- Entry features and signs
- Specialized impact studies (for example, noise studies near airports) and mitigation costs
- Letters of credit, bonds, and other forms of surety
- Completion agreements and other development documents
- Final subdivision approval, punchlist and repairs
- Insurance
- Real estate taxes during holding period and special assessments
- Other transportation access and interface (e.g., bus or train connections)
- Contribution to public facilities, impact fees, and similar proffer costs
- Architecture and engineering fees
- Testing and inspections (fees paid to local regulatory agencies and consultants for professional testing)
- Hard costs of construction of buildings, garages, and related structures
- Amenity package and costs (parks, tot lots, pools, tennis courts, club houses, etc.)
- Merchandising, advertising and marketing costs

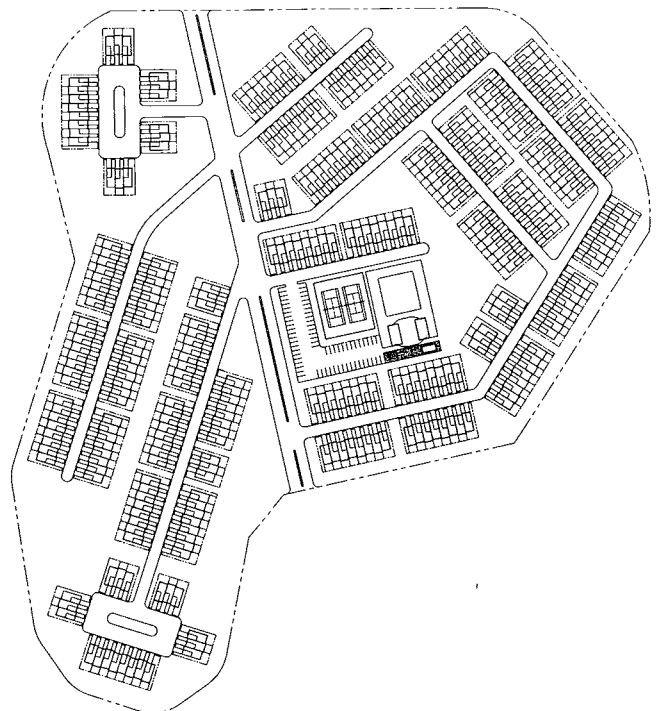
Given that entire courses of study are devoted to techniques of investment analysis, this discussion will limit itself to the broadest definition of the practice. Every dollar that goes into a project has a cost associated with it. If it is a borrowed dollar, it has a defined interest rate, fees, and a principal repayment schedule. The lender has made its own evaluation of risk, including its evaluation of the viability of the primary source of repayment (i.e., the project and its sales or leasing probability), secondary sources of repayment (i.e., personal guarantees, additional pledged securities, etc.), the experience and track record of the developer/builder, and the regulatory environment within which each such lender must function. The lender has priced its investment return based on market conditions for alternative uses of its available funds.

### INVESTMENT ANALYSIS

The timing of payment for the land becomes a significant factor in the success of a project. The seller and purchaser will negotiate the balance between the price paid directly for an all-cash settlement in the beginning of the development cycle, a delay of settlement during the contract term while pursuing rezoning and other regulatory entitlements, and payment as sales progress. In general, the later in the

process that the purchaser must produce dollars to pay the seller, the higher the price to the seller. Effectively, a landowner who contracts with a developer/builder for a land price based on yield in the zoning process and payment when units are sold has effectively become a partner with the developer/builder, as his land value is at risk until the complete payment is received.

Those dollars not borrowed are considered equity, or funds of the owner. In general, all such funds are fully at risk and require unlimited commitment with regard to time and amount. If the project fails to achieve its projected sales goals, the owner's funds must continue to carry the costs of the project and the loan. If the engineering feasibility failed to disclose a significant subsurface condition that leads to increased costs for excavation and removal of inadequate soils, the owner's funds must pay the additional costs of remediation, as well as continued carrying costs during the delay in the project. If the market study failed to disclose a significant competing project planned on nearby property that will dominate the market during the same years the subject project is coming to market, the owner's funds must bear the difference in project costs. Overall, the increased risks borne by the owner of the project demand that yields be substantially more than the debt rate to justify the investment. It is not uncommon for required returns for institutional investors to be between 20 and 35% per annum. Although many smaller entrepreneurial developer/builders do not define their required returns so clearly, the same elements are present in determining the advisability of undertaking any particular project, given

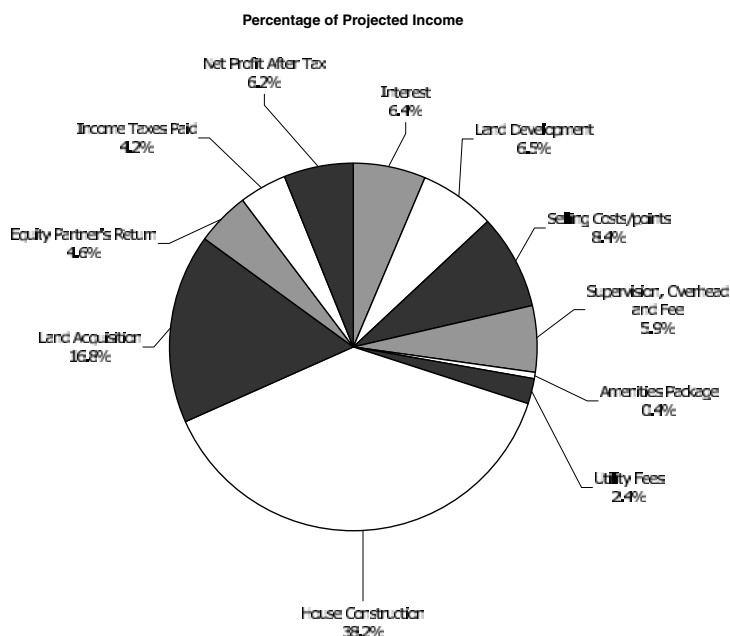


**FIGURE 10.1** Site plan for sample economic feasibility analysis.

**TABLE 10.1 Summary of Financial Projections**

LINE* NUMBERS	PROJECT SUMMARY	TOTALS	PER LOT 320	PERCENT
<b>Income</b>				
	Gross sales	\$127,418,934	\$398,184	100.00%
<b>Expenses</b>				
1 + 2	Predevelopment costs	\$145,000	\$453	0.11%
3	Design	\$500,000	\$1,563	0.39%
4 + 5 + 6	Land Development	\$7,700,000	\$24,063	6.04%
7	Amenities package	\$500,000	\$1,563	0.39%
8	Utility fees	\$3,040,000	\$9,500	2.39%
9	Marketing/advertising	\$470,000	\$1,469	0.37%
10	Real estate taxes/insurance	\$1,336,050	\$4,175	1.05%
11	House construction	\$48,622,216	\$151,944	38.16%
13	Land acquisition	\$21,440,000	\$67,000	16.83%
14	Overhead & supervision	\$4,361,929	\$13,631	3.42%
15	Developer's fees	\$1,869,398	\$5,842	1.47%
22	Selling costs/points	\$10,193,515	\$31,855	8.00%
34	Interest on land	\$5,499,360	\$17,186	4.32%
41	Interest on loan	\$2,693,388	\$8,417	2.11%
47 + 48	Equity partner's return	\$5,814,111	\$18,169	4.56%
	Total expenses	-\$114,184,967	-\$356,828	-89.61%
	Net profit before tax	\$13,233,968	\$41,356	10.39%
60	Income taxes paid	-\$5,293,587	-\$16,542	-4.15%
	Net profit after tax	\$7,940,381	\$24,814	6.23%

\*See the following pages for detailed calculations and explanations of items in this summary. Line numbers refer to Lines on the detailed explanations.



**FIGURE 10.2** Percentage of projected income.

**TABLE 10.2 Sample Economic Feasibility Analysis: Residential Project for Sale**

All figures shown in thousands of dollars  
 \*\*Average cost or income per lot expressed in whole dollars

NOTES	YEARS:	UNIT COST	1	2	3	4	5	6	7	8	9	10	TOTALS	AVERAGE PER LOT **
<b>Expenditures</b>														
1	Feasibility		\$70										\$70	\$219
2	Zoning		\$75										\$75	\$234
3	Engineering design		\$100	\$200	\$200								\$500	\$1,563
4	Grading/earth balance/walls				\$500								\$500	\$1,563
5	Land development: community				\$2,500	\$1,500							\$4,000	\$12,500
6	Land development: on lots	\$10,000			\$500	\$500	\$500	\$500	\$500	\$500	\$500	\$200	\$3,200	\$10,000
7	Amenities package				\$500								\$500	\$1,563
8	Utility fees	\$9,500			\$3,040								\$3,040	\$9,500
9	Marketing/advertising				\$100	\$100	\$50	\$50	\$50	\$50	\$50	\$20	\$470	\$1,469
10	Real Estate Taxes/insurance				\$337	\$288	\$243	\$197	\$148	\$85	\$23	\$15	\$1,336	\$4,175
11	House construction	\$140,000			\$7,000	\$7,000	\$7,210	\$7,426	\$7,649	\$7,879	\$8,115	\$3,343	\$48,622	\$151,944
12	<b>Subtotal</b>		\$245	\$200	\$6,677	\$9,388	\$8,503	\$8,174	\$8,347	\$8,514	\$8,687	\$3,578	\$62,313	\$194,729
13	Land acquisition		\$250	\$0	\$2,966	\$3,037	\$3,037	\$3,272	\$4,188	\$4,188	\$502	\$0	\$21,440	\$67,000
14	Overhead and supervision		\$17	\$14	\$467	\$657	\$595	\$572	\$584	\$596	\$608	\$250	\$4,362	\$13,631
15	Developer's fee		\$7	\$6	\$200	\$282	\$255	\$245	\$250	\$255	\$261	\$107	\$1,869	\$5,842
16	Interest on loan				\$641	\$629	\$537	\$407	\$281	\$158	\$40	\$0	\$2,693	\$8,417
17	Interest on land				\$1,458	\$1,336	\$1,093	\$841	\$543	\$208	\$20	\$0	\$5,499	\$17,186
18	<b>Total expenditures</b>		\$520	\$220	\$12,409	\$15,330	\$14,021	\$13,511	\$14,193	\$13,919	\$10,119	\$3,936	\$98,177	\$306,804

## T A B L E 1 0 . 2 Sample Economic Feasibility Analysis: Residential Project for Sale (Continued)

### NOTES TO SAMPLE ECONOMIC FEASIBILITY ANALYSIS

#### Expenditures:

1. Feasibility: Costs spent prior to acquisition for market analysis, engineering evaluation of soils, environmental factors, utilities, zoning and land use analysis, and other studies to determine feasibility of the project.
2. Zoning: Attorneys' fees, application fees, and engineering drawings to apply for change in zoning of the property. In this example, this is spent prior to acquisition of the land.
3. Engineering design: Costs of professional services for design of the project to be constructed, including boundary survey, topography, flood plain studies, site plan, subdivision plan, plans and profiles, easements, utility design, and other engineering requirements. Some of the funds spent during the study period under the contract and for the design of the basic elements of the subdivision will be spent prior to acquisition of the land, and some design work will continue after the project is underway.
4. Grading/earth balance/walls: Movement of soil on the site, including, when applicable, removal of poor soils and trucking replacement soils from offsite. In this example, the subject property is sloping and will require retaining walls and movement of soil from one part of the site to the other. This line item will vary dramatically from one project to another and is unique to the individual lay of the land and composition of the soil.
5. Land development: Community: Costs of construction of improvements that will benefit the entire community, including such things as water, sewer, gas, telephone, electric, cable and any other utilities, streets, sidewalks, curb and gutter, street lighting, and landscaping. In this stage all public improvements are brought to the individual lot lines, but not onto the lots. Other than earth movement, community land development is the first construction cost.
6. Land development: On lots: As sections of lots are sold, the individual lots are finished with utilities, lead walks, driveways, etc., brought from the lot lines to the houses themselves.
7. Amenities package: In this example, the project will have a pool, two tennis courts, a playground and a small clubhouse with showers and restrooms in a central location. This amenity package will generally be constructed after some of the houses are sold and occupied and families have started to move into the community to enjoy them.
8. Utility Fees: Availability fees paid to relevant governmental agencies and utility companies to tie into the public utility systems, such as water, sewer, gas and electric availability. These are often called "tap fees," and they vary widely among jurisdictions and may include payments for extension of services in some instances. These are one-time fees and generally are purchased for the entire community early in the project to assure availability of services for the entire project.
9. Marketing/advertising: These costs start as soon as the project can be shown and continue for the duration of the sales program. Marketing and advertising costs include ads in newspapers and magazines, brochures and flyers describing the project and showing options for floor plans and finishes, broker events, promotional gifts and other materials used for promotions.
10. Real estate taxes/insurance: Real estate taxes and liability and hazard insurance for the project. This figure drops over time as lots are sold.
11. House construction: This line item includes all of the hard costs of constructing the houses in this example, including foundations, structure, exteriors, roof, interior finishes, appliances and fixtures.
12. Subtotal: This subtotal represents expenses which can be funded under the construction loan (Lines 1 through 11, included).
13. Land acquisition: There are an infinite number of ways to purchase land; however, in this example, the purchaser is placing a deposit of \$250,000 initially, which is applied to the down payment at closing. Closing occurs soon after the rezoning is completed (in year 3 of this example) and 15% of the total land value is paid at closing. Thereafter, the remaining cost of the land is paid as the lots are sold (generally called a "take-down" of lots). If regular payments for the land stay ahead of lots sold, the lots are released from the deed of trust upon sale of the completed house, without further payment. However, when the sales program is going well and lots are being sold quickly, the builder/developer often has to pay for additional lot releases at the closing of sale of the houses. Releases ahead of schedule are always for more than 100% of the initial purchase price per lot. This example reflects the sale of 50 houses per year and the payment for the land follows accordingly.
14. Overhead and supervision: Costs of rent, equipment and supplies for the project office and on-site personnel, the construction manager who oversees the job, employee benefits, accounting costs, electricity and other utilities during construction, use of vehicles on the job-site, and other costs of running the "business" of the project.
15. Developer's fee: The fees paid to the professional developer or development firm for comprehensive oversight of the project, including all aspects of zoning, design, financing, construction, interaction with owners, management of sales personnel, marketing and all other aspects of the project. In smaller firms, this role is often assumed by the owner and may not be compensated by a separate fee. In larger firms, it may be a fee or group of fees paid to various departments which perform these individual functions. In some instances, the owners of the project hire the professional managerial assistance to perform this function and pay a fee based on a percentage of expenditures on the project or a percentage of sales within the project.
16. Interest on Loan: Interest paid to the institution financing construction and development costs.
17. Interest on Land: Interest paid to the seller of the land on the unpaid balance of the purchase price.
18. Total expenditures: Total of Lines 12 through 17, representing all expenditures for the project except the Equity Investor's Return.

**T A B L E 1 0 . 2 Sample Economic Feasibility Analysis: Residential Project for Sale (Continued)**

All figures shown in thousands of dollars  
 \*\*Average cost or income per lot expressed in whole dollars

NOTES	YEARS:	UNIT COST	1	2	3	4	5	6	7	8	9	10	TOTALS	AVERAGE PER LOT **
<b>Net Cash from Sales</b>														
19	No. of lots sold per year		0	0	0	50	50	50	50	50	50	50	320	320
20	Average sale price of house	5.00%	\$300	\$315	\$331	\$347	\$365	\$383	\$402	\$422	\$443	\$465	\$398	\$398
21	Gross sales per year					\$17,364	\$18,233	\$19,144	\$20,101	\$21,107	\$22,162	\$9,308	\$127,419	\$398,184
22	Selling costs/points	-8.00%				-\$1,389	-\$1,459	-\$1,532	-\$1,608	-\$1,689	-\$1,773	-\$745	-\$10,194	-\$31,855
23	Land releases at closing					\$0	\$0	-\$235	-\$1,385	-\$2,769	-\$502	\$0	-\$4,891	
24	Loan payment per lot					\$203	\$203	\$203	\$203	\$203	\$203	\$203	\$203	
25	Total loan payment					-\$10,157	-\$10,157	-\$10,157	-\$10,157	-\$10,157	-\$10,157	-\$4,063	-\$65,006	
26	Net Cash from Sales		\$0	\$0	\$0	\$5,818	\$6,617	\$7,221	\$6,951	\$6,491	\$9,729	\$4,500	\$47,328	\$147,900
<b>Land Acquisition</b>														
27	Scheduled payments	6	\$250		\$2,966	\$3,037	\$3,037	\$3,037	\$2,803	\$1,418	\$0	\$0	\$16,549	
28	Releases allowed				\$83,750	38.40	36.27	36.27	33.47	16.93	0.00	0.00	0.00	
29	Releases taken				0.00	-50.00	-50.00	-50.00	-50.00	-50.00	-50.00	-20.00	-20.00	
30	Balance of released lots				38.40	24.67	10.93	-2.80	-16.53	-33.07	-50.00	-20.00	-20.00	
31	Additional release required				0.00	0.00	0.00	2.80	16.53	33.07	50.00	20.00	20.00	
32	Additional payments for re-leases				\$84	\$0	\$0	\$235	\$1,385	\$2,769	\$502	\$0	\$4,891	
33	Balance due		\$67	\$21,440	\$18,224	\$15,187	\$12,149	\$8,878	\$4,690	\$502	\$0	\$0	\$0	
34	Interest on average balance	8.00%			\$1,458	\$1,336	\$1,093	\$841	\$543	\$208	\$20	\$0	\$5,499	
<b>Lots Sold</b>														
35	Lots sold					50	50	50	50	50	50	20	20	
36	Lots remaining					270	220	170	120	70	20	0	0	
<b>Construction Loan</b>														
37	Construction Draws				\$7,122	\$9,388	\$8,147	\$8,503	\$8,347	\$8,514	\$8,687	\$3,578	\$62,313	\$195
38	Interest draws				\$0	\$641	\$537	\$629	\$407	\$281	\$158	\$40	\$2,693	\$8
39	Payments				\$0	-\$10,157	-\$10,157	-\$10,157	-\$10,157	-\$10,157	-\$10,157	-\$4,068	-\$65,006	
40	Balance due				\$7,122	\$6,994	\$4,522	\$5,969	\$3,119	\$1,756	\$445	\$0	\$0	
41	Interest	9.00%			641	\$629	\$407	\$537	\$281	\$158	\$40	\$0	\$2,693	

**TABLE 10.2 Sample Economic Feasibility Analysis: Residential Project for Sale (Continued)**

**Net Cash from Sales:**

- 19. No. of lots sold per year: The number of lots sold in this example each year is 50, starting in the 4th year of the project, or one year after acquisition of the land. The final year shows 20 houses, or the balance of the project being sold.
- 20. Average sale price of house: In this example, the houses are projected to sell for \$300,000 in the first year, with an average increase in sale price of 5% per annum. By the time the first houses are constructed and sold, the price has escalated to \$347,000 and to \$456,000 by the 10th year. The average sale price of the houses is shown as \$398,000.
- 21. Gross sales per year: This line reflects Line 19 multiplied by Line 20.
- 22. Selling costs/points: This is the cost of commissions, recordation taxes and fees, homeowner's warranty package, administrative closing costs, incentive programs such as the builder paying some of the home purchasers' points for loans, etc.
- 23. Land releases at closing: This line reflects those instances where it is necessary to make additional payments on the land in order to release lots being sold (see Line 13 and 27-33).
- 24. Loan payment per lot: The amount of the construction loan attributable to each lot being sold, usually representing more than 100% of the value of such improvements, which is repaid to the institutional lender when each lot is sold and released from the construction loan deed of trust.
- 25. Total loan payment: This line reflects Line 19 multiplied by Line 24.
- 26. Net cash from sales: The net amount of cash received from closing of sales of houses, being Line 21 minus Lines 22, 23, and 25.

**Land Acquisition:**

- 27. Scheduled payments: This line represents the schedule agreed between the seller and the purchaser for how the price of the land would be paid, without regard to the pace of sales of houses. This schedule is agreed at the time the contract of purchase of the land is written.
- 28. Releases allowed: This line calculates how many lots would be allowed to be released from the deed of trust based on the payments schedule initially agreed.
- 29. Releases taken: This line shows how many lots are required to be released based on the pace of sales.
- 30. Balance of released lots: This line compares Line 28 to Line 29 and indicates either (a) that sufficient funds have been paid to release the required number of lots, or (b) that additional funds must be paid in order to obtain release of the required number of lots.
- 31. Additional releases required: Indicates when additional funds must be paid in order to obtain release of the required number of lots.
- 32. Additional payments for releases: Line 31 multiplied by the release price, in this example \$83,750 per lot or 125% of the initial purchase price of the land per lot.
- 33. Balance due: Balance due to the seller to pay for the land. Note that the full land price (\$21,440,000) is fully paid by a combination of Line 27, Scheduled payments (\$16,549,000), plus Line 32, Additional payments for releases (\$4,891,000).
- 34. Interest on average balance: Interest paid on the remaining unpaid balance of the land price.

**Lots Sold:**

- 35. Lots sold: The number of lots sold each year.
- 36. Lots remaining: The number of lots remaining unsold at the end of each year.

**Construction Loan:**

- 37. Construction Draws: The amount of funding from the institutional lender each year to cover agreed construction costs, in this example, 100% of land development, house construction, marketing and interest on the loan (Line 12).
- 38. Interest Draws: The amount of interest due on the construction loan is allowed to be drawn under the loan.
- 39. Payments: Payments made against the construction loan when houses are sold, usually based on an agreed formula or amount negotiated in the construction loan documents.
- 40. Balance due: Remaining balance outstanding on the construction loan.
- 41. Interest: Interest paid on the construction loan.



**TABLE 10.2 Sample Economic Feasibility Analysis: Residential Project for Sale (Continued)**

All figures shown in thousands of dollars  
 \*\*Average cost or income per lot expressed in whole dollars

NOTES	YEARS:	UNIT COST	1	2	3	4	5	6	7	8	9	10	TOTALS	AVERAGE PER LOT **
<b>Equity</b>														
42	Total Expenditures		\$520	\$220	\$12,409	\$15,330	\$14,021	\$13,511	\$14,193	\$13,919	\$10,119	\$3,936	\$98,177	\$306,804
43	Loan Draws		\$0	\$0	-\$7,122	-\$10,029	-\$9,132	-\$8,711	-\$8,754	-\$8,795	-\$8,846	-\$3,618	-\$65,007	-\$203,146
44	Net Cash from Sales		\$0	\$0	\$0	-\$5,818	-\$6,617	-\$7,221	-\$6,951	-\$6,491	-\$9,729	-\$4,500	-\$47,328	-\$147,900
45	Capitalize required return		N/A	\$26	\$64	\$344	\$603	\$538	\$388	\$237	\$125	\$0	\$2,326	\$7,268
46	Balance of Equity		\$520	\$765	\$6,117	\$5,944	\$4,820	\$2,937	\$1,812	\$682	-\$7,649	-\$11,832	\$0	\$7,268
47	Required return	10.00%	\$26	\$64	\$344	\$603	\$528	\$388	\$237	\$125	\$0	\$0	\$2,326	\$7,268
48	Equity Partner's additional return	15.00%	\$39	\$96	\$516	\$905	\$807	\$582	\$356	\$187	\$0	\$0	\$3,488	\$10,901
<b>Taxes on Income</b>														
Per Lot														
49	Sales Income		\$0	\$0	\$0	\$17,364	\$18,233	\$19,144	\$20,101	\$21,107	\$22,162	\$9,308	\$127,419	\$398,184
50	Land		\$0	\$0	\$0	-\$3,350	-\$3,350	-\$3,350	-\$3,350	-\$3,350	-\$3,350	-\$1,340	-\$21,440	-\$67,000
51	Development Costs		\$0	\$0	\$0	-\$1,857	-\$1,857	-\$1,857	-\$1,857	-\$1,857	-\$1,857	-\$743	-\$11,885	-\$37,141
52	House Costs		\$0	\$0	\$0	-\$7,000	-\$7,210	-\$7,426	-\$7,649	-\$7,879	-\$8,115	-\$3,343	-\$48,622	-\$151,944
53	Interest on Loan		\$0	\$0	-\$641	-\$629	-\$537	-\$407	-\$281	-\$158	-\$40	\$0	-\$2,693	-\$8,417
54	Interest on Land		\$0	\$0	-\$1,458	-\$1,336	-\$1,093	-\$841	-\$543	-\$208	-\$20	\$0	-\$5,499	-\$17,186
55	Equity Investor's Return: Annual		-\$26	-\$64	-\$344	-\$603	-\$538	-\$388	-\$237	-\$125	\$0	\$0	-\$2,326	-\$7,268
56	Equity Investor's Return: Accrued		\$10,901								-\$3,488		-\$3,488	-\$10,901
57	Other annual expenses		-\$25	-\$20	-\$1,104	-\$1,327	-\$1,143	-\$1,065	-\$1,033	-\$987	-\$941	-\$393	-\$8,037	-\$25,117
58	Selling Costs		\$0	\$0	\$0	-\$1,389	-\$1,459	-\$1,532	-\$1,608	-\$1,689	-\$1,773	-\$745	-\$10,194	-\$31,855
59	<b>Net Taxable Income</b>		-\$50	-\$84	-\$3,547	-\$128	\$1,045	\$2,279	\$3,543	\$4,855	\$2,577	\$2,744	\$13,234	\$41,356
60	Income Taxes (Paid) or Credits	40.00%	\$20	\$34	\$1,419	\$51	-\$418	-\$912	-\$1,417	-\$1,942	-\$1,031	-\$1,098	-\$5,294	-\$16,542

**TABLE 10.2 Sample Economic Feasibility Analysis: Residential Project for Sale (Continued)****Equity:**

42. Total expenditures: The total amount spent on the project, Line 18.
43. Loan draws: Required equity from the investor is reduced by the construction loan draws used to fund costs, Lines 37 and 38.
44. Net cash from sales: Required equity from the investor is also reduced by the net cash from closing of sales, being Line 26.
45. Capitalize required return: The required return (calculated at Line 47) for the prior year has been added to the outstanding balance of equity owed, such that interest is compounded including prior years' interest.
46. Balance of Equity: At any given time, the balance of funds owed the equity partner equals Line 42, Total expenditures; minus funds from the construction lender shown on Line 43, Loan draws; minus Line 44., Net cash from sales; plus capitalized required return, Line 45. This balance of equity is earning the required return and is the total investment on which the overall return is calculated.
47. Required return: In this example, the equity partner's required return is 10.00% per annum—this line is 10.00% of the balance shown on Line 46.
48. Equity partner's additional return: In this example, the equity partner requires an additional annual return of 15.00% for each year funds are invested—calculated on the balance shown on Line 46. This portion of the return is not capitalized (therefore, the interest is not compounded); however, it must be paid before any other parties in the joint venture can calculate profit.

**Taxes on Income:**

49. Sales income: Gross income, Line 21.
50. Land: Purchase price of the land, divided equally among the total number of lots, and deducted from income as the lots are sold.
51. Development costs: Lines 1 through 8, divided equally among the total number of lots, and deducted from income as the lots are sold.
52. House costs: The total of all house costs on Line 11, deducted from income as the houses are sold. (This program assumes the houses sold in any given year are those constructed in that same year.)
53. Interest on Loan: Line 41, interest on the construction loan, is deducted in each current year, as incurred.
54. Interest on Land: Line 34, interest on the remaining balance of the land price, is deducted in each current year, as incurred.
55. Equity Investor's Return: Annual: Line 47, deducted in the year incurred (even though it is capitalized rather than paid current).
56. Equity Investor's Return: Accrued: Line 48, deducted in the year paid (at the end of the investor's participation).
57. Other annual expenses: The totals of Line 9, Marketing/advertising; Line 10, Taxes and insurance; Line 14, Overhead and supervision; and Line 15, Developer's fee, deducted from income in the current year. Although practice can vary on the sequence of charging these items against income, by the end of the project the result is that all such costs have been charged against income.
58. Selling costs: Line 22, deducted from income as incurred.
59. Net taxable income: Line 49, minus the sum of Lines 50 through 58.
60. Income Taxes: This example uses a blended tax rate of 40.00% to reflect 35% federal taxes and 5% state taxes on ordinary income. Of course, this will vary depending upon the form of ownership, and the tax rate of the owners.

other opportunities at hand. In any instance, opportunity costs should be represented as a baseline cost—for the equity dollars invested, the interest not earned on those dollars has an identifiable value and should be considered a cost of the project.

In addition to evaluating the income potential and cost structure, the investment analysis usually looks at financing alternatives. The right combination of debt and equity given the development and sales climate is essential. In many cases with residential projects, lenders involved with the short-term development financing view this as an entrée to the long-term home mortgage market and structure programs for financing sales to home buyers that all but eliminate closing costs, thus becoming an incentive to the sales process. The requirement for up-front funding of predevelopment and infrastructure costs makes it necessary for significant construction draws prior to the first possible sales or creation of an income stream.

Timing and a thorough understanding of the interrelationships in the process to bring the property to market also play a critical role in the investment analysis. The developer must estimate the timing of expenditures as well as the income from product as sales are made, each month, quarter, and year through ultimate completion of sales or leasing. Understanding the critical path of decision-making is vital to keeping the project on track and not wasting carrying costs idling with a lag time because applications were not filed early enough or materials not ordered sequentially. As the project progresses and construction and carrying costs increase, having laid a solid foundation in the feasibility period can mean the difference between a project delivered on time and within budget, versus a financial failure for lack of early discovery and project management.

## ECONOMIC FEASIBILITY ANALYSIS

The final economic feasibility analysis combines the market analysis and the marketability study with development and construction costs, debt structure, and required rate of return, then tests the outcome. Institutional investors (e.g., insurance companies, pension funds, investment trusts) generally have fairly well-developed criteria. Required financial returns are generally based on returns available from other investments or their prior experience on other projects. If the standard statistical analysis does not yield the hope of achieving the required return (hurdle rate), the investment is not made. At the other end of the scale is the small to mid-sized local or regional developer/builder, whose decision is more based on a gut feel for the marketplace and previous success with particular product types. The early form of feasibility study is looking for the deal killer so that one can stop spending money sooner rather than later on a project that does not show promise of meeting the overall return criteria.

Who does the economic feasibility analysis? Some such analyses are performed by real estate consultants, appraisers, brokerage firms (for residential product, those who specialize in sales of new homes often do this), and some

accountants have the financial background to do this. Most often, though, the final analysis comes from the developer/builder themselves—those who have spent the time in the field and know their individual product type, the process, the cost structure, and sales or leasing patterns.

With larger developer/builders, the decision tree generally follows this pattern:

1. What are the financial goals?
2. What is the tested product type?
3. Where is the desired location in which to build?
4. Find the property, buy and develop it at a cost that is within the acceptable financial guidelines.

Conversely, with smaller and mid-sized regional developers, the process often is more driven by availability of land and the local political and community environment, which has the effect of reversing the critical path of the process:

1. Where are the opportunities?
2. Find available land.
3. Determine what can be built on it.
4. Define a product and control the costs in such a way to make it profitable.

In all development projects, *location*, *timing*, and *cost control* are the critical elements that must be evaluated carefully and exercised constantly in order to deliver a financially successful project. The well-known axiom that “Location, Location, Location” is the most critical factor is certainly equaled by the admonition that “Time, Time, Time” can be the greatest enemy. Many projects have fallen victim to financial disaster because of changing markets, the ever-present interest and carrying costs, fluctuating cost of money, rework of improperly designed or constructed features, and poor project management. The only solution is to identify, during the feasibility period, the greatest number of variables and learn as much as possible about their impacts on the project, then schedule and control the process in such a manner that, when the surprises come (and they WILL come), they do not destroy the overall profitability of the project.

The following basic example demonstrates how these elements come together in the investment analysis.

### Example: Multiyear Residential Development Assumptions

*Land.* Approximately 30 acres of property zoned for 1 unit per acre but comprehensive planned for up to 12 units per acre. The land will be purchased upon completion of zoning and site plan approval, which will take two years to complete. Payment for the property will be as follows: \$250,000 deposit at risk after the feasibility period (four months), 15% down payment at closing, interest at 8% per annum on the remaining balance, principal payments due

in six equal annual installments thereafter. Lots will be released from the deed of trust at price equal to 125% of the per-lot value. Annual payments count toward releases, and funds paid for releases count toward the annual payment next coming due. Purchase price of the land is \$67,000 per lot approved under the site plan.

**Property Conditions.** The land is dramatically sloping such that it is ideal for garage townhouses, but requires retaining walls and significant stormwater management techniques to discourage wet basements and other adverse water conditions. All utilities are at the lot line, but significant street improvements are required along the property frontage. The total lot yield on the property ends up being 320 lots due to the constraints of the topography and the requirements for setbacks and other community features.

**Product.** Market studies indicate demand for townhouses to sell in the range of \$300,000 during the year 2001, with an expected sales rate of 50 units per year. Escalation of values in the marketplace for this product have averaged 5% per annum during the past 10 years, although such escalations are somewhat unpredictable in their pattern of year-to-year increases. A pool, two tennis courts, a playground, and a small clubhouse with showers and restrooms will be constructed in a central location.

**Financing.** A lender will advance 100% of the costs of land development, house construction, marketing and interest on the loan, up to 70% of the sale value of the houses under construction at any given time. The loan bears interest at the prime rate plus 2%, and prime has averaged 7% during the past five years. The lender requires that land acquisition costs and interest on the land, overhead, and supervision be funded from equity.

**Equity.** The builder/developer is a regional firm backed by an institutional joint venture partner that requires a minimum equity return of 10% per annum to be capitalized annually, with a target overall return of 25% per annum for the duration of time the funds are invested. The developer/builder will be allowed a developer's fee paid on a current basis equal to 3% of the hard costs of the project. After payment of the 25% per annum return to the Equity Partner, the remaining profits go to the developer/builder.

**Tax Assumptions.** It is assumed that the ownership entity is a partnership, limited liability company, or Sub-S corporation that is engaged only in this project as its business purpose, and that the entity will be taxed at the individual tax rates for ordinary income at the federal and state levels. For the purposes of this analysis, a blended rate of 40% is used.

#### ADDITIONAL READING

Bookout, Lloyd W., Jr., *Residential Development Handbook*, 2d ed., ULI—the Urban Land Institute, Washington, D.C., 2000.

Galaty, Fillmore W., Wellington J. Allaway, and Robert C. Kyle, *Modern Real Estate Practice*, 15th ed., Real Estate Education Co., Chicago, 1999.

Kone, Linda D., *Land Development*, 9th ed., National Association of Home Builders, Washington, D.C..

Lewis, Ralph M., *Land Buying Checklist*, 3d ed., National Association of Home Builders, Washington, D.C.

National Association of Home Builders (NAHB), *Land Development Manual*, NAHB, Washington, D.C., 1974.

Sinclair, Joseph T., *Real Numbers: Analyzing Income Properties for a Profitable Investment*, Richard D. Irwin, Homewood, Ill., 1993.

Sirota, David, *Essentials of Real Estate Investment*, 7th ed, Real Estate Education, Chicago, 2001.



# PART II.C REZONING

## CHAPTER 11

# THE REZONING PROCESS

*Lawrence A. McDermott*

### INTRODUCTION

Several primary objectives guide the land development design team during project formulation and design. Chief among these is the client's mandate that the plan be efficient in its use of land and cost-effective in the use of other resources. The plan must also result in a marketable project that appeals in both cost and layout to the developer as well as the potential user. The plan must be able to receive local government approval. Sometimes these plan/design objectives compete with those of both local officials and neighborhood residents. As a result, approval procedures for development ventures influence the project result as much as the local standards and principles used in project design and can significantly affect project appearance, finance ability, economic return, design budgets, land costs, development costs, consumer costs, and market appeal during the development review process. The most satisfactory plan for the developer, no matter how well designed, is not necessarily the best plan in the community's eyes, nor is this always the plan that gets approved.

Although this chapter focuses mainly on the approval process at the local level, the reader should be astutely aware that state and federal agencies might also need to review and approve specific aspects of the project. Depending on the nature of the project, state agencies such as the Department of Highway and Transportation or Department of Health may need to be included in the approval chain. Additionally, the Environmental Protection Agency (EPA) and Corps of Engineers (COE) may have to affix their stamp of approval and issue permits as well (please see Chapter 6 for additional discussions on this topic).

Community involvement in the review of development proposals increases the likelihood of change from the pro-

ject's original form. Residents' involvement typically surfaces during rezoning and special exception procedures. Opposition to a given proposal is often in response to concerns regarding the loss of privacy, open space, or wooded play areas. Residents frequently express fears about traffic congestion, potential flooding, damage to sensitive environmental features, or school capacity. During the public hearing process, these concerns, even if exaggerated or resolved by the proposal, can instill lingering doubts that can destroy or dramatically alter a project.

Similarly, residents who are predisposed against a rezoning or special exception application frequently succeed in gaining input in project design. Not normally subject to discretionary review, they have ample opportunity to ask elected leaders to intervene on their behalf. As illustrated in Chapter 44, "Case Study—Union Mills," the growing sophistication of citizens and their power to influence all development decisions often leads to stopped or delayed developments. Despite legislation affording property rights, the way in which the public is involved in the design and review process can influence the proposal's success or failure. Although the court often rules to uphold property rights, a lengthy delay and the expense of litigation can make it a hollow victory. Project engineering and carrying costs as well as project finances can imbalance investment and expected return. The developer may face a changed market that needs a different product. Or worse, the potential end users may no longer be able to afford the cost of purchase or lease.

For these reasons, the manner in which the development team approaches the rezoning and special exceptions review process, with its roots in government and public relations, is as important as his approach to the technical process of

analysis and design. The land design team acts as a bridge between the developer and the other participants in the land development process, including the governing body, other review commissions, the jurisdiction's administrative staff, and community residents. How effectively the land design team manages this responsibility is critical to a project's acceptance and approval. Communication links must be established and maintained throughout project design and review. Since communications between the other parties in the process cannot be controlled, the development team must develop rapport and support within each entity to create advocates for its plans, ideas and objectives. This chapter discusses the analyses and procedures required to accomplish that goal.

### TECHNICAL ANALYSIS OF REZONING

A developer's intentions frequently differ from the land use allowed by the zoning district in which the property is located. Consequently, the parcel's zoning must be changed before development can proceed. To accomplish this, the developer must demonstrate, through protracted and complex analytic and legislative procedures, that the proposed zoning is suitable for the property and that the proposed use is appropriate for the community. Even when the comprehensive plan recommendations support the zoning change, the developer must earn the support of neighborhood residents for a project that might dramatically alter their community, which can be a formidable task.

Success in the rezoning process depends largely on the technical analyses performed by the land design team. However, consideration of the rezoning proposal frequently takes place in a politically and emotionally charged atmosphere. Private sector motivations and public sector policies often come into conflict with each other and with residential citizens' concerns and desires. Therefore, the land design team plays an important role in building strong community relations throughout the project. The rezoning process represents a major up-front expense to the land developer and comes with few guarantees that the goals of that investment will be realized. In view of the legal ramifications of rezoning and constitutional concerns for due process, the development team relies heavily on its legal consultants throughout the process. Legal advice is important in ensuring that technical procedures are adhered to and that government actions are consistent with the requirements of state and local laws. During the rezoning process, it is typical for the legal advisors to take the lead.

### Reasons for Rezoning

From the land developer's perspective, the most prevalent reason for rezoning is to increase the land use intensity for higher yield and greater profit potential. Developing a tract to the maximum density permitted by its existing zoning does not necessarily mean profits to the developer. In fact, in some instances development at existing zoning is not economically feasible. In a typical economic market, the

highest and best use of a property is a critical factor in setting land value and prices. This land value is established based on the scarcity of developable land in general and the demand for property of similar potential. The community's comprehensive plan and zoning ordinance play an important role in determining highest and best use. For example, a community's comprehensive plan might suggest that a 20-acre parcel of ground located near a commercial area should be developed at a multifamily residential density of perhaps 15 dwelling units to the acre. Although existing zoning allows one dwelling per acre, based on the comprehensive plan recommendation, the land's highest and best use becomes multifamily. Its value is based on the demand for multifamily development, the scarcity of multifamily zoned land, and the potential yield of the land.

Other motivations exist for seeking rezoning approval. As noted in Chapter 2, comprehensive plans often set a range of densities and uses, rather than a finite density. In evaluating the plan to determine potential uses, a feasibility analysis may suggest land uses for which there is considerable market demand in the area. The situation may exist where an intended use may be more compatible to the surrounding area than existing uses or uses permitted by existing zoning. Further, the developer may wish to rezone because of a preference for or bias against a particular type of development.

### Background Investigation

The development team's initial step in preparing an application for rezoning is to conduct an analysis of the community's comprehensive plan. Many jurisdictions require a finding that the proposed zoning and subsequent development comply with the provisions of the comprehensive plan even if under the law of the jurisdiction the comprehensive plan is a "guide" and not a "mandate" for specific development. Absent support in the plan for the new zoning classification, it is unlikely that the governing body would approve the rezoning application. Approval of a rezoning that is inconsistent with the comprehensive plan invites legal action by enraged citizens against both the developer and the public officials who adopted the zoning change.

As noted in Chapter 2, adoption of the comprehensive plan is the culmination of extensive analysis and community participation. Consequently, findings and recommendations play a central role in defining the development program. Where the plan has recently been revised, it offers a strong perspective of current community opinion. These attitudes, towards both development in general and the client's property in particular, will be raised during the review of the rezoning application by the approving authority. Neighborhood residents are extremely diligent in demanding that their elected officials apply literal interpretations of comprehensive plan language that they had a hand in developing. The land design team must anticipate, respect, and accommodate these attitudes.

Preparation of the rezoning application requires that the land design team conduct two levels of analysis of the comprehensive plan: first, an analysis of jurisdiction-wide policies, identifying the constraints and incentives imposed during the development process, and secondly, incorporation of parcel-specific policies that address detailed requirements into the development plan. Typical considerations are discussed below. Although much of this familiarity comes from experience with previous projects, the land design team should maintain an in-house reference copy of the comprehensive plan. This copy should be annotated to reflect recurring issues and themes encountered in previous project plan reviews to aid the land design team in anticipating future conflicts.

Each jurisdiction's comprehensive plan is unique to that community, and a comprehensive listing of the types of policies encompassed in a plan is not possible here. However, the land design team should refer to the following list, which can be referenced as a framework for identifying significant development issues, both jurisdiction-wide and parcel-specific, that must be addressed and resolved by the proposed development. Depending on local practice, these factors offer an indication of the types of studies and the level of detail of analysis required as part of the rezoning application.

- *Development patterns and “futures” scenarios:* These define and distinguish areas where development is both desired and discouraged, usually identifying ranges of development density, intensity, and criteria against which development proposals will be measured.
- *Adequacy of public facilities, growth management and systems management programs:* These often suggest the commitment of financial and/or land resources to internal and external improvements in order to accommodate proposed development, including the location of such facilities, the timing or phasing of construction, and the impact of these factors on the development proposal.
- *Environmental and cultural preservation:* The identification of the type of natural and man-made resources given priority by the community suggests the type of investigation needed to identify such resources and the method of protection and preservation to be used. Such criteria often affect project yield by reducing buildable land area and impact project design and construction.

The comprehensive plan's geographic and functional components also provide information directly related to the client's property. This information refers to specific circumstances that must exist or conditions that may be applied in reviewing development proposals. The plan's recommendations for individual parcels consider impact on the com-

munity and surrounding properties. Several factors play a role in assessing that impact and determining future use. Depending on parcel size, the ability with which development can be controlled by either the developer and jurisdiction varies. As that control increases, usually with larger properties, the developer has broader opportunities for incorporating features that make the project compatible with its surroundings. The comprehensive plan may recommend a range of land use intensities and establish conditions under which the jurisdiction considers different proposals. For example, the plan may call for a residential use range from five to eight dwelling units per acre, allowing the high range of eight DU/AC only with the satisfaction of specific development conditions.

As part of the background analysis, the land design team simplifies project design by assembling a checklist of comprehensive plan guidelines. This checklist is instrumental in preparing the rezoning application, particularly so far as it raises issues of concern to citizens and community groups that must be addressed. Application format may vary significantly from jurisdiction to jurisdiction; however, typically included is basic information as to ownership, location, existing and proposed zoning, and justification for the zoning change request. The planning staff, governing body, and civic groups will consider all issues during public hearing testimony. Early identification of these issues by the land design team provides the opportunity to consider and resolve potential conflicts and/or contradictions with the comprehensive plan.

The following list includes typical conditions and specifications that the land design team can expect to find in the plan:

- Base density or intensity recommendations for the client's property.
- Unique conditions imposed to reduce impacts, both created by or imposed upon the development. These may include:
  - Screening, buffering, open space or landscaping, requirements beyond the minimum to protect project or adjoining properties
  - On-site requirements for gradation of density;
  - Special requirements for preserving on-site historic, heritage, geological or archaeological features
  - Setback requirements from major roads, parks, or other features which exceed the minimum specified in the zoning ordinance or other regulation
  - Sound attenuation requirements
- Conditions imposed to justify increases in allowable density or intensity beyond base recommendations. These include:
  - Parcel consolidation to bring small tracts into a larger parcel under a single developer's control



- On-site improvements, intended primarily for occupants of the project, beyond those normally required
- On-site amenities and services such as unique recreation facilities, open space, day care, public art
- On- and off-site improvements or dedication of land or contributions for public facilities and services, such as roads, schools, parks, utilities, and housing
- Innovation in plan design or a mix of uses intended to make the project less reliant on existing community infrastructure
- Innovations in layout and structural design to minimize the impact of off-site nuisances, such as noise
- Criteria concerning the relationship between the proposed project and other land uses that would support and serve the development.

### Comprehensive Plan Amendment

Sometimes the comprehensive plan is outdated and the development team may believe that it no longer accurately reflects the community or reflects good planning principles. This occurs when a significant amount of time has passed since the adoption of the existing plan. In such instances, the land developer may have an opportunity to seek an amendment to the plan to reflect conditions that have changed since its adoption. Although the plan amendment process varies greatly from jurisdiction to jurisdiction, the opportunity to amend the plan will be available in some form.

Specific authorization to undertake a comprehensive plan amendment might require a vote by the governing body. Typically, the actual process is not unlike the one the community follows in originally adopting its plan with the primary difference being that it is focused on a very specific geographic area and is primarily land owner-initiated. It should be noted, however, that in some jurisdictions anyone can nominate a plan amendment whether or not they own the land subject of the proposed plan amendment. In undertaking a plan amendment, the land development team prepares an economic and demographic inventory of the vicinity. The proposed revisions to the plan, both maps and text, are prepared, along with justification for the change. When the proposed amendment is prepared, notice is given to the community through mailings, advertisements, and posted signs. The draft amendment is reviewed by the professional staff and will be the subject of meetings and public hearings before the community's planning commission and governing body. Adoption of the amendment enables the developer to continue with the rezoning phase in accordance with the new plan recommendations.

This process is lengthy and uncertain and can add as much as a year to the development process. However, often the plan amendment and rezoning processes are conducted concurrently. Absent an expression of initial support from the staff and governing body, it is usually not a wise investment of resources to proceed with an out-of-turn plan amendment. In such cases, it is prudent to wait for the community's normal plan amendment process to introduce the proposed amendment.

### ADMINISTRATION OF THE REZONING PROCESS

The rezoning of property is a prelude to relatively permanent alteration of a community's physical and economic structure. In view of its importance, and due to the responsibilities and obligations prescribed by local and state legislation, analysis of a rezoning application requires an intense commitment on the part of local government. The governing body must be assured that the zoning change is in the best interest of the public health, safety, and welfare of the general public and is consistent with the guidance adopted in the community's comprehensive plan. The governing body must be satisfied that development proposals accommodate the concerns raised by its constituency and resolve the issues outlined by its professional staff. In many cases, constituent concern outweighs issues identified by staff.

Due to the level of analysis required, zoning applications present urbanizing communities with a tremendous workload. This is true not only for the staff, but also for the elected and appointed officials who must review and render recommendations and decisions on the proposal. To manage this workload, jurisdictions often limit the time frame within which rezoning applications can be received and heard, which facilitates the allocation of review time and scheduling of meetings. Applicants who fail to meet a prescribed deadline are forced to wait for the next period to file applications. In addition, many jurisdictions establish a waiting period before previously denied rezoning applications that have since been amended can be refiled. This prevents the jurisdiction from being burdened by the need to conduct frequent reviews of projects submitted for the purpose of testing the waters and finding the right formula to win approval. This limitation on refiled also forestalls the submission of frivolous applications.

The development team may have only one opportunity at presenting its proposal. Further, typically a year or more can pass between the time of the applicant's filing of the rezoning request and the governing body's decision. In view of the time and expense involved in the rezoning process, the development team must take maximum advantage of the singular importance of this opportunity. This mandates overcoming potential obstacles and using all available resources in formulating and justifying the development program. Those who will review submission materials, those who approve the rezoning, and those who live near the project are perhaps the three most important resources

available to the development team. The approach that has proven to be most effective in project development is the early involvement of these groups in determining a site's development potential and the issues, problems, and opportunities for resolving conflicts. As stated earlier, processing of a rezoning application without the early inclusion of these three groups invites opposition and increases the likelihood of failure.

### The Professional Staff: Decision Influencers

Typically, in a growing jurisdiction the governing body will delegate the responsibility for performing the rezoning applications to the professional planning staff. The staff also manages the application review process, which involves distributing of applications to other government agencies and assembling responses for presentation to the advisory (planning commission) and approving (governing body) authorities. In some cases, the planning staff is in a position to resolve conflicts among agency reviews. In addition, the staff will frequently engage in preliminary negotiations with the development team.

As is often the case in larger communities and in those broken into smaller planning areas, staff members are assigned to each of the community's geographic areas. Each may play a specialized role as coordinator for all planning and development activity. These individuals frequently oversee dissemination of information and respond to public inquiry on land use, zoning, and development issues in the assigned area. These activities place the staff in a unique position and establish them as an important resource in project formulation.

The staff offers a critical communication link between the development team and the other important process participants, and consequently the team must take advantage of this valuable resource. Staff members have access to the historic records concerning prior zoning and/or development actions. Further, these professionals are often in direct communication with elected officials whose constituency is affected or concerned about development proposals. Seen as advocates for private citizen issues, staff members are frequently sought after by community residents and other civic groups. Many such groups have become diligent in monitoring development proposals and equally diligent in attempting to build allies as a first line of defense in protecting their neighborhood. Lastly, the staff is responsible for administering and interpreting comprehensive plans and regulatory policies and ordinances as they relate to each development proposal.

The staff can be either a powerful advocate or a difficult adversary. A staff member who becomes enthusiastic about a proposal will often press that support in meetings with the planning commission, governing body, and even civic groups and influence the incorporation of his ideas into the plan. The opposite is also true in that the professional opinion of the staff is often made without the influence, or burden, of political and economic needs. The commitment to

uphold that opinion can be strong. In addition, it might be axiomatic to suggest that the lower-ranking public staff, those likely to be directly involved in coordination and plan review, often assume the greatest control over development submissions. Although there are ample opportunities to have decisions of these professionals overruled, with both administrative and legislative actions this can be counterproductive. The relationships established among staff members, the land developer, and the land design team extend well beyond the geographic and calendar boundaries of a single project. Through development of strong relationships with the staff, information is more readily obtainable concerning the attitudes and desires of community residents and elected officials. In addition, the staff is usually more receptive to proposals when members of the development team are perceived as forthright, knowledgeable, cooperative, accommodating, and understanding.

In working with the professional planning staff, the following questions should be asked early in the project design phase, preferably as part of the project feasibility study:

- Have there been recent rezoning or development proposals concerning the subject property?
- Has the property been considered during recent comprehensive plan reviews?
- In either case, what issues have been raised by civic leaders, elected officials, or government agencies?
- Are the community's residents organized and united in their concern and consideration of development proposals in their neighborhood. More specifically, who or what organization appears to take the lead?
- Does the governing body or planning commission have members who are elected or appointed by district; have those members established a formal or informal network for receiving input from community residents? Have these individuals been in contact with the planning staff to discuss the subject or nearby property?
- What constituent issues or complaints relative to land development in general appear to be commanding the attention of elected or appointed officials?
- What specific recommendations does the staff member have for the future development of the property?
- What are the most critical issues that must be resolved by the rezoning application and development proposal?
- Is the staff member aware of other development proposals being considered or reviewed in the vicinity of the subject property?

The answers to these questions will provide the critical historical perspective needed in considering the development potential and design of the property, help focus attention on those issues that could terminate or delay a project and provide important clues concerning the local leadership and decision-makers in the community. Finally, by directly involving the planning staff in the project, the staff becomes part of the team, whose opinion is important and assistance valuable. To the extent that those opinions are addressed in project design, the professional planning staff may feel obligated to help in the support of the proposal in subsequent public forums.

Staff, other than the professional planners, are also involved in application review. In each agency that comes in contact with the plan, individuals familiar with specific issues relative to their expertise and program also are important sources of information and potential support.

### **The Political Leaders: Decision-Makers**

The community's governing body is responsible for devising and implementing policies and strategies to manage the economic, demographic, social, and environmental resources. With respect to land use and development issues, the governing body must distinguish between development proposals that provide the greatest benefit to the community and those that impose the greatest costs. At the same time, unique relationships exist between the community's elected leadership and its citizens and those in the business and development community. The governing body and those whom it appoints to advise and implement policy, such as the planning commission, have a responsibility to consider the needs and attitudes of both of these groups. The fact that they are elected and their terms in office determined by the satisfaction of the electorate, often poses a significant dilemma in making land use decisions. However, many take their leadership role seriously and are willing to use that role to build support for development proposals they believe to be in the community's best interests.

The timing of local legislators' formal involvement in the rezoning process varies among jurisdictions. However, the development team should seek informal opportunities to meet with members of the governing body early in the feasibility analysis, particularly for large and potentially controversial projects. Elected officials do not like surprises, therefore, the earlier they are informed of a development proposal the better.

When such opportunities become available, the team's primary objectives are twofold. The first is to provide the official with information on the project and opportunities for the community. The second is to learn from the official about outstanding concerns he or she may have, as well as to attempt to identify any predisposition towards the use of the property. Such meetings can become useful in determining if the official is likely to take an advocacy role in the process or, conversely, to oppose the rezoning for whatever reason.

### **Assessing the Community**

Public hearings that have community residents and the development team standing shoulder to shoulder in support offer the greatest potential for successful rezoning applications. The most skillfully designed and economically sound plan can fail if the jurisdiction's elected leaders see that community support is lacking. The support or opposition of neighbors and residents is frequently a measure of how well the team communicates throughout the project design and application review process. As with the staff relationships, the communication links and established levels of trust will extend far beyond the current project's boundaries. Establishing this relationship of trust is particularly important where projects are multiphased or where the development team expects to undertake additional activity in the vicinity.

Initially, community fact-finding must be undertaken. The first step in this activity is to determine the dynamics of local community leadership to identify those community leaders whose opinions are sought after, both by other residents and by the jurisdiction's elected leadership. These community leaders are the people whose actions have impact on the opinions of their neighbors. Ultimately, these will be the people with whom the development team must work to establish clear lines of communication. These community representatives will almost certainly become involved in the development process, whether or not their participation is at the urging of the development team. Therefore, it falls to the team to seek out these individuals, with the goal of aligning favorable attitudes towards the development program. This is most successfully accomplished by involving the community early in the design process.

How community leaders share information, opinions, and attitudes with other residents, and how residents communicate with each other, must also be determined. Whether the residents are organized under formal homeowners or community associations or are unstructured, with various individuals taking active roles on ad hoc issues, should be determined. Homeowner associations are organizations formed by developers of new subdivisions to enable residents to manage community open space or maintain a community's amenities and common elements. As these communities become fully operational, the developer's input in the organization lessens and ultimately is eliminated. These associations soon become forums for the exchange of opinion on a myriad of internal and external issues that may affect the community. This is particularly true in jurisdictions encompassing broad geographic area, such as counties, where community identity is not tied to incorporated political boundaries. Smaller associations may be linked in a hierarchical network of larger umbrella associations formed for the express purpose of monitoring development activity and government actions. In addition to organizations representing community residents living

within a single development, they may also represent broader geographic subareas of the jurisdiction. This is often the case in incorporated towns or cities.

If the development team is not already familiar with the active community groups and outspoken community leaders, the planning staff can usually provide assistance in the identification of these key players in the development process. This knowledge is based on their experience from prior contacts during previous comprehensive plan reviews or nearby zoning or subdivision activities. In addition, the planning office or jurisdiction's office of community affairs often maintains a listing of formal community associations. These groups often request to be notified about pending development applications or proposed legislation that may affect their community.

Members of the governing body or planning commission may also be approached for information about community leaders. Sometimes council, board, or commission members are elected or appointed to represent specific wards or districts within the community. If this is the case, they often have established a formal organization that advises them on development or legislative matters.

The files of previous development applications, not only for the specific property but also in its general vicinity, are a valuable resource for the development team. Given the freedom of information, or "sunrise" laws, these files are generally open for public inspection. They often contain transcripts from previous public hearings, lists of those who have provided testimony, and letters from neighbors and groups who felt moved to comment about the proposal. File records may also contain staff memos concerning meetings with interested parties. All potential sources of community interest must be identified. Residents often are not satisfied with the answers they may receive from their local elected leaders. It is not uncommon for them to approach their elected state and even federal legislators to ask for intervention in a local issue. Members of the development team may wish to contact the offices of these individuals to learn more about opinion leaders and their concerns.

Such research takes time, a luxury not always available to the development team, which often must act quickly during the feasibility phase of a project due to land purchase contract or other time constraints. The land design team will find it helpful to compile and maintain permanent office records about neighborhood and community leaders. This list should include their geographic area of interest and typical concerns expressed about development and government administration. However, the team must understand that the location of residents in support or opposition of a development proposal is not always predictable. Attitudes towards protection and progress vary, as do concerns regarding enhanced or diminished property value.

Business operators often become extensively involved in attempting to influence development decisions. It cannot be assumed, however, that employers and owners of businesses will be supportive of development projects, whether

they are commercial or residential ventures. This faction is frequently concerned about competition from new business, compatibility, access, or other conflicts in operation. The word "residents" is used liberally throughout this text. However, a business owner, even one in a business closely aligned with the building industry, becomes a "resident" when a development proposal attempts to become a neighbor. As with other residents, attitudes towards a specific project will reflect the way in which that proposal addresses local issues. Decision-makers are often very sympathetic to the concerns of the community's business interests, reasoning that they have made a long-term financial commitment to the community. The recognition of their contribution to the community's tax base is also an important factor in considering their comments about a proposal.

### **Identifying the Issues: Working with the Community**

The focus and intensity of public reaction to any development proposal largely depend upon several factors that are often beyond the immediate control of the land development team. These factors include the level of frustration with quality-of-life issues experienced by residents, their dissatisfaction with the responsiveness and policies of local government, and even the workmanship of their homes and experience with its builder. Important quality of life issues include traffic congestion, quality of schools, parks, recreation facilities, environmental health, and aesthetics. Of the resources already identified, such as political leaders and staff, rezoning files often prove to be the most easily accessible in identifying concerns with a development proposal that are likely to be raised during the review process.

After the development team identifies the appropriate community organizations and leaders, it must determine how best to approach the residents of the community in which a development project is located. The development team must understand that in general, the public has very little concern about project economics or the role of the economic factors in designing the project. The issue of private property rights become secondary to perceptions about the personal impact and the welfare of not only the immediate community. This is often true even when a development proposal is supported by the jurisdiction's comprehensive plan as a way of fulfilling its goals. Even if the project has the support of the community's elected officials, they will be hard pressed to justify approval when it is contrary to the desires of their constituents. Their tenure in office can be shortened if they habitually ignore the constituents' desires with respect to land use.

At the same time, the objective of the development team is to get the public to accept the project or, at a minimum, to lower the degree of opposition. Although the team may never succeed in having the community fully support the project, it may be successful in guaranteeing that opposition is withdrawn or not expressed publicly. This type of consensus building is called "informed consent" by some public administrators and observers (Institute for Participatory

Management and Planning, 1984, p. 16). It is generated by intentionally drawing in and actively involving all components of the community and others in the decision-making process in the act of plan formulation. If objectors can be made to recognize and acknowledge the development team's earnest attempts to accommodate concerns, opposition to the proposal may be withdrawn.<sup>1</sup>

The communication and education process must begin with the development team itself. Since land development is a complex and extremely involved process, the individuals playing a major role in the project must acquaint themselves with needs and concerns of the neighborhood in which the proposed development will be located. The key individuals involved with the project at this stage, such as the developer, lawyer, planner, engineer, and architect, should be completely familiar with the project. Contact with the public to introduce the proposed project can be initiated in several ways. The exact means of exposing a development proposal to the public depends on the experience of the design team, its familiarity with the area, the complexity of the project, and the organizational structure of the municipality. Therefore, it is important to assemble a development team that is made up of people who are sensitive to the locality's concerns, familiar with local ordinances and administration of policies, and attuned to the political attitudes of the area.

### Preparing for Community Interaction

Before the team approaches neighborhood groups, it is a good practice to initiate meetings with public agencies to present a schematic drawing of the project. Some communities have formalized this phase, calling it a preapplication meeting. The schematic proposal might be a reasonably scaled, preferably single-sheet drawing of the site layout. Unless formal requirements direct otherwise, the drawing should be in very loose sketch format so as to preclude possible interpretation as a firm, final proposal. Sufficient information should be provided to assist the staff in locating the property and getting a sense of its relationship to existing development and potential uses that might be experienced in the vicinity. While the staff may be generally familiar with the community, they often are not specifically familiar with all of its neighborhoods and business areas. Even in small jurisdictions, it is not uncommon for the staff to rely fully on their own maps, aerial photographs, and publications. Therefore, any help the development team can offer in characterizing the vicinity of the project is useful, particularly at this informal stage. The less time it must spend in collecting information, the more time it will have to dispense it.

The following information should be shown on the sketch or included as supporting material in tabular form

with the proposal. Much of this information will be needed later, although in much greater detail, during formal rezoning and development processing.

- A vicinity sketch, showing the location of the property in relation to the larger community.
- Relevant comprehensive plan or zoning maps showing the property and adjoining parcels and lots.
- The parcel configuration, showing boundaries, and the existing topography, with five-ft contour intervals sufficient.
- The transportation network that serves the property, including abutting and nearby roadways.
- Major utility locations, both those that serve the property and major transmission facilities that may traverse the property. Capacity assessments of water and sewer facilities are also helpful if there may be questions concerning adequacy.
- Suggested layout and interior road alignments, showing lots, development sites, major buildings, and other major features, such as open space and preservation areas. The smaller the project, the greater the detail that should be shown. On a small 5-acre residential proposal, the sketch might show a rough road and lot layout. A larger 100-acre planned residential development might only show major road alignment and land bays with an indication of housing type and density.
- Calculations showing parcel size, the maximum development capacity permitted under the existing zoning and the proposed zoning classifications, and the yield expected to be achieved by the developer.

While discussions at the meeting should focus on the merits of the project, the meeting should be approached as a listening session. The development team should attempt to elicit as much information from the staff as possible. The intent is to identify information about possible restrictions relevant to the project, potential problem areas, and what the expected concerns might be of the citizens in proximity to where the project is located. The professional staff is attuned to the public's concerns. Information on the issues of concern of the civic and neighborhood groups will provide the developer a clue on the type of opposition that can be expected and also how the project might be structured to temper this opposition. Knowing what to expect from the citizen groups, the developer and his team can prepare the necessary information and strategy. Design input from the professional staff should also be welcomed. Incorporating specific staff recommendations in the early formulation of a project will help to strengthen staff support throughout the life of the project. Any suggestions incorporated as a direct result of a staff suggestion can provide the impetus for a favorable staff report.

<sup>1</sup> This condition is called SEACA (Substantial Effective Agreement on a Course of Action) by the Institute of Participatory Management and Planning (1986), p. II-2.

### Building Community Consensus

Based on prior knowledge of the community, its leaders, and the subject property gained through experience or background research, the development team may judge the project to be highly controversial. In such instances, it will be advisable to contact the citizen groups prior to sharing the proposal with public agencies. This approach may enhance the relationship, establish good rapport, and build confidence between the development team and the community. Communications with community groups must be open and forthright. The team must guard against any action on its part that diminishes the established level of trust, and it must operate with the explicit intent to listen to concerns and incorporate suggestions where possible.

Eventually, knowledge of the project will be widespread. The main objective is for the development team to be the original source of information so that rumors about a project are not circulated throughout the community. If this scenario materializes, the citizens will be misinformed and develop preconceived opinions. The spreading of misinformation can doom a project before it gets off the ground. The development team must ensure that the community becomes involved as soon as possible and has accurate information on which to base its support and/or opposition.

In summary, notification and participation should be viewed as a parallel, rather than linear, process. That is, one group should not be notified, and its issues addressed, before the next is notified. Dispelling the dissenting attitudes of the public can best be achieved by communication and education. Many decisions on land development projects have to stand the test of a public hearing. Rezoning applications are subject to public hearings, and the project must have the citizens' support in order to have a reasonably good chance of approval by the elected officials. Public opinion on an application is a strong element of the decision-making process. If the turnout at the public hearing is substantial and the groups are organized in opposition, the project stands little chance for approval. Unless the development team has been successful in its community relations, a large turnout is indicative of the resistance that will be presented.

### Special Exceptions

Preparation and review of the special exception application is very similar to that of a rezoning. Application procedures and submission requirements are contained in the zoning ordinance. The most significant difference is that review is most often focused on physical appearance and operation of the use and the municipality usually is empowered to impose conditions on the applicant. The development team must assess the financial and practical impacts of these conditions on the project. Unlike zoning changes, which run with the land, special exception approvals are specific to the applicant, with such approvals usually remaining valid

for a specific period within which construction must begin and be diligently pursued.

As more municipalities seek to narrow the range of uses allowed by right in each zoning district, the distinctions between rezonings and special exceptions have become less evident. However, an important principle of the special exception process is the presumption that uses specified as special exception uses meet the intent of the zoning district subject to certain conditions or performance criteria. This presumption eliminates one of the major obstacles faced in rezoning.

### Variations

Variance application procedures are similar to those established for special exceptions; however, the applicant who is seeking a variance must demonstrate that full compliance with the zoning regulations would impose undue hardship in developing or redeveloping a site. This is accomplished by preparing plans or analyses showing the effect of strict compliance, alternative solutions, and the proposed solution to the ordinance provision. Supporting documentation must include an analysis of adjoining structures to illustrate the effect of the variance on them. It may be persuasive to show that existing development is already developed as the applicant intends. When variances are sought in conjunction with a rezoning application, the jurisdiction may allow the two to be processed concurrently. Otherwise, separate applications are required. Most communities provide pre-printed forms and checklists relating to their procedures, submission requirements, and fees associated with the application. In many jurisdictions, a variance requires a quasi-judicial set of determinations and findings. The applicant and the land use attorney should ensure that the requisite determinations or findings are made to ensure the legal validity of the variance approval.

### PRESENTATIONS

In making presentations at civic association or neighborhood group meetings, the development team should prepare a well-defined agenda. The team members may be technically proficient in their fields of expertise; however, organizing and conducting presentations for citizen group meetings require distinctly different skills. Inherent negative attitudes towards development will not be changed quickly. The team will need to educate the citizens on the project and the process and inspire their confidence. If all members of the development team are known for their good work, the community will be more receptive to the team and the project. Conversely, if the team is viewed as money hungry and as not keeping its commitments or respecting its potential neighbor, negative feelings on the part of the community are certain to surface.

At least one member of the team should be comfortable at public speaking. At such meetings, it is preferable that the development team assign a speaking role to a limited number of its members. Logical distribution of topics for

presentations may not always be the best approach. Some of the development team members will be there as support and to provide answers to technical or procedural questions, as well as to absorb the issues and concerns from the citizens in order to devise solutions for the negotiation details. The team should know who is to speak and when and on what topics and to what degree of detail. Even the attire of the development team during community meetings should be evaluated. The team should consider the intended audience in balancing casual versus formal, sophisticated versus simple, flashy versus conservative, and urbane versus homespun. The objective is for an outsider to gain citizen consent, support, and approval. Rehearsal of the presentation and preparation for likely questions is important. Physical layout of the presentation room, audiovisual equipment, and audience comfort should be planned in advance if possible. Anticipate the size of the crowd and provide for overflow. When the audience cannot see or hear, impatience can lead to heckling. The presentation team must guard against perceptions that information is being withheld or that the team considers the meeting to be a required, not a needed, step in the process. The presentation team should provide for audience feedback. The meeting should be a forum for disseminating information on the project and identifying and addressing potential conflicts.

In organizing the agenda for these meetings, consideration should be given to the type of project that is proposed. Different projects will require different approaches. Although the public agencies and legislative officials might be able to offer advance information on the topics of concern of the citizens, the development team should realize that other issues may come up during the meeting. A preliminary social hour around plans propped on easels can help flush out serious objections that may be raised during the presentation.

The team should prepare to answer questions on items such as

- Density and open space
- Concerns about neighborhood change, influx and spread of high-density residential and commercial areas
- Traffic generation, impact on congestion, neighborhood convenience, the likelihood of cut-through traffic
- Proposed road and intersection improvements, including pavement widths, sidewalks, traffic lights
- Bike paths and trail systems, which are often required by local government but feared by neighbors as an intrusion on privacy and threat to security
- Screening, buffering, and landscaping
- Noise, lighting, signs, and other disturbing activities involved in the operation of the land use
- Architectural style and quality

- Price ranges of the finished product
- School-age children
- Environmental impacts, both to natural areas and to private property
- Storm drainage and erosion-control measures
- Machinery and equipment activity, dust and cleanup during and after construction
- Other topics relating to the economic, physical, and social impacts to the community

Know where the flexibility lies, as concessions will have to be made by both sides. The team may have to meet with several groups to introduce the project. Each group should be met with independently so that the team can get an idea of the issues. Some of the groups will have similar concerns, and the issues of one group may conflict with those of another. The team should identify which people in a group have the influence and arrange to meet with them.

Prior to formal community meetings, an initial introduction of the project should be made to a small core group of people who represent the overall organization. The best meeting place for this assembly would be at one of the houses of the core group. The objective is to keep the group at ease and comfortable. Emotions can run high over land development issues, and the informal surrounding may tone them down. The core group is the perfect forum in which to discuss the merits of the project as well as possible negative impacts. However, suggestions should be made as to how the negative impacts will be mitigated. Provide the necessary information to this core group and let them take it to the larger group for discussion. The development team should arrange to meet with the larger group after all the group's members are familiar with the project.

After initial meetings, the development team should evaluate comments. Analysis will include the practical and financial costs of modifying the plan proposal. All parties will not be satisfied with the final plan. However, if the majority of the people can be satisfied and the development team has made a legitimate attempt to accommodate outstanding issues, the project stands a reasonable chance of approval.

Perhaps the most frequent issue for the team to address is neighboring residents' concern regarding the compatibility of the proposed project with existing residences in the community. This is particularly true with residential proposals when single-family attached units are proposed within or adjacent to a neighborhood of predominantly single-family detached housing. The objection is expressed most often as a fear of devalued property, with concern that a product of lesser quality is intruding. Underlying this attitude is the belief that the income of the new residents will be substantially less than their own. Existing residents are often shocked to learn the new home prices, probable mort-

gage payment, and income requirements for new housing. In addition, townhouses need not conjure up images of inner city rowhouses. Differing architectural treatments can be employed to enhance the new project's compatibility, even to the point of making the new structures appear to be large single-family residences. Buffering, landscaping, tree preservation, and retention of open space areas along the project's boundaries may also eliminate fear of the new project and win community support.

Concerns about traffic congestion are also high on the list of neighborhood objections to new projects. Sometimes those objections are withdrawn when commitments are made to improve the road network. This can be accomplished by providing street widening, turn lanes, traffic control devices, or techniques to prevent cut-through traffic from neighboring arterial roads. As with all design modifications, the cost of the proposed improvement must be balanced against expected return on investment. However, if incorporating the changes means that the plan is more likely to win the support of the community and can be made more marketable at the same time, rezoning approval is more likely to be granted.

A change to a different type of land use, for example, from residential to a commercial district, presents a different scenario. The neighbors that the development team must communicate with will likely come from a greater distance. Community associations often band together when they fear that spreading commercialization threatens their community stability. In this case, as with a more intense residential proposal in a residential area, there are a significant number of opportunities to address important concerns by modifying a proposal to address community concerns.

Despite the majority of the groups having been successfully negotiated with, there may be other individual citizens who remain strongly opposed to the project. The team should anticipate those individuals who will publicly oppose the project and should be prepared to identify and address the specific issues revised during public hearings. It is important to demonstrate that outstanding objections have been noted and that the team has attempted to make all reasonable efforts to resolve them. It is advisable to document important plan revisions and compromises that have been made throughout the early stages of the project.

### Presentation Information and Graphics

Facts, figures, and graphics should be prepared to support the statements made at all meetings. In view of the various levels of sophistication and understanding of land development and local process of the audience, graphics should be simple and easily readable. The team should be flexible, and presentation graphics should imply that flexibility. Expensive rendered site plans and building elevations present nicely; however, as the sophistication of residents and civic groups grows, so does their skepticism about the information shown on these drawings. The graphics must be prepared to reflect honesty, and they must be drawn in such

a manner as to imply accommodation to change. If the drawings look too finished, too final, too slick, it leaves the impression that the team has little flexibility for revision in the final plan design.

In addition to the materials identified earlier for pre-application meetings with the public staff, following are recommendations for the presentation graphics use at community meetings:

- As shown in Figure 11.1, development plans should be rendered (colored) to aid in visualizing the layout of roads, lots, buildings, and open space. Plans should be mounted on boards to facilitate their display. If meetings are large, multiple copies can be prepared for display around the room. Slide and overhead projection of the proposed development plan is also an effective means of presentation.
- To the extent possible, include adjoining homes and properties on the graphics. If adjacent property owners are expected to be present, be in a position to locate their property and refer to the owners by name during the presentation.
- If new landscaping is proposed, a typical plan should be shown. In plan view, use reasonable representations of tree canopies at a given maturity, whether at time of planting or at 10 years of age, and only show those trees that are intended to be planted. Communities and neighbors have been misled by an overzealous use of tree stamps, only to find that the proposed plantings are much less than represented on the graphic. In some communities, the developer may be bound by what is shown on site plans, even if it exceeds ordinance requirements.
- If clearing of existing wooded area is proposed, show the proposed clearing line accurately. This is particularly important if the clearing line is near the property boundary and the existing trees serve as a buffer to the adjoining property.
- Show building materials and architectural style as accurately as possible; however, a qualifier should be included on the elevation to allow flexibility in final materials and architecture. This is especially true if renderings will be used during future public hearings where graphic representations are perceived by the audience to be absolute and final. If renderings are not intended to be an accurate representation of final design, that fact should be made clear and repeated often.
- To demonstrate that what is on paper is not set in stone, use overlays, inserts, pasteovers, and alternate drawings to demonstrate other design schemes, changing designs, and changeable designs.
- Scale models may be useful to represent the relationship of site massing and other features. Due to the



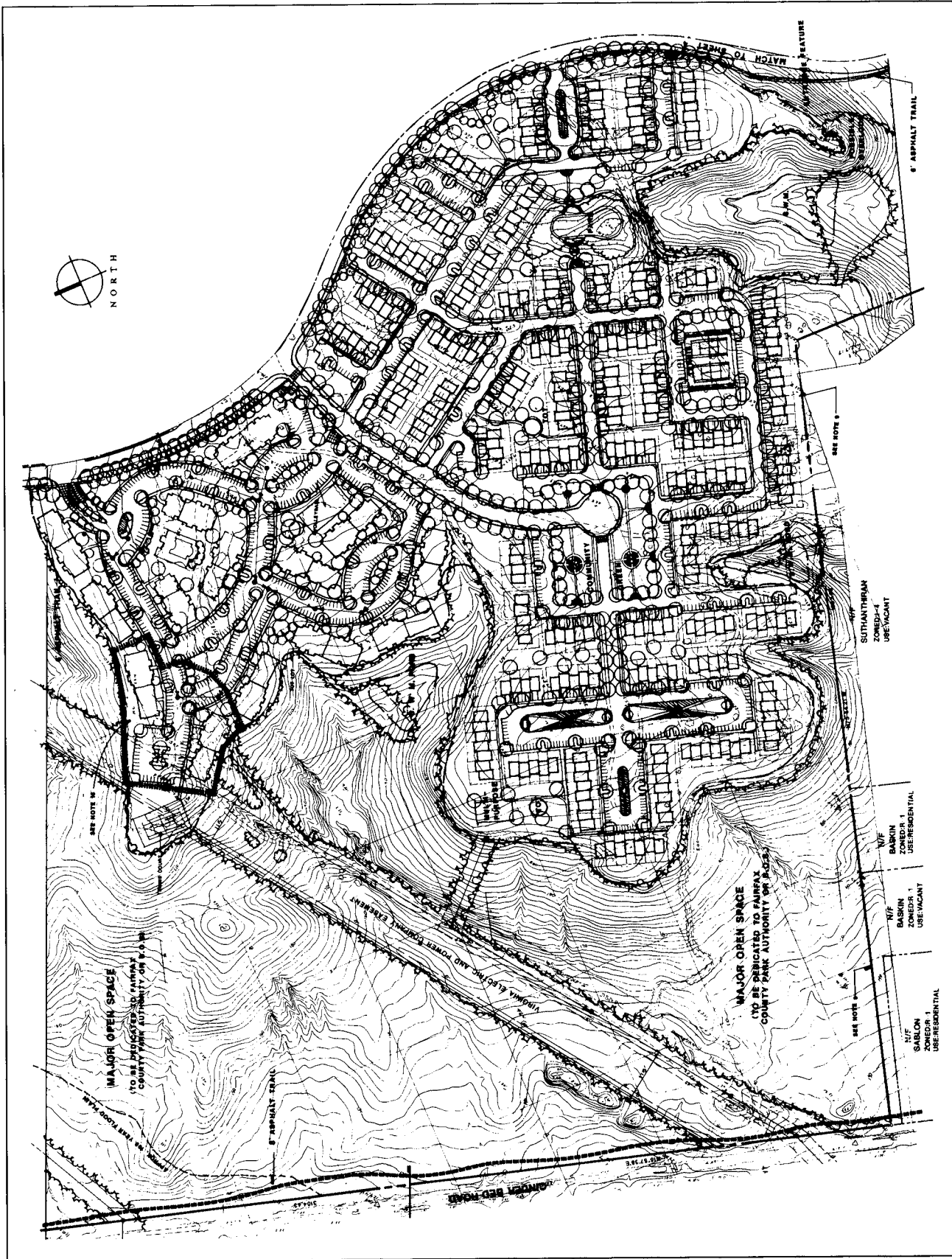


FIGURE 11.1 Typical graphics used in community group presentations.

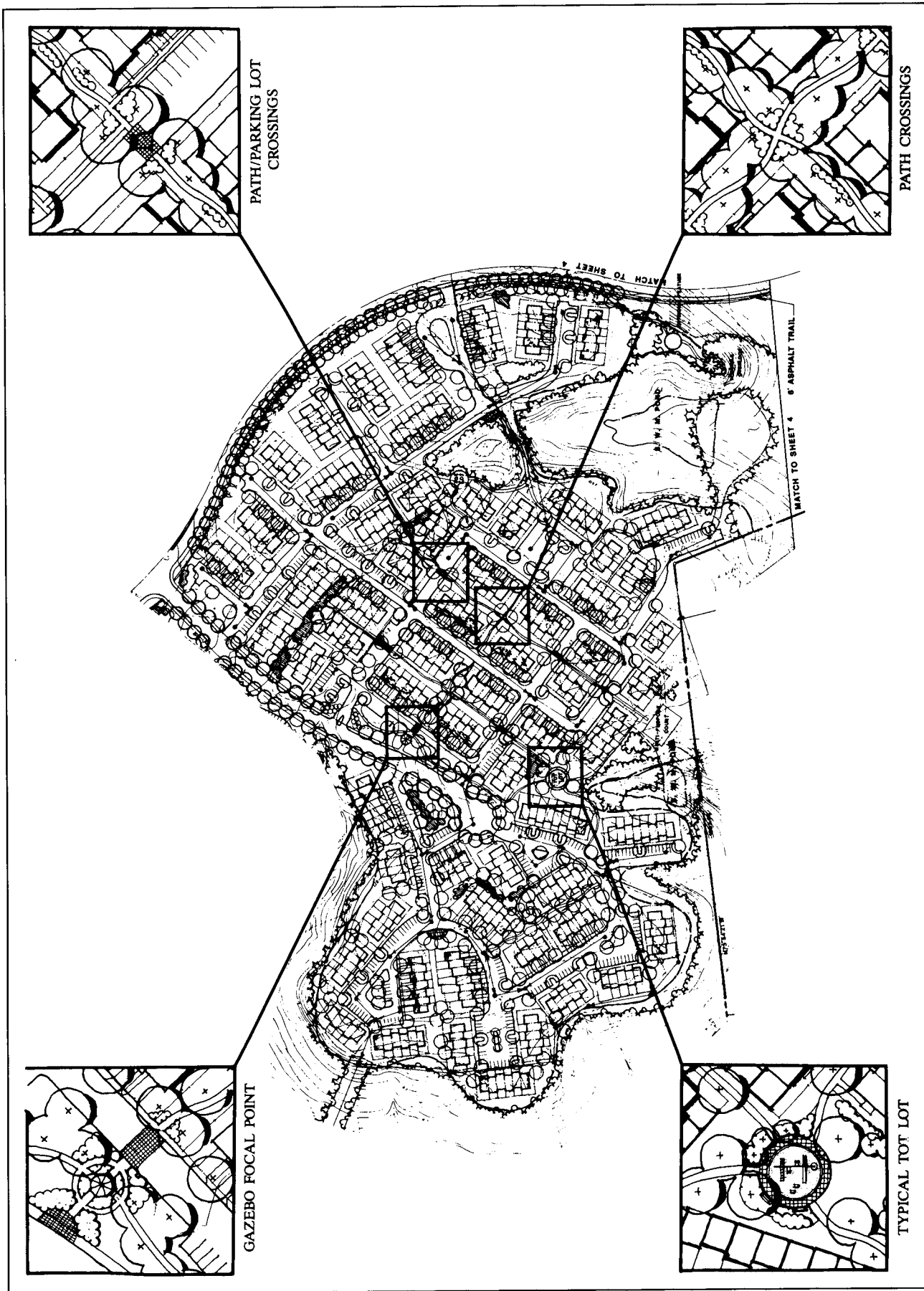


FIGURE 11.1 (Continued) Typical graphics used in community group presentations.

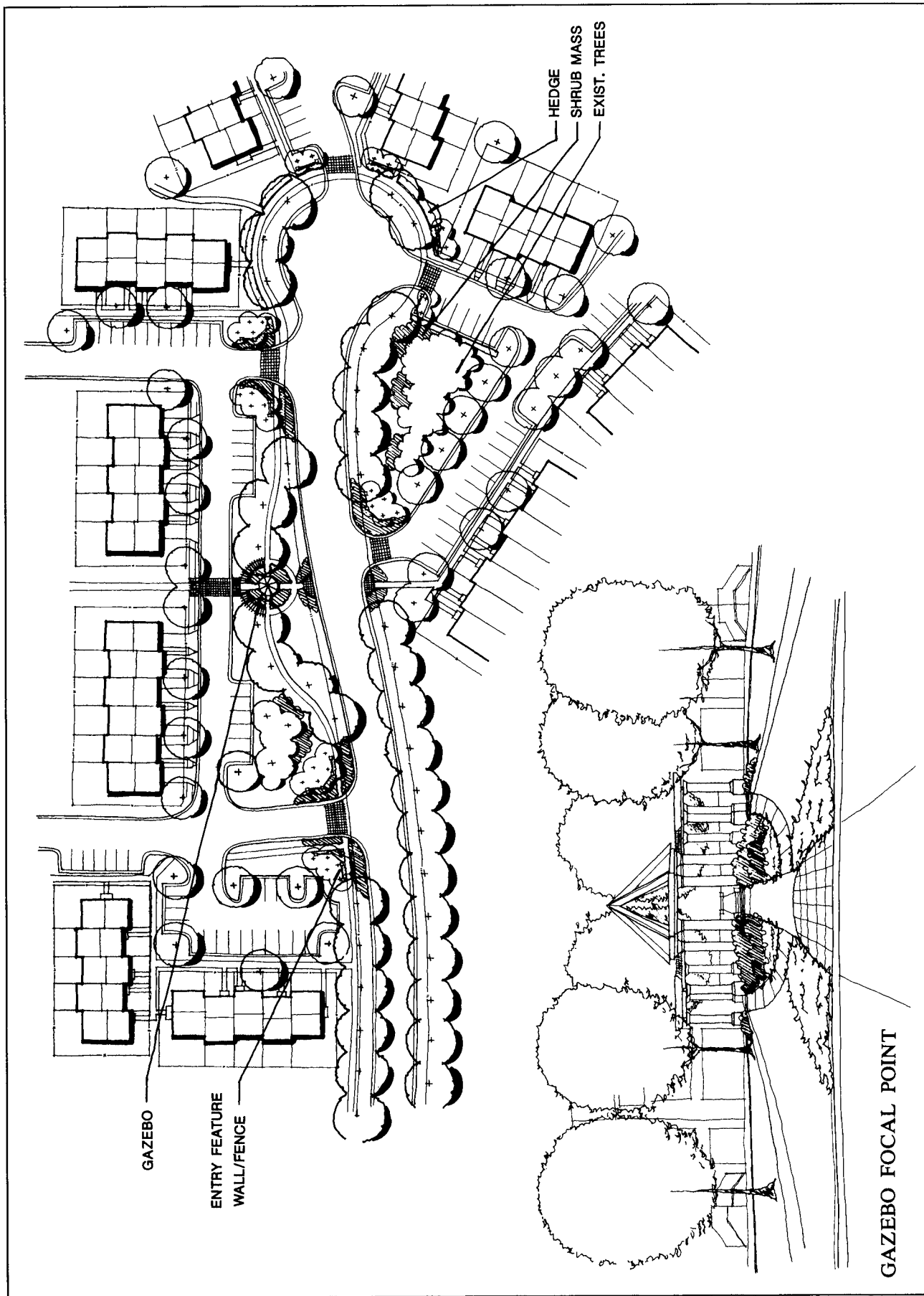


FIGURE 11.1 (Continued) Typical graphics used in community group presentations.

high cost of models, their value should be evaluated against other project variables. The precautions used in preparing plans and renderings should be considered in model production.

- In certain situations, it may be necessary to organize a site visit, using balloons to depict proposed building heights and skylines and survey markers and ribbons to demonstrate building footprints and clearing limits.
- More complex or sophisticated projects may benefit from the use of computer graphics to show how new structures will fit in with the existing environment.

### Addressing the Issues in the Formal Rezoning Process

In addition to convincing the general public to consent to or support the project, the development team confronts a formidable hurdle in generating support and approval of the public agencies and the legislative body. These agencies are concerned with the health, safety, and welfare of the community as a whole, and the benefits of the project to the immediate locality must be weighed against the benefits or impacts to the larger community. As discussed in Chapter 34, the restrictions or limitations imposed by approving agencies are complex. Procedures are established and requirements imposed in an effort to protect the health, safety, and welfare of the community and are intended to be uniformly applied. The analysis and decisions made in conjunction with the rezoning process are designed to ensure not only that the project addresses the projected and desired land use patterns but that it protects the health, safety, and welfare of the entire community.

The procedures followed in many communities vary, depending on state enabling legislation, local ordinance, and custom and practice. In general, applications are prepared in accordance with specific ordinance standards that specify the narrative and graphic information that must be included and the format for presenting the proposal. Such information usually includes extensive analysis and studies addressing possible impacts of the new development on the community and the demand for additional public services that it will generate. Standard application forms and information checklists are usually available from the municipality's planning or zoning office. Public notice is required by way of certified mailings and posting of the property. The notices generally contain information regarding the proposed use, scheduled public hearings, and the responsible staff person of the coordinating agency. The municipality usually advertises public hearings in local newspapers and public cable TV, if available, as well as in community association bulletins and newsletters. The applicant must follow legislated administrative procedure to the letter, as adversary groups can delay or terminate projects where proper notice has not been given. State legislation generally mandates that action of the governing body must be com-

pleted within a certain specified time period, including specific findings relating to the application.

The professional planning staff or planning consultant for the jurisdiction reviews the application for compliance with recommendations of the comprehensive plan and related ordinance requirements and established public policy. The staff will distribute the application to other agencies and request that they review the proposal relative to their own area of expertise. Upon completion of its reviews, the staff will often schedule meetings with the applicant to relay its findings. Such a meeting will focus on the changes needed to be incorporated by the development team in order to secure an endorsement from the staff. The development team may take advantage of this opportunity to negotiate with staff and, perhaps, reconsider its proposal and submit revisions to the original plan. The staff will then make a nonbinding recommendation to review authorities.

The rezoning application will then be reviewed by an advisory panel, such as a planning commission or zoning hearing examiner, usually in an open forum. In some jurisdictions, these reviews are sequential, that is, the planning commission will review the application and forward its recommendations to the hearing officer. Depending on the jurisdiction, public hearings may be scheduled before either or both of these reviewing bodies. The public hearing provides both the public and the development team an opportunity to present evidence that supports their position. In most cases, the recommendations made by commission or hearing examiner are advisory to the governing body; nonetheless, they carry a lot of weight because most often the recommendations are a result of citizen input.

Following review of the advisory panel, the team has an opportunity to rethink the proposed development plan in light of the comments received from the panel. In some instances, the team may wish to defer the public hearing with the governing body to allow time to revise their proposal in hopes of addressing issues raised by the advisory panel. Such revisions may or may not be required to be reviewed again by the advisory body. However, the planning and professional staff is usually involved in review of the amended application to determine the appropriateness of the modifications and the success with which the applicant has resolved outstanding issues.

The jurisdiction's governing body considers the application, along with recommendations from the professional staff and from its planning commission or hearing officer, prior to rendering a decision on the rezoning. In most jurisdictions, the governing body conducts a public hearing as part of its consideration. In some jurisdictions, however, the governing body may only consider the evidence and public record presented in the application at previous hearings, with direct or ex parte communication with legislators concerning the application prohibited. A simple majority of the members of the governing body, that is, one more than half the number of voting members, is usually required to

decide upon a rezoning application. Some communities require a supermajority to adopt a rezoning for which the staff or planning commission have recommended denial.

The governing body often suggests or imposes conditions or modifications that the developer must incorporate into its proposal before it takes formal action. Further, it may direct the applicant to establish communication with neighborhood groups to resolve outstanding differences, deferring its decision until the proposal is modified. Zoning ordinances usually establish procedures for anyone aggrieved by the decision of the governing body to appeal that decision in the court system. A waiting period is often established after an approval is granted, in order to allow appeals to be filed.

### Review of the Application Submittal

The land development team should be prepared to address comments from a broad variety of local and state agencies. The following agencies or functions typically are involved in the review of rezoning applications (see Figure 11.2):

- Long-range comprehensive and current planning zoning and subdivision functions are often, but not always, included in a single department. Personnel responsible for long-range planning usually determine if the proposed use and development conforms to the recommendations of the comprehensive plan. They also identify lands proposed for development that may be designated in the plan for public use, such as for highways, schools, parks, libraries, or fire or police stations. The staff will determine the extent to which the proposal addresses specific concerns raised in the plan and whether the manner in which these issues were addressed is consistent with established plans and policies.
- Personnel in current planning will determine compliance with zoning ordinance requirements, such as lot size, setback, and density requirements. The analysis will also focus on other uses that may be allowed by the zoning district being requested, which uses may or may not be desirable. With the continuing emphasis on the protection and preservation of the environment, most municipalities have established a department or group that reviews all development proposals and rezoning applications in terms of potential impacts on the environment and compliance with federal regulations.
- The public works or transportation department typically addresses proposed improvements to the transportation network, the adequacy of streets, and public utility systems. In some jurisdictions, a state-level transportation agency will be involved. Similarly, water and sewer systems may be run by a regional authority serving several jurisdictions.
- Fire-protection officials are frequently involved in the review to monitor development activity to assist in locating future station facilities.
- School districts typically perform an analysis of potential school enrollment generated by new development. This review is often geared towards determining adequacy of the existing system and identifying sites needed for future facilities.
- A local parks and recreation department will review applications to evaluate recreational needs generated by the new development as well as the impact on existing facilities. This department may also be responsible for preservation and acquisition of important environmentally sensitive areas, such as stream valley, floodplains, and wetlands, the control of which may be included in the comprehensive plan.
- The local utility companies will be involved in confirming system adequacy.

As noted in the previous chapter, some jurisdiction's ordinances and enabling authority do not allow or require developers to be bound by a development plan used to justify the rezoning. Where the proposed zoning district allows a wide range of uses or intensities, there may be reluctance to approve the rezoning due to concern that a project more intense than that being proposed could be built in its place, either now or in the future. In such instances, it may be possible to seek rezoning to a less intense zoning district that allows uses similar to that which is being proposed but does not permit those uses viewed as too intense. In some cases, a district that is lower in the hierarchy may allow the proposed use as a special exception, or may not allow a use to which residents object. The elimination of uncertainty may serve to moderate opposition to the plan.

### The Public Hearing and Role of the Development Team

The public hearing scheduled by the planning commission, hearing examiner, and governing body is often the only formal opportunity for the development team to address the entire body directly to discuss its intentions and the merits of the application. In addition, this forum provides citizens with an opportunity to express their opinions about the project.

In many jurisdictions, the meeting sessions during which development application-related public hearings are held are lengthy. In rapidly developing communities, the meeting agendas are often crowded with hearings for development applications that include projects of all size and varying degrees of impact. The hearings are often scheduled in the evening or on weekends to accommodate commission members, who may serve on a voluntary, part-time basis, as well as to increase opportunities for citizen participation. For the governing body, the public hearings may be part of

COMMONWEALTH OF VIRGINIA  
 COUNTY OF FAIRFAX  
 APPLICATION FOR ZONING MAP AMENDMENT

APPLICATION NO. \_\_\_\_\_  
 (Assigned by Staff)

PETITION

TO: THE BOARD OF SUPERVISORS OF FAIRFAX COUNTY, VIRGINIA

I (We), \_\_\_\_\_, the applicant(s),  
 petition you to adopt an ordinance amending the Zoning Map of Fairfax County, Virginia, by reclassifying from the  
 \_\_\_\_\_ District to the \_\_\_\_\_ District the property described  
 below and outlined in red on the Zoning Section Sheet(s) accompanying and made a part of this application.

PROPERTY DESCRIPTION

1. LEGAL DESCRIPTION:

Lot(s)	Block(s)	Subdivision	Deed Book	Page No.
--------	----------	-------------	-----------	----------

2. TAX MAP DESCRIPTION:

Map No.	Double Circle No.	Single Circle No.	Parcel(s)/Lot(s) No.	Total Area(Ac. or Sq. Ft.)
---------	-------------------	-------------------	----------------------	----------------------------

3. POSTAL ADDRESS OF PROPERTY: (If any)

4. ADVERTISING DESCRIPTION: (Ex. South of Rt. 236, 1000 feet west of Rt. 274)

5. PRESENT USE: \_\_\_\_\_

6. PROPOSED USE: \_\_\_\_\_

7. SUPERVISOR DISTRICT: \_\_\_\_\_

The name(s) and address(s) of owner(s) of record shall be provided on the affidavit form attached and made part of this application.

The undersigned has the power to authorize and does hereby authorize Fairfax County staff representatives on official business to enter on the subject property as necessary to process the application.

\_\_\_\_\_  
 Type or Print Name of Applicant or Agent

\_\_\_\_\_  
 Signature of Applicant or Agent

\_\_\_\_\_  
 Address

\_\_\_\_\_  
 Telephone No.

\_\_\_\_\_  
 Home

\_\_\_\_\_  
 Work

Please provide name and telephone number of contact person if different from above.

DO NOT WRITE IN THIS SPACE

Date application received: \_\_\_\_\_

Application Fee Paid: \$ \_\_\_\_\_

Date application accepted: \_\_\_\_\_

Form RZ (10/89)

**FIGURE 11.2** In addition to this application, Fairfax County has a four-page instruction memo listing 16 other requirements and fees that must also be submitted.

an even longer agenda reflecting their role in managing the municipality. Hearing rooms are often uncomfortable, filled with observers, applicants and their attorneys and consultants, citizens, and the news media. The proceedings may be televised for the benefit of those unable to attend. The public hearings frequently run for hours, often into the early morning, as participants in each case present testimony reflecting their position on the application. Controversial cases may have long lists of preregistered speakers, witnesses, and experts. At times, testimony in one case may motivate observers to participate in another case while waiting.

Members of the commission or governing body usually are given a packet of materials prior to their meeting reflecting the day's agenda. These information packets are often lengthy, with endless photocopies of development applications, maps and plans, engineering analyses, traffic studies and other rationale, staff analyses and recommendations, as well as letters of support and opposition. Members often engage in interminable debate, sometimes insightful, sometimes philosophical, sometimes taking a role of devil's advocate and oftentimes politically motivated. The questions they may ask of their staff, the applicant, or others who testify may be probing may be critical, supportive, and sometimes even inane. Often the person asking the question knows the answer and merely wants it spoken for the record.

Given the incredible amount of information provided to the members of the governing body and the numerous decisions required to be made in a single session, the key task of the development team is to get the members of the governing body to focus on the development plan they have brought to the table. The governing body may know the exact location of the project or have only general familiarity with the area and consequently place heavy reliance on the analysis and recommendations provided by the professional staff. Due to the lengthy agendas, relatively little time is allotted to each speaker and little or no time afforded to the development team to rebut or refute the reports or testimony of citizens or the staff. In the most controversial of cases, the citizens groups may even hire their own expert witnesses. Community residents themselves may be attorneys or traffic engineers or members of any number of professions whose testimony is given as much credence as that of the development team. The skill with which the development team approaches the hearing will have a strong bearing on the outcome.

In view of the limited time given for formal presentation of the development proposal, perhaps only 10 or 15 minutes, the team member making the presentation must be well organized. He or she must establish and adhere to a well-defined agenda that focuses on the main points the team needs to make:

The speaker should begin with a personal introduction and describe his role on the development team and introduce other members of the team who might be present. If the developer is not generally known to the board members

or the public, credibility and reputation should be established. This can be accomplished by referring to other local projects with which the developer may have had a role, or using renderings or slides of successful projects elsewhere. The presentation should include a comprehensive, yet brief, outline of the proposal. In particular, emphasis should be placed on how the recommendations of the comprehensive plan led the development team to pursue the proposal. A brief assessment of how the proposed project creates and/or satisfies economic or marketplace demand should be presented. Benefits to the community in terms of tax revenue, job creation, public improvements, and aesthetics should be addressed.

It is important to note the critical issues that were identified during the feasibility/technical analysis phase and how they were addressed and mitigated. The presentation should also focus on the relationship of the proposed development to the zoning ordinance and the adapted comprehensive plan. In addition, it is helpful to discuss issues raised by the community and how the plan was revised to resolve them. Citizen support usually makes the decision-making by the governing body easier.

If a development plan is included as part of the application, a rendered version of the plan should be available at the hearing for discussion. Depending on the requirements of local ordinance, the plan should closely represent what is proposed to be constructed. It should be understood that items shown in excess of minimum ordinance requirements may become binding upon the developer. Even if not binding, their omission in actual construction may prove damaging to the developer's reputation.

The team must prepare to address any and all types of questions that might be asked of it. For important, complex, controversial projects with high visibility, key development specialists who have been involved in analysis and design of the proposal should be present and prepared to address a myriad of technical questions. The presentation should anticipate questions and criticisms that might be lodged against the proposal by board members, citizens, or staff. Throughout the course of the hearing, members of the board or commission may call upon the development team to respond to questions or issues raised by other's testimony. In answering, the respondent should use the opportunity to stress related points that may have been omitted or abbreviated in the formal presentation. Often, at the close of the hearing the board will refer back to the professional staff for concluding arguments or analysis of the presentation. Consequently, it is important to have worked closely with the staff during its analysis as they often have the last word.

The legislative or governing body normally renders its final decision at the close of the hearing. Its decision will be based on the information presented by the applicant, advice from the staff, commissions, and testimony from the citizens. If it is believed that continued negotiations between the developer and the opposing groups can achieve greater accord, the public hearing or decision may be con-

tinued or deferred to a later date. The outcome of the vote is contingent upon the response of the citizens and how sympathetic the board or zoning administrator is towards the project. It is imperative that the development team proceed through the various stages by the book, such as proper notices sent out at the right time, all applications at the state level and local level filled out and properly submitted, proper notification given. The negotiation and hearing process may take several months and possibly several years, depending on the complexity of the project and the number of citizen groups involved. The developer should anticipate this time frame and establish the purchase contract accordingly.

#### REFERENCES

Institute for Participatory Management and Planning. 1984. *The Consent Builder's Bulletin*, vol. 1, no. 4, Winter.

Institute for Participatory Management and Planning. 1986. *Citizen Participation Handbook*, Laramie, WY.

International City Management Association, *The Practice of Local Government Planning*, International City Management Association. Washington, D.C., 1979.

National Association of Home Builders (NAHB), *Subdivision Regulation Handbook*, NAHB, Washington, D.C., 1978.

Urban Land Institute (U.L.I.), *Working with the Community: A Developer's Guide*, U.L.I., Washington, D.C., 1985.

#### ADDITIONAL READING

David Jensen Associates, Inc., *How to Win at the Zoning Table*, National Association of Home Builders, Washington, D.C.

Engineers and Surveyors Institute, *Improving Preparation, Review & Approval of Subdivision and Site Plans in Fairfax County*, Fairfax, VA., 1988.





## PART III

# CONCEPTUAL DESIGN

*Dennis Couture, A.S.L.A., R.L.A.*

### Introduction

Conceptual Design represents the initial effort of describing alternative plans that satisfy the development program objectives in light of the site characteristics identified in the site analysis. It requires full recognition of the client program components, initial site assessment, site context, and planning and regulatory controls.

Sketches, functional diagrams, or concept plans are normally completed to illustrate a framework for the given development program. They generally represent diagrams of the potential distribution of land uses and major circulation requirements. The effort seeks to meld all pertinent comprehensive or master planning information with site-specific considerations to illustrate how the site might best be developed. For large-scale development, this initial cut at the design effort is often accomplished in what are termed “blob” or “bubble” diagrams. The intent is focused less on sophisticated graphic technique and more on fostering dialogue, a preliminary review and assessment, and confirmation of design direction among the design team participants. Concept plans reflecting land use distribution and associated circulation are reviewed in the context of their preliminary implications on infrastructure requirements as well as economic, functional and political feasibility. This phase, similar to the more subjective assessment included in the initial feasibility and site analysis steps, moves from the review of general intent to actual physical diagramming of use arrangements. This level of conceptualization is generally completed prior to the investment of resources and time resolving more detailed levels of design.

### STEP 3: CONCEPTUAL DESIGN

There are generally multiple solutions for the design of any one site. It is common for alternative concept plans to be

developed as part of this initial conceptual design phase. The ensuing discussion with the development team may prompt a dialogue as to the pros and cons of the various solutions and identify problems the designer alone may not have anticipated. The end products associated with the concept design phase may be a series of sketches or diagrams highlighting the distribution of land uses and preliminary infrastructure requirements. The diagrams themselves may be annotated to underscore the advantages or disadvantages associated with the depicted solution and memorialized for future reference.

As demonstrated in Figure III.1, conceptual level sketches generally include preliminary delineation of such considerations as:

- Points of site access together with an initial alignment of major vehicular circulation routes. The principal road network should be developed in the context of any public comprehensive or master planning policies or recommendations.
- A distribution of major land use elements by type. The delineation should reflect the approximate area requirement for each use to begin to gauge the massing or relative area requirements necessary to accommodate individual program components.
- Delineation of areas mandated or desirable as open space based the previously completed site analysis, such as floodplain, protection areas, wetlands, mature woodland, and streams and stream valleys.
- Preliminary determination of the need and location of major public facilities such as schools, parks, fire

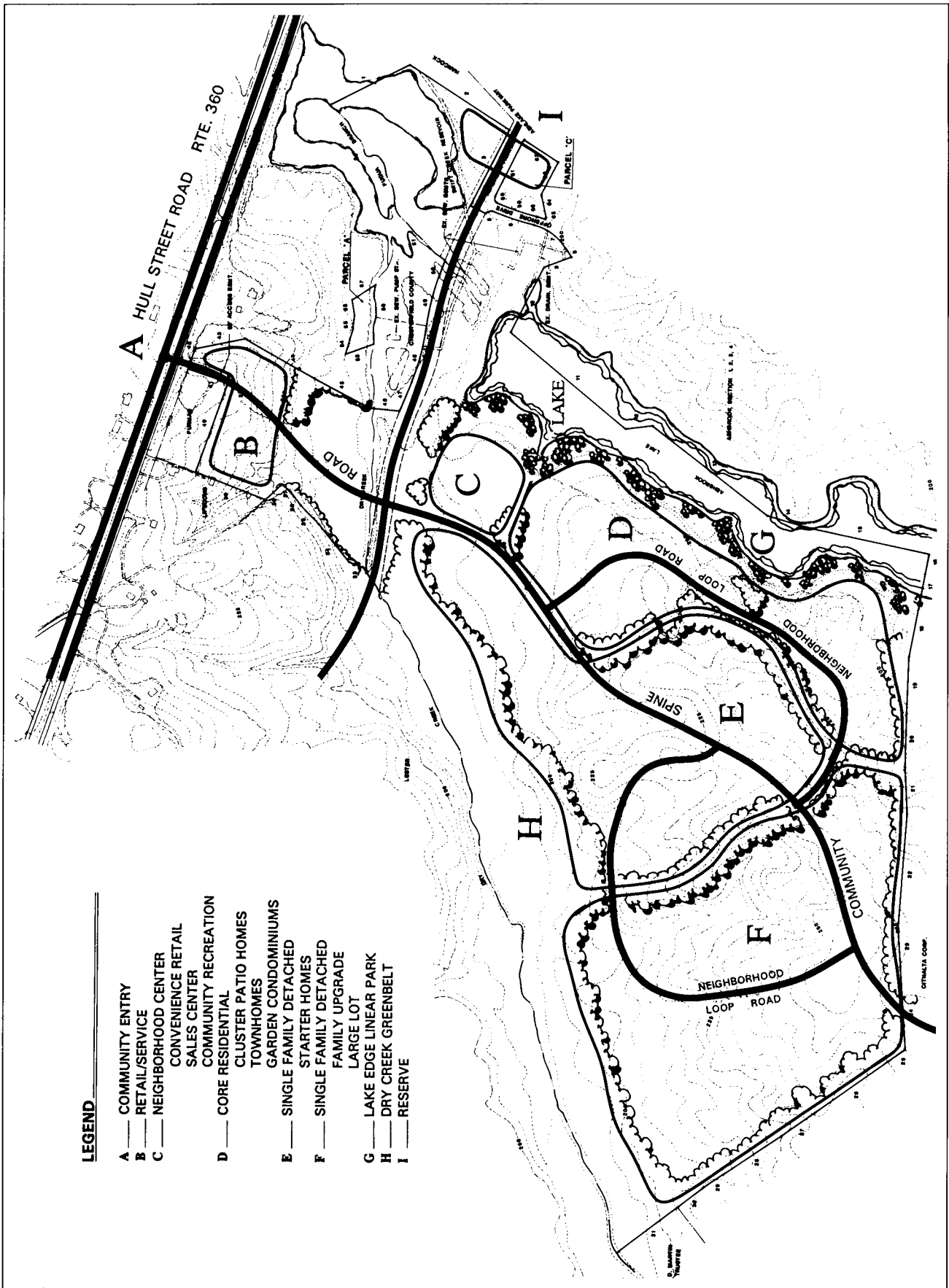


FIGURE III.1 Conceptual design.

stations, and libraries, as identified in public planning documents

While normally not a formal public submission document, conceptual sketch plans do provide information suit-

able for early discussions with appropriate public officials. They provide sufficient information to obtain an informal assessment by appropriate public agents/officials as to the development plan's compliance with public comprehensive and land use policies and the local political climate.



## CHAPTER 12

# DEVELOPMENT PATTERNS AND PRINCIPLES

*Dennis Couture, A.S.L.A., R.L.A.*

### INTRODUCTION

This chapter presents an overview of some of the more prevalent land use types and development patterns that comprise our contemporary built environment. It offers a general discussion regarding development intensities and design patterns, including some of the more widespread influences on development activity. Such influences range from aesthetic and prevalent community attitudes regarding land development project design to the more formative factors associated with environmental, political, and economic considerations that guide site design and engineering activity.

Perhaps the most overriding consideration for any land development endeavor should be the project at hand in the context of the larger community. Community in this sense is more than the physical characteristics of a given project's surrounds. It embraces the extant sociocultural, political, and economic paradigms that model the dynamics in any given jurisdiction, resulting in the land development types and patterns that are built. As local context offers a guide to project design opportunities, so do more universal constructs that have evolved in response to the real and perceived shortfalls of contemporary development trends. Sensitivity to the urban-suburban-rural distinction in development types is simplistic. Development considerations that are mindful of the community-making and place-making opportunities and design principles associated with traditional design, such as traditional neighborhood design (TND), neotraditional, and new urbanism, afford a renewed perspective on both the dynamics of community and the opportunities for meaningful site and project design. Similarly, constructs associated with the advocacy of smart

growth and green infrastructure offer meaningful insight into responsible land development activity. These latter considerations are appropriate across the spectrum of urban-rural development activities.

An extensive overview of the basic tenets associated with these more recent ideological approaches to land development is beyond the purview of this chapter. Contemporary literature and professional journals abound with information on both theory and practical application. Familiarity with this information is particularly relevant for land design agents given that these approaches are perhaps more intertwined in the livability and marketing side of land development and site design than might be evident from in-place land use regulatory controls, statutes, and zoning codes.

This chapter provides an overview of prevalent contemporary land use types. Residential and select nonresidential building considerations are presented in isolation of each other. Although this format may be considered anathema to proponents urging greater integration of potentially compatible or supportive land uses, extensive discussion of such endeavors, even in the context of current mixed use and planned developments, is not the objective here. Rather, identifying some of the base considerations associated with select land use types may provide a cursory foundation for better appreciating and identifying what may be unique to each use and how they may be best integrated.

A variety of factors determine the type and configuration of a given land use, including the expectations of landowners, users, and consumers; the availability of land; the cost of construction; the natural attributes of the site; and the character of the surrounding community. These are modified to a great degree by local regulations, which are based largely on more encompassing issues of health, safety and

welfare, local considerations relative to the nature of local commerce, prevalent community attitudes, and past community practice.

An examination of the zoning ordinance in all but the smallest communities reveals great variety in the categories of permitted land uses. Typically, this includes residential, commercial (sometimes broken down into retail and office uses), and industrial uses. Depending on the size, location, and maturity of the community, a jurisdiction's zoning document may establish other land use categories to reflect its unique economic base, such as manufacturing, agriculture, maritime, mining, forestry, and institutional uses, to cite a few. Furthermore, the zoning ordinance, as discussed elsewhere in this book, normally identifies a series of development standards, commonly expressed as minimum requirements focusing on such elements as lot size, building height, yard requirements, open space, and impervious cover.

That architectural styles vary in different geographic regions in the country is no surprise. Regional variation in history, culture, and heritage affects development patterns in density and configuration as well. Different names are often used to describe similar types of land use.

Most local regulations are extremely specific concerning permitted density, height, bulk, and setback requirements for most land uses. To attempt to assemble and reproduce this wide range of specifications would be neither productive nor useful. The discussion of land uses, development intensities, and the design principles in this chapter is intended as a primer.

If there is a common denominator throughout contemporary development, it may be its heavy reliance on the automobile as the principal means of transportation. The suburban development pattern is characterized by a strong dependence on the car for both work- and leisure-based trips. Historically, the separate clustering of residential and employment areas at density levels low enough to make public transportation infeasible have reinforced this. The single-occupant automobile continues to be the most prevalent form of transportation in the suburbs. This phenomenon places a unique set of requirements on resulting land use types and development patterns. Attention to a common need for both vehicular circulation and on-site parking is necessary. Streets and roads provide the common linkage between and within land uses and the principal framework of the resulting development patterns. Similarly, parking requirements for the automobile are a major design element in the sizing and organization of individual land uses.

## DEVELOPMENT TYPES

Ideally, a community seeks to guide its growth in a manner consistent with its heritage, common goals, and economic interests. Local land use activity is guided by comprehensive or master planning, representing a compendium of policies, usually subjected to arduous public participation, focusing on topics such as population and economic

growth, housing, transportation, community facilities, and land use. While a comprehensive plan may contain a mapped graphic representation of what the community might look like at some point in the future, the local zoning ordinance and zoning map are the tools that provide the day-to-day implementation effecting land use decisions relative to the overall comprehensive plan policy objectives. A clear understanding of the distinction between these documents is imperative to understanding the extant land use patterns associated with development activity in any given community. Decision-makers in land development and design must be sensitive to the rationale for the type and use intensities identified in the comprehensive plan. They must also be fully apprised of the design standards and criteria governing the size, form, and character of proposed development, as identified in the local zoning ordinance and similar regulations.

It may be useful for the novice to consider a local community as a single large planned community. The comprehensive plan and its associated policy statements represent the collective public vision to guide community growth. Various ordinances and regulations provide the means to implement that vision. Together they provide the two important elements that establish the foundation for land development patterns in a community.

Historically, zoning ordinances have relied on the delineation of distinct geographic zones or districts that accommodated the development of one land use type. These districts generally included residential, commercial (normally inclusive of office and retail uses), and industrial categories. Subcategories of each use may stipulate gradations in density within a land use, prompting alternative development patterns or building programs. While much land activity is rooted in a single zone/single use development pattern, more flexible development patterns and land use combinations are fostered by development controls that perpetuate planned, planned unit, and mixed use developments. The objective of these latter categories move beyond singular use and attempt to foster a framework for greater variety in permitted land uses and arrangements. Prior to discussion of individual development types, it will be valuable to outline briefly some of the more encompassing planning terms.

## Conventional Subdivision

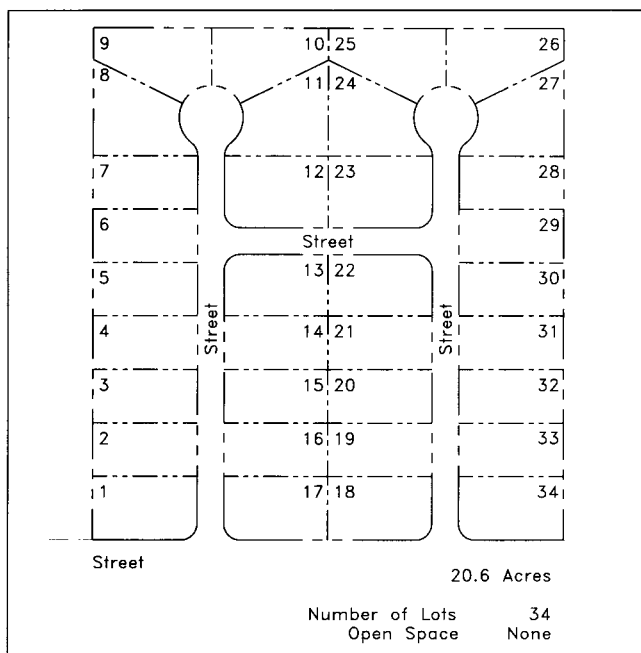
The historic basis for much suburban land use is the conventional subdivision, created by the division of larger land tracts into smaller land units. Such subdivision generally requires new streets for accessing the newly created smaller land units or lots. The subdivided acreage provides individual lots for houses and lots or parcels for nonresidential uses. While subdivisions have become varied in pattern, lot size, street alignment, open space network, and in some cases a mix in permitted land uses, conventional development continues to be predicated on the simple subdivision of land. It has been the mainstay of suburban residential

development and accommodation of proximate retail, employment, and public facilities.

The larger tract acreage is divided in a manner that provides for complete transfer of land ownership to subsequent users. Streets are normally incorporated into public ownership, and community open space, if provided, may be deeded to an appropriate public or semipublic entity. Single or multiple builders may on a lot-by-lot basis, initiate construction. Figure 12.1 below represents a typical conventional subdivision.

### Cluster Subdivision

Cluster development relies on subdividing larger properties into smaller lots or parcels, but differs from conventional subdivision in that it typically results in a land plan with a greater percentage of the overall acreage set aside in common, community, or conservation open space. To achieve this open space, local regulations generally allow a reasonable reduction in individual lot size and associated setbacks, provided there is no increase in the overall number of lots that would otherwise be permitted under comparable conventional subdivision. The resultant land area not placed in individual building lots is normally devoted to common open space. This development pattern allows for the concentration of building activity on the more usable areas of a site and ideally results in a reduction in development costs associated with site grading and supporting infrastructure. Cluster subdivisions are also often used to protect environmentally sensitive areas of a given site. The reduction in individual lot size and associated exterior yards is compensated for by the communal open space that is made available.



**FIGURE 12.1** Conventional subdivision.

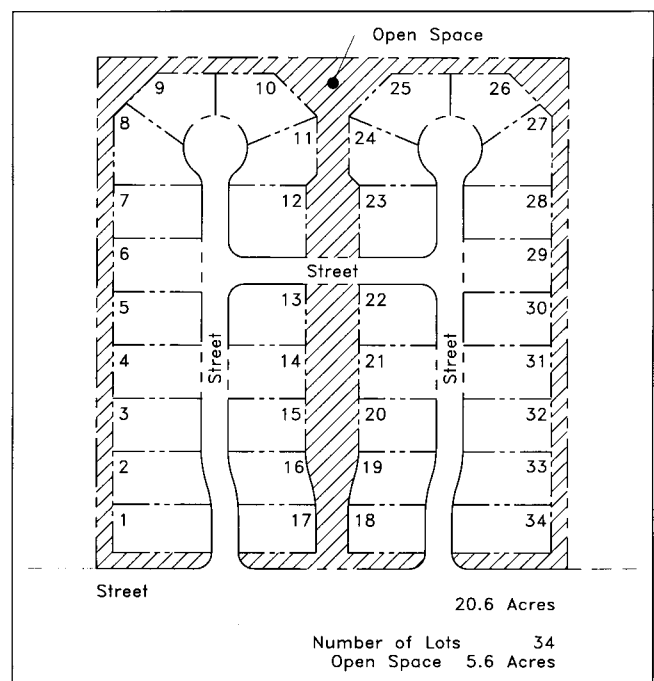
Cluster subdivision requires increased attention to the details of design. Reliance on smaller lots necessitates regard for other methods of maintaining privacy and buffering between residences. Often greater controls are placed on unit design and orientation. Design controls often exceed standards enumerated in zoning and subdivision regulations. It is common for cluster development to be executed as planned developments, as discussed below. A typical cluster subdivision is presented in Figure 12.2.

### Planned Unit Development (PUD)

A planned unit development (PUD), as its name implies, is planned as a single entity and usually represents some minimal-sized contiguous acreage, as specified by local ordinance. It contains single or multiple residential clusters or planned units with provision for public, quasipublic, commercial, or industrial areas. Normally, nonresidential uses are provided in a ratio to residential uses, as stipulated by ordinance. Planned unit developments generally provide for a mix of housing types and land uses and situate development within an overall open space system, with an emphasis on amenity provisions and preservation of natural resources. Figure 12.3 is representative of a planned unit development.

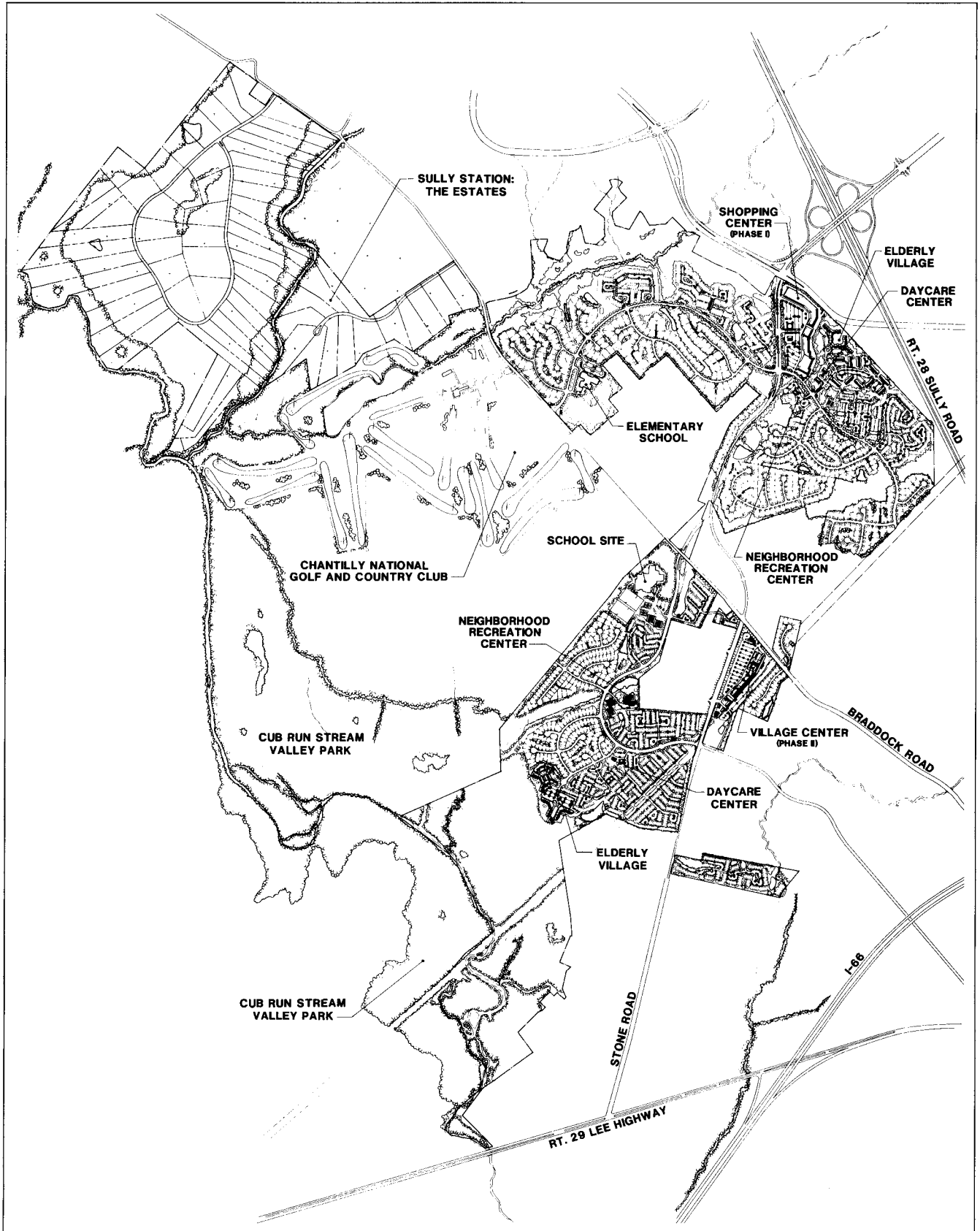
### Planned Community or Master Planned Community

The planned community usually provides for a wider range of residential land uses and includes commercial-retail, employment, recreation, and institutional uses to create a balanced hierarchical community. Planned communities represent large land assemblies that attempt to satisfy a full



**FIGURE 12.2** Cluster subdivision.





**FIGURE 12.3** Planned unit development.

range of lifestyle and support elements to sustain its resident population. Normally, development is undertaken by a single owner or master developer who in addition to orchestrating the financing, planning, and design of a project may construct the required infrastructure and community amenities, making sites available to others for individual building construction. Figure 12.4 represents a master planned community.

## DEVELOPMENT CONSIDERATIONS

A development team must consider the effect that each type of land use has on environmental, aesthetic, financial, operational, and marketing characteristics of a land development project. They should understand the relationship between these characteristics and their effect in offsetting the impacts of development. These may range from physical limitations inherent in the site to a community's predisposition to a proposed program of land development. Following are some characteristics that warrant consideration in their implications on resulting land use patterns.

Generally, there is a direct relationship between the intensity and density of development and its impact on the site and surrounding community. These concerns may focus on the following:

### Environmental Impacts and Opportunities

- *Site disturbance:* More clearing and grading is required as buildings, site improvements, and infrastructure occupy an increasing proportion of land. There is normally a reduction in the opportunity to retain natural grade as density increases. Opportunities to preserve existing wooded or natural areas are similarly reduced as building program coverage is expanded. Lower building density does not in itself result in less impact. It is the combination of existing conditions, proposed program, and resolution of program detail that eventually delineates the degree of impact. An equivalent program in a more compact development envelope can have substantially less site disturbance impact than a comparably sized program dispersed over a greater amount of acreage. Similarly, building programs that are tailored to unique site conditions can have less impact on land than generic solutions. For example, a residence designed for a steep site will have a better fit and require less grading than one originally designed for level terrain.
- *Stormwater management:* As the impervious surface associated with a given development program increases, so does stormwater runoff, along with the increased potential for non-point source pollutants. Conversely, greater opportunities exist for storm drainage control, erosion and sediment control, and best management practices, in that higher density can assist in financially compensating for construction costs asso-

ciated with these mitigation measures. The concentration of density within a site may require less extensive required mitigation activity, or at least contain it.

### Aesthetic Impacts and Opportunities

- *Context:* Compatibility with adjoining land uses can become problematic as proposed development intensity increases. Careful site planning and design can assist in mitigating this problem. Structural solutions may be costly, and buffering solutions may be land consumptive.
- *Architectural design:* More intense development generally requires more stringent architectural controls to ensure overall visual coherence. Modest building elements on large lots or parcels benefit from distance between improvements. Individual units capture their own identity and are less visually and functionally dependent on their neighbor. As building size increases or parcel size decreases, increased proximity warrants greater attention to issues of compatibility.
- *External views:* Larger buildings or more intense building programs may impact the extent and quality of exterior views. Denser development requires careful site planning to ensure privacy and minimize the visual and noise impact of abutting uses.

### Financial Impacts and Opportunities

- *Infrastructure:* While higher-density development generally requires higher total infrastructure costs, it may result in lower construction, operation, and service costs on a per unit or square foot basis.
- *Land:* Higher-density development permits land costs to be distributed within a larger development program, resulting in lower per unit land costs.
- *Site amenities:* Although higher density may require a more elaborate amenity program, the increased project development yield and resulting revenue may provide an incentive for such provisions.
- *Construction:* Typically, more intense development results in higher total construction costs. It may foster certain economies in construction due to the potential for distributing costs over a larger program base. Concentrated building programs generally provide a lower per unit construction cost than a similar program dispersed over a larger area. Such reduction results from savings associated with a more compact building envelope and shorter lengths of expensive infrastructure.
- *Public/semi-public facilities:* Aggregate and per unit operation and maintenance costs of select facilities may be reduced with denser development patterns. This is particularly important for major capital facilities, such



**FIGURE 12.4** Planned community.

as a sewerage treatment plant. These may be funded by municipal bonds or special assessment or development association fees and rely heavily on project usage to retire debt. The building of major private facilities, such as utility transmission lines, also relies on demand to justify new construction.

- *Tax revenue:* Tax revenues generally increase on an acreage basis as a result of public and structural improvement, and subsequently increased density normally results in a higher average per acre assessed value.

- *Municipal service:* Public service improvements can more readily be supported by concentrated population and development patterns, which afford a more efficient delivery system.

### Operational Impacts and Opportunities

- *Maintenance:* Occupants realize lower per unit maintenance costs due to economies of scale associated with larger, more intense development patterns.

- *Energy:* Per unit energy cost savings are generally realized in development that is more compact.

### Marketing Impacts and Opportunities

- *Market exposure:* Increased scale, intensity, or quality of development may foster a visual presence, which translates into greater market exposure. Depending on the actual development program, this may increase project appeal to a broader spectrum of the market or increase awareness among targeted consumer groups.

- *Community identity:* Larger-scale developments can provide additional justification for amenities, which may, in turn, increase community identity and resident/tenant affinity with a project.

## RESIDENTIAL LAND USE

Typically, housing is the most prevalent land use within a jurisdiction. Residential design requires sensitivity to the pragmatics of sound construction economics, an understanding of site conditions, and an appreciation for consumer lifestyle preferences. Similarly, sound residential planning and design requires a genuine appreciation for the individual dwelling unit.

### Single-Family Detached

In a single-family detached residential dwelling, each individual living unit is a freestanding structure. Each dwelling unit normally occupies a separate recorded lot. In a conventional lot arrangement, the single-family home is surrounded on all sides by property or yards reserved for the occupant's exclusive use. The lot must meet a minimum or average size prescribed in the community's zoning ordi-

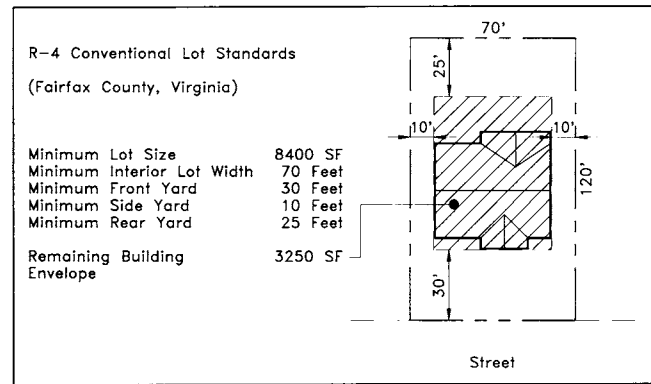


FIGURE 12.5 Typical conventional lot.

nance. Similarly, in a cluster lot, the single-family structure is surrounded on all sides by yards reserved for the occupant's exclusive use; however, the lot may be smaller than normally prescribed under conventional zoning and the yard setbacks are generally lessened. Figure 12.5 is representative of a typical conventional subdivision lot, and Figure 12.6 represents its cluster equivalent.

Utilization of cluster lots requires greater attention to house siting and house-to-house relationships, given the need to preserve individual unit privacy and maintain community design standards among smaller average lot sizes. The benefit of cluster development rests with the potential reduction in per unit infrastructure costs, given the aggregation of dwellings on only a portion of the project acreage. Similarly, the conservation of community open space in cluster arrangements and its amenity appeal may offset market concerns about smaller lot sizes and the potential reduction in individual unit privacy. Conventional or cluster arrangements alone do not dictate lot size. Relatively speaking, large and small house types may be developed in either arrangement. While acre and multiple-acre home sites are common in suburban areas, more typical single-family detached lots ranged from  $\frac{1}{4}$  to  $\frac{1}{3}$  acre in size throughout most of the last several decades. However, current trends are moving toward smaller lot sizes, given escalating land costs and changing lifestyle requirements. Single-family de-

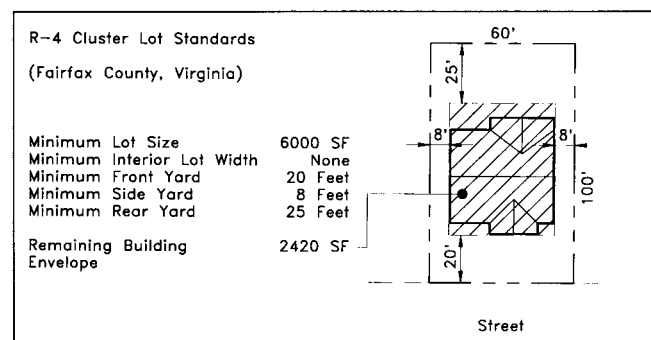


FIGURE 12.6 Typical cluster lot.

tached dwellings on 2000–5000-ft<sup>2</sup> lots are increasingly common in both in-fill developments and planned communities.

Lots are normally square or slightly rectangular in shape. Minimum lot widths are generally governed by local zoning ordinance. Decisions to exceed that dimension are often prompted by market demand for additional distance between dwelling units, the dimensions of the proposed house, or preferred driveway approach to the garage. An attached side-loaded garage requires a wider lot than a similarly sized front-loaded facility.

Lot lines normally run perpendicular to the street frontage. Variations in lot configuration commonly include pie-shaped lots around cul-de-sacs and flag or pipestem lots. The latter convention provides limited lot width along the public street frontage to accommodate driveway access and a widening of the lot at the building setback line sufficient to satisfy the desired house type and its yard requirements. Examples of these lot configurations are presented in Figure 12.7.

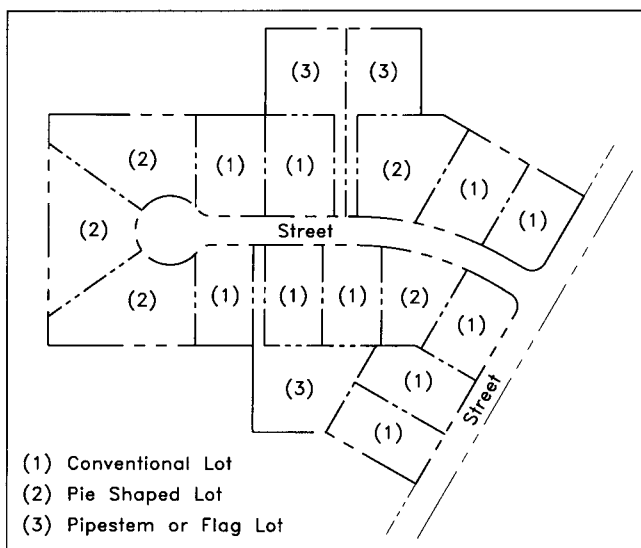
Pipestem or flag lots were originally designed to allow access to otherwise landlocked parcel acreage. Use of this technique recognizes the environmental and economic advantage in substituting private drive lengths to tap land that otherwise would require additional street length and potentially greater site disturbance and infrastructure costs. The negative attributes of this technique include the potential burden on homeowners to individually maintain longer driveways or private street lengths, potential access constraints for emergency vehicles, and possible undesirable house-to-house relationships, as pipestem dwellings may be perceived to be in the rear yards of adjacent residences. However, judicious use of pipestem lot arrangements can provide distinct benefits in residential design when its use, resulting lot size, dwelling orientation, and access consid-

erations are based on sound site planning and community design criteria.

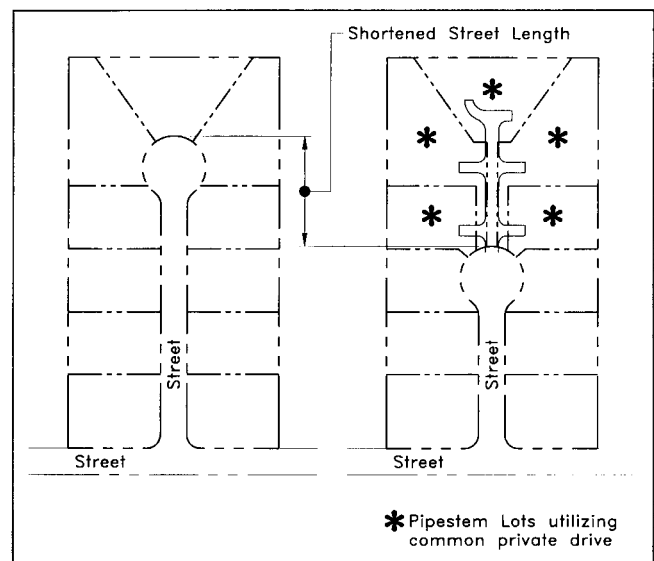
Zoning ordinances generally provide setback and yard requirements for single-family residential development to ensure functional side, rear, and front yards. Front yard setbacks are generally a minimum of 20 to 30 ft to accommodate the length of a parked vehicle in the driveway. Front yard setbacks may vary depending on the nature and scale of the adjacent street. Lesser setbacks may be acceptable where alternative parking arrangements are provided, as is the case where rear-accessed alleys or service drives are employed. Deeper setbacks may be required along more heavily traveled arterial streets than on neighborhood streets or cul-de-sacs with less traffic volume.

Demand for affordable single-family detached housing in a time of escalating land prices has prompted greater reliance on smaller lots. The smaller lot size not only appeals to builders and developers who can realize lower per unit infrastructure costs and higher per acre unit yields but responds to consumer preference for the reduced maintenance requirements associated with the smaller yards. Smaller lots in well-designed communities with provisions for common amenities appeal to a lifestyle that seeks convenient accessibility to diverse leisure activities. Safeguards to a well-designed small lot community reside in well-conceived architectural design of the unit and sensitive site planning.

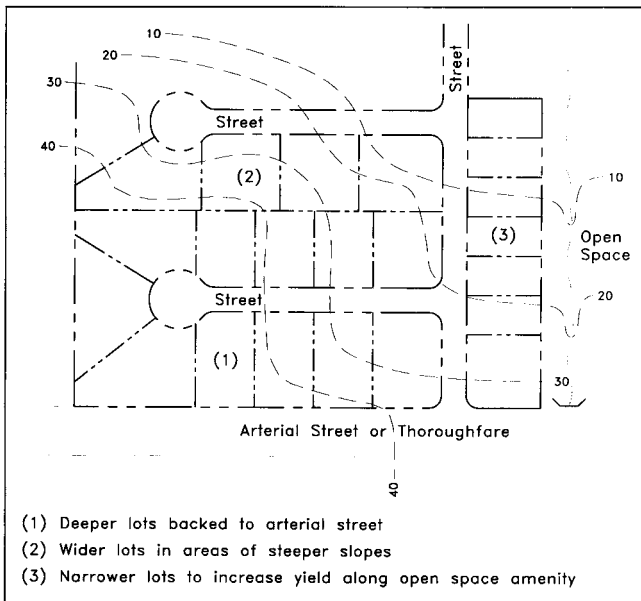
**Design Considerations.** While conventional subdivision regulations normally prescribe setbacks for front, rear, and side yards, dwelling unit design, variation in lot sizes, lot dimensions, and concessions suited to cluster or unique detached housing prototypes provide opportunities to tailor design to local market and site conditions. Several ground rules can provide a strong foundation to residential layout design. These include:



**FIGURE 12.7** Example of variation in lot size due to site characteristics.



**FIGURE 12.8** Example of the use of flag lots in lieu of extended street length.



**FIGURE 12.9** Example of variation in lot size due to site characteristics.

- Lot and place houses in a manner that responds to the site's natural features.
- Avoid extremes in either random plan organization or uniformity to the point of monotony in the pattern of development.
- Establish appropriate block lengths to minimize long views of repetitive house-fronts, garages, and driveways as well as foster convenient vehicular and pedestrian movement.
- Ensure yard dimensions and unit orientation that promote individual dwelling legibility and sufficient yard privacy.
- The street pattern should result in a coherent and legible circulation system to support an identifiable sense of place.
- Building setbacks should reinforce the street hierarchy.

### Single-Family Semidetached

The single-family semidetached dwelling, commonly referred to as a duplex unit, consists of two living units that occupy a single structure separated by a common wall. Each dwelling has its own exterior entrance and is located on a separate recorded lot. Each dwelling unit is surrounded on three sides by property reserved for the occupant's exclusive use.

### Single-Family Attached

Single-family attached dwellings include housing types with multiple dwellings in a building arrangement where two or more dwellings share common walls, and in some cases

common floors and ceilings. Each unit has a separate private entrance at grade level and usually a contiguous exterior yard reserved for the occupant's exclusive use. Variations may include:

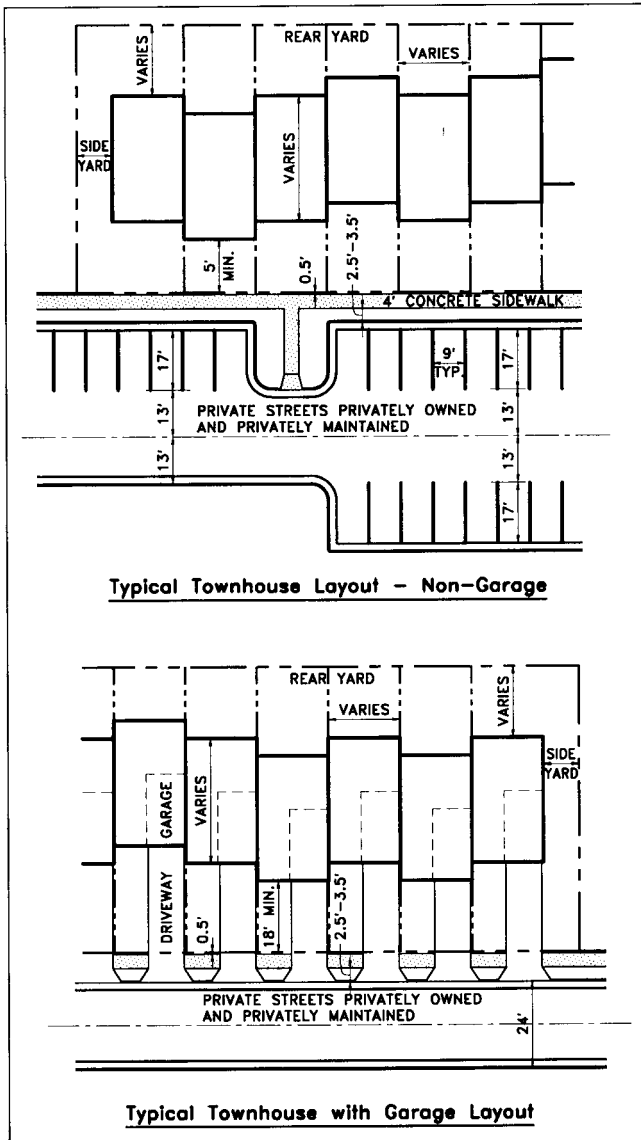
**Townhouse (Row House or Townhome).** Made up of three or more units attached side-by-side, the townhouse, long a common urban housing prototype, has become a popular component of the suburban housing market. A common or party wall separates individual units, each with an at-grade entry. Property on at least two sides, normally front and back, is generally reserved for the occupants' exclusive use.

The townhouse provides flexibility in plan and style, with design variations that include integral or detached garages. Indoor/outdoor relationships generally focus on the rear yards, although they may orient to small front courtyards. Exterior yard privacy may be enhanced with landscaping, fencing, or privacy walls. Lot widths can vary depending on the desired character of the unit. While conventional suburban townhomes may range from 16 to 24 ft wide, units of 32 ft or greater are not uncommon when market considerations warrant a street presence approaching that of a single-family detached residence. At the more conservative widths, lot depths may range from 60 to 100 ft. Densities normally range from 7 to 12 units per acre. Depending on the unit width, parking requirements, site conditions and desired community character, densities of up to 20 units per acre are not uncommon in more urban settings. The figures below illustrate a range in townhouse configuration, including the more conventional front-loaded unit, courtyard or clustered townhouse court, and alley-served units.

Each townhouse dwelling has a separate front door, although end units may be side entry. Generally, each house has its own utility connections. Typically, local ordinances prescribe the maximum number of side-by-side units that may be aggregated into a single townhouse "stick" or building group. While sticks of six to eight townhouses have been a common development practice, combinations of three to four units, comparable in scale to a large single-family residence, are popular in some high-end market segments.

Building lengths with more than eight attached units are more commonly found with narrow, 18-ft or less average unit widths. Shorter lengths, and a higher percentage of end units are generally associated with wider upscale townhomes or in instances where parcel dimensions preclude longer lengths.

While the urban townhouse was configured in a linear arrangement with units oriented to the street, the suburban townhouse community has taken on a more organic arrangement. This has occurred in response to site conditions, which have accommodated a more expansive layout, as well as a proclivity toward arrangements in which the units' fronts face towards the parking lots and whose rears are open-space amenities. The urban street setting, with its attendant parallel parking and supplemental rear service alley access, had been preempted, in the suburbs, by the front



**FIGURE 12.10** Typical townhouse condition.

service drive and its associated perpendicular parking. Variations do include townhouse layouts, which focus on a court and mews arrangement of units. The former generally results in a cluster of townhomes fronting on a common parking area, often referred to as parking court or parking bay. The common vehicular circulation area serves as the organizer for the cluster. The mews arrangement focuses on a pedestrian open space to organize the front facades of the individual townhomes, with parking occurring at the terminus of the pedestrian area or on the rear or underside of the units.

**Back-to-Back Townhouse.** While considered an attached multifamily housing type in many jurisdictions, the back-to-back townhouse often divides the length of a traditional townhouse stick with an additional common wall that serves to separate units organized along opposing facades. The resulting aggregate building form has two front yards. The double frontage prompts a need for careful site plan-

ning to ensure the convenient placement of adequate parking for each unit. Reliance on a single exterior exposure for each residence requires subtle design considerations to balance the semipublic nature of a front entry with any privacy requirements associated with the singular exterior living space, which occupies the same facade.

**Other Townhouse Variations.** Another variation to the townhouse is the stacked or piggyback townhouse. This may include a multistory townhouse unit over a single-story flat or a multistory townhouse over a similar multistory unit. Such arrangements work well on sites where topographic change will allow upper and lower units to be accessed at grade on opposing sides of a building. In extreme instances, the lower unit may be reduced to a single exterior exposure.

Alternating townhouses with stacked flats in the same building can produce additional diversity. The flats, sometimes referred to as coach homes may be located either at building ends or in interior locations. This arrangement is often classified as a multifamily residential product subject to zoning requirements associated with that use category. Figures 12.13 and Figure 12.14, respectively, illustrate the combined townhouse/coach house and piggyback/stacked townhouse.

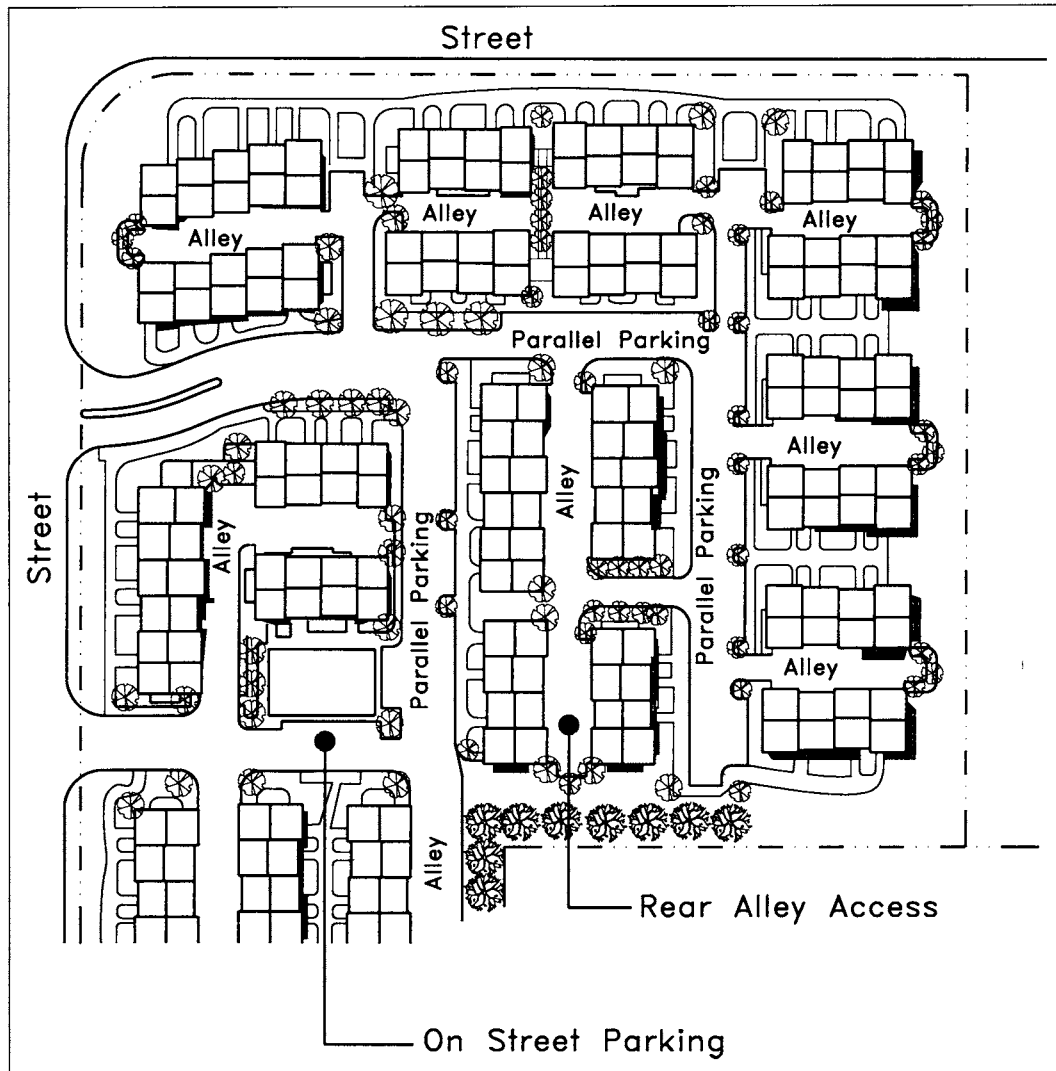
**Multi-Plex.** Used with a prefix representing the number of attached units, such as duplex, triplex, quadraplex, etc., multiplex residences consist of two or more units in various configurations, each with its own separate at-grade exterior entrance. Piggyback townhouses and manor houses are often considered multiplex units. The yards surrounding a typical multiplex unit may be reserved for common occupant use or portions of the yards may be designated for an individual unit's exclusive use. Figure 12.15 shows a variety of multiplex combinations.

As the number of units in this type of attached residential arrangement increases, unique opportunities and constraints in both design and construction arise. Unit design, solar orientation, indoor/outdoor relationships, privacy provisions, convenience of automobile access, and parking must be weighed against the increase in density and reduction in per unit land, construction, and infrastructure costs.

### Multifamily Residential

Multifamily structures are characterized by four or more units in a single building, with units sharing access to the exterior by one or more common entrances. Structures may be one or more stories, and multifamily communities or developments may consist of one or more buildings on a single tract or lot. Structures are surrounded by common open space, which often contains on-site recreational facilities for resident use. Private drives or service roads provide access to surface parking lots or in some cases structured parking, private garages, or carports.

Most zoning ordinances specify detailed design criteria and development standards for multifamily projects. Project density is usually set as a maximum number of dwelling units per acre or, in some instances, may be expressed as a



**FIGURE 12.11** Traditional townhouse unit arrangement.

maximum permitted square feet of building or floor area coverage per acre of land.

Minimum standards are adopted to guard against overcrowding, ensure access to light and air, protect privacy, and ensure compatibility between land uses. Most jurisdictions seek to locate multifamily residential development in areas where land costs are high. Typically, this type of development coincides with areas of considerable existing or planned investment in public infrastructure, or where proximate commercial retail and office activity would benefit from higher-density residential development. In this regard, multifamily housing often provides the buffer between non-residential activity and single-family attached and detached developments.

Due to the more intense traffic generation resulting from the higher densities associated with multifamily housing, convenient access to major collector or arterial streets is desirable. Depending on the size of a development, local fire and public works officials may require at least two entrances or connections to a public street. This allows access

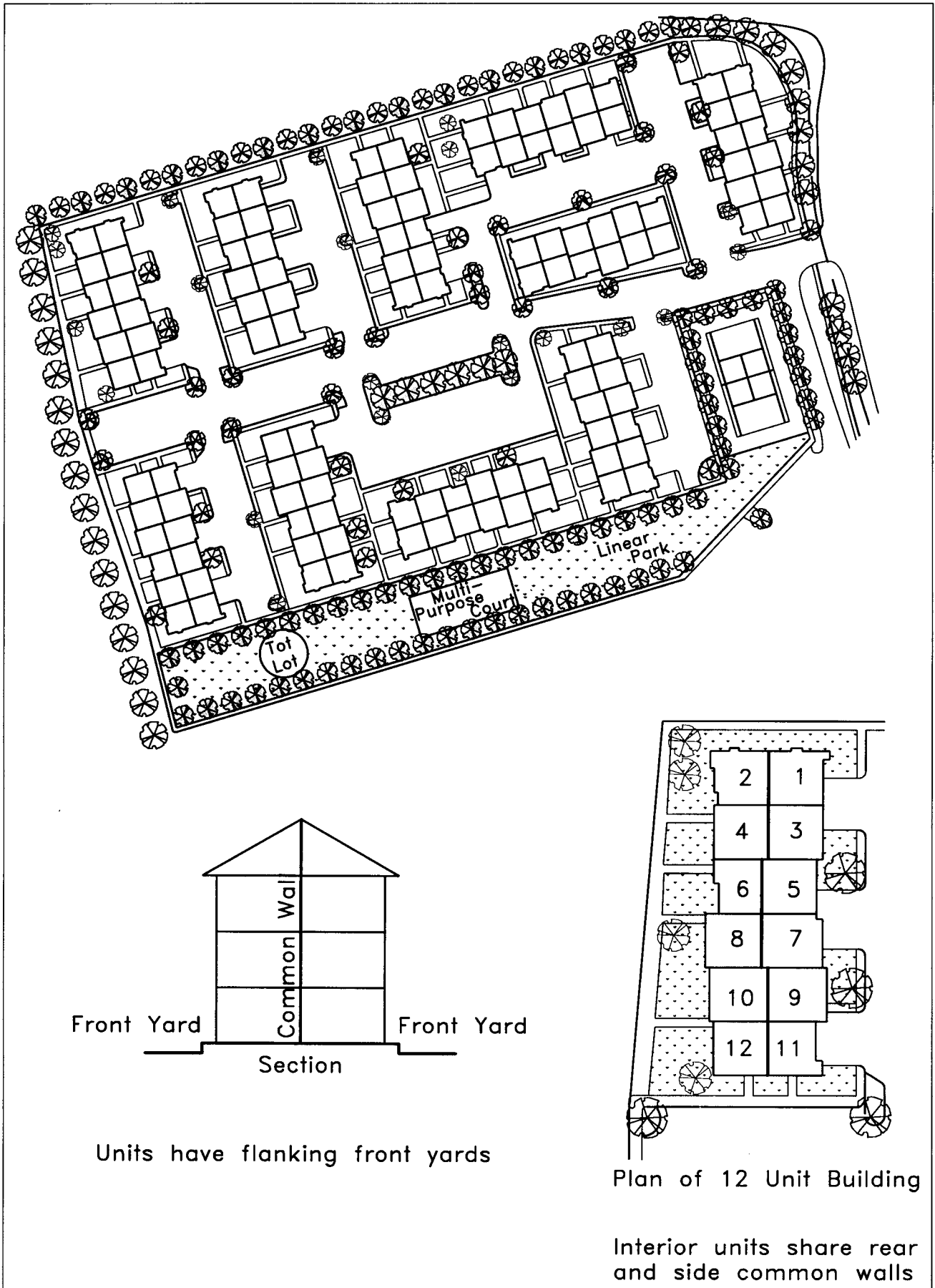
to be maintained in the event that one entrance becomes obstructed.

Site planning for multifamily developments requires an appreciation and understanding of the proposed building design. The higher densities, size of structures, and relationship between structures and site elements leaves less latitude than may be afforded in site design for less dense residential housing types. The placement of driveways, location and sizing of parking areas, and other infrastructure considerations require knowledge of building dimensions, elevations, and orientation.

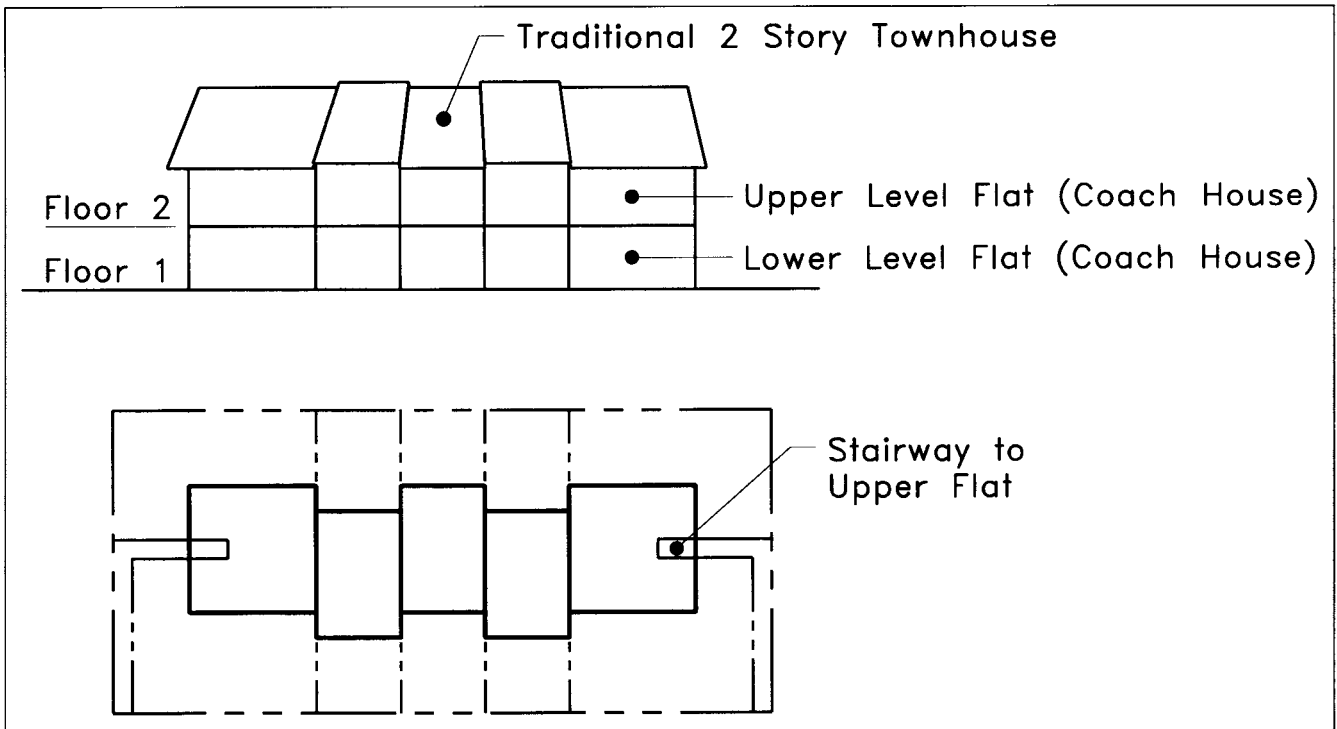
Multifamily structures are generally categorized as either garden or low-rise, mid-rise, or high-rise structures. Distinctions and characteristics of each of these are described below.

*Garden apartments*, sometimes referred to as walk-up or low-rise, consists of one- to four-story structures. The individual apartment units may be arranged along corridors or around common stairwells, which may be enclosed within the structure or unenclosed and integrated into the





**FIGURE 12.12** Back-to-back townhouse.  
 Downloaded from Digital Engineering Library @ McGraw-Hill ([www.digitalengineeringlibrary.com](http://www.digitalengineeringlibrary.com))  
 Copyright © 2004 The McGraw-Hill Companies. All rights reserved.  
 Any use is subject to the Terms of Use as given at the website.

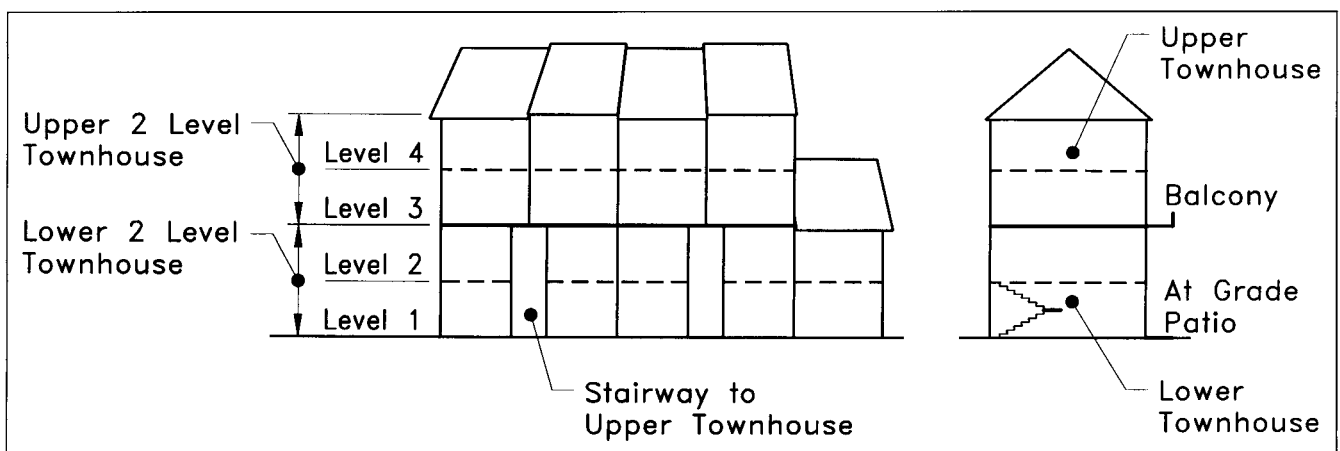


**FIGURE 12.13** Combination townhouse and coach house arrangement.

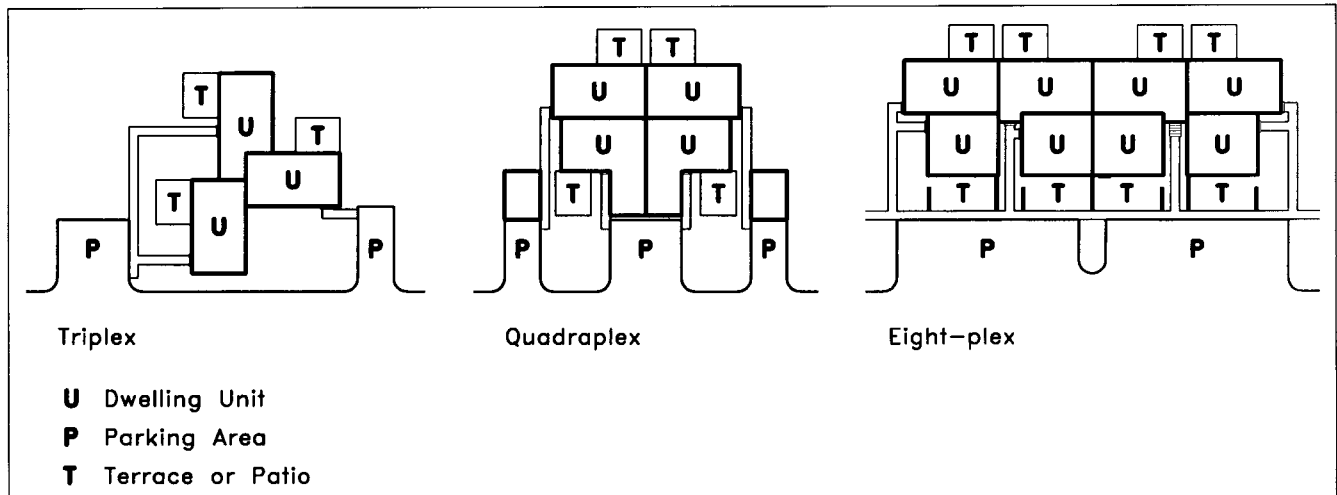
exterior architectural design. While some units may have direct private entrances at grade level, normally units share common entries. As the name implies, the garden apartment focuses on melding the dwelling unit to the building grounds. The building units are generally organized around landscaped open space and parking areas. Ground floor units may have direct access to exterior patios or garden areas, and upper floors may have balconies or terraces, providing each individual unit with exterior living space.

Garden apartment building configurations vary depending on a myriad of marketing, program, and site considerations. They can range from single-loaded buildings in

which the individual unit runs from the front to the rear of a building (through-units) to double-loaded arrangements where in each unit has a single (front or rear) exposure. In terms of site design, awareness of interior room organization and relationship to exterior space is important to promote desirable views and to enhance individual unit privacy. This is particularly important where parking is concentrated near the building. Provision of intermediate landscape between the unit and the parking area enhances the livability of the dwelling, particularly if it is the unit's sole exterior exposure. Figure 12.16 illustrates single- and double-loaded garden apartment layouts.



**FIGURE 12.14** Piggyback or stacked townhouse.



**FIGURE 12.15** Typical Multiplex Combinations: triplex, quadraplex, eightplex.

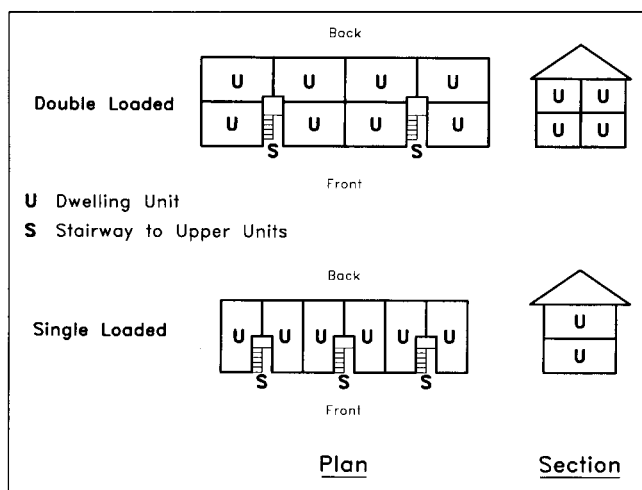
Garden apartments afford significant flexibility in site design due to the adaptability of various building forms to divergent site conditions. Unit densities can range from 10 to 20+ units per acre, depending on the building configuration, unit sizes, number of stories, parking, open space, and amenity packages. In an effort to appeal to varying markets, garden apartment communities increasingly promote amenity packages that expand beyond the traditional swimming pool to include facilities complete with spas; exercise, game, meeting and community rooms; and expanded site improvements including tot lots, court facilities, trails, and park courses. Multifamily residents increasingly desire amenities comparable to those provided in single family residential communities. Even sheltered parking such as carports, or more infrequently individual garages, is an increasingly popular element in select garden apartment communities.

*Mid-rise and high-rise multifamily residences* are generally four- to eight-story structures. High-rise residences normally exceed eight stories. Both rely on elevators for vertical circulation. Individual dwelling units access common corridors. Units are generally arranged along opposite sides of the corridor, providing a single exterior exposure for the principal interior living spaces. While the economies of construction weigh heavily in favor of this arrangement, unique market considerations may prompt alternative design, such as single-loaded corridors, as in the case where a unique amenity mandates that all units have comparable exposure to the attraction.

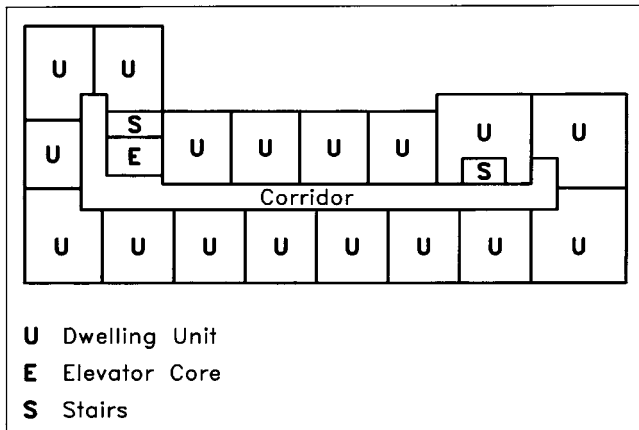
Internally, buildings are organized around the elevator, utility, and a stairwell core, which may be central to the building floor plate. Entered at a common lobby, or multiple lobbies at differing levels, if terrain permits, the corridors penetrate the building extremities providing access to the individual residential units. Additional stairwells positioned at the further reaches of the corridor provide an alternative means of access/egress. The concentrated need for service and delivery is generally satisfied by a central loading area, which is often located to the rear or side of the building removed from main tenant access and view. Figure 12.17 is illustrative of a typical mid- and high-rise multifamily floor plate.

Four- to eight-story multifamily buildings can achieve densities of 30 to 40+ units per acre. Provision for parking increasingly relies on subsurface or structured arrangements to maximize land use efficiencies and promote tenant convenience and safety.

Many of the amenity provisions of mid- and high-rise residential buildings may be internal to the structure, but exterior elements may include a full range of recreational and leisure activity areas. In more urban settings, this may include indoor or rooftop swimming pools, landscaped arrival courts, and lush landscaped perimeters. On more ex-



**FIGURE 12.16** Example of double-loaded and single-loaded garden apartments.



**FIGURE 12.17** Typical mid-rise and high-rise multifamily housing floor plate.

pansive sites, the provisions may include tennis courts, elaborate pool facilities, and exterior exercise and recreation offerings in a park-like setting.

### Site Design: Residential Development

Local zoning codes and planning documents generally provide the principal guidelines and criteria governing residential development patterns. These include stipulations as to means and methods for ensuring code compliance. Some of the more conventional considerations normally include:

- Maximum permitted density, including means and method of calculation. Density calculations may require the omission of all or portions of a site area encumbered by environmental constraints such as steep slopes, floodplain, wetlands, major utility easements, and public rights of way
- Minimum acreage assembly for select residential types. Often cluster or planned development districts require a minimum amount of acreage to be eligible for that zoning district
- Minimum yard requirements, including front, rear, and side yard requirements, may be expressed as a minimum dimension or relationship to the height of the building
- Minimum open space provisions, including potential distinction among usable open space, dedicated open space, and common green space. Ordinances may discount or cap the amount of public open space allowed as part of the project open space calculation. Similarly, a percentage of the total open space may need to meet certain location, distribution, or performance criteria, such as size, shape, and topographic considerations, or be exclusive of floodplains, utility easements, or wetlands

- Screening and buffering standards, given the specific use or use intensity proposed in relationship to the nature and density of adjoining uses
- Maximum lot coverage
- Minimum lot dimensions and frontage requirements, including distinction between interior and corner units/lots, pipestem or flag lots, and cul-de-sac lots
- Maximum number of pipestem lots permitted
- Lot restrictions regarding inclusion of environmentally sensitive areas and utility easements
- Parking and access criteria, which may include the number of parking spaces required, limitations on public or private street length, and street loading capacities
- Maximum building height and methods for calculating height
- Minimal landscape or tree canopy requirements

In attached and multifamily housing types, considerations may include:

- Minimum unit width or size
- Limitations regarding maximum number of units that may be grouped together
- Maximum distance between unit entry and parking
- Variation in architectural facade and setbacks
- Minimum setback from common property lines
- Minimum distance between buildings
- Standards and criteria for determining the number and placement of loading and service areas
- Building and fire code requirements

### NONRESIDENTIAL LAND USE

Principal nonresidential development in the suburban environment focuses on retail, office, and industrial land uses. The following discussion highlights the more prevalent building types and site considerations appropriate to each.

#### Market Considerations

Provision of retail, office, and industrial development programs can be prompted by a myriad of factors, including observation of a demonstrated need for local distribution of goods and services at a neighborhood or community level; recognition of an opportunity to fulfill a local jurisdictions development goal in expanding either local or regional economic base; or recognizing local trends focusing on the growth, consolidation, or relocation of business ventures within an area. Regardless of the genesis or justification for such facilities, site design and site execution must be si-

multaneously sensitive to the requirements of the provider, prospective tenant and patron, site, and surrounding community.

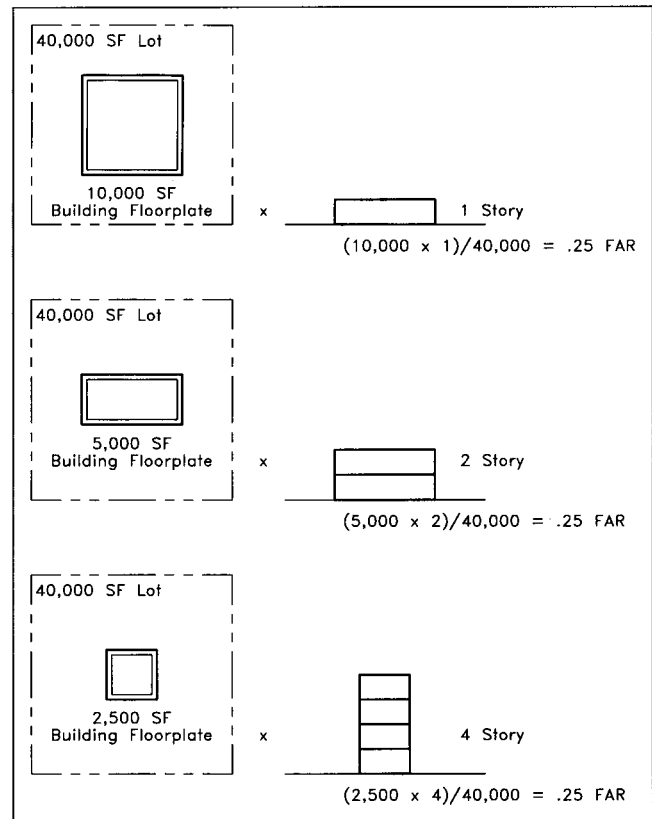
### Design Considerations

As with all land development activity, commercial site design criteria stem from two principal sources. The first consists of standards associated with the prospective use. This includes the spatial characteristics and site provisions requisite to housing and servicing the facility and its tenants and customers. The second set of criteria concentrates on local land development plans and controls. While the zoning ordinance is the primary gauge for establishing site performance, other considerations, such as environmental, fire, and safety compliance, factor heavily into facility and site design. This set of criteria may vary among jurisdictions and the nature and intent of the specific land use under consideration.

**Common Standards.** Subsequent to determination that the proposed use is appropriate for a given property, setback, height, bulk, and other dimensional and performance standards constitute the principal site design criteria. These standards are normally contained in most jurisdictional subdivision and zoning regulations.

**Floor Area Ratio.** Nonresidential development is most frequently programmed and sized on the basis of the aggregate square feet of built structure relative to development site area. The common method of computation is the floor area ratio (FAR), a method of measurement expressed as the relationship of total building square feet to the total site area. For example, as shown in Figure 12.18, a 10,000-ft<sup>2</sup> building located on a 40,000-ft<sup>2</sup> parcel would constitute a floor area ratio of 0.25 (building area divided by parcel acreage). The number of floors associated with the built product does not influence the floor area calculation. In the cited example, the 10,000-ft<sup>2</sup> building might be constructed as a two-floor building of 5,000 ft<sup>2</sup> each, or a four-story building of 2,500 ft<sup>2</sup> each. The floor-area-ratio is 0.25 in all cases. Most jurisdictions do not include parking structures in floor area computations. Similarly, exclusions may include portions of the building that are below grade or space programmed for select circulation, service, and support facilities. It is important to be aware of the method for calculating FAR in each jurisdiction.

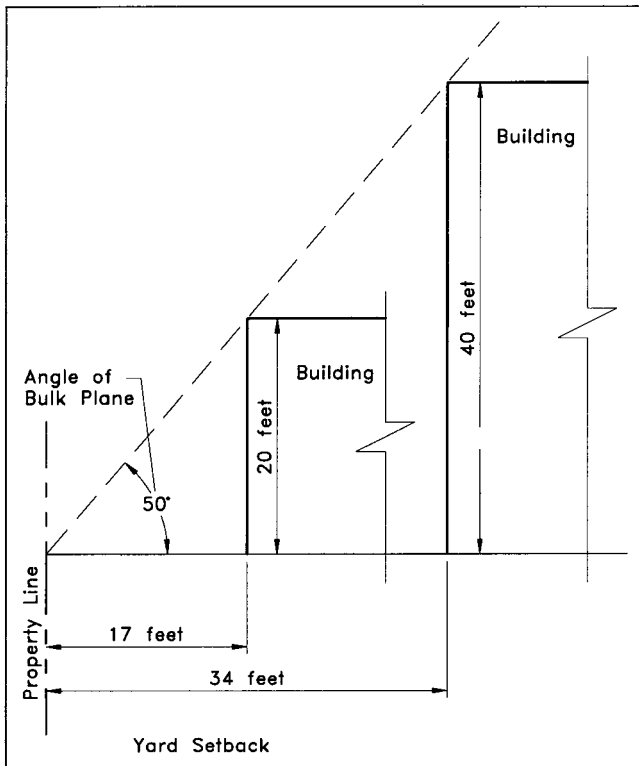
**Lot Size and Bulk Regulations.** Minimum lot size, building height, bulk, setback, and yard requirements are normally stipulated in the individual zoning categories for nonresidential uses. The criteria generally vary in relation to the intensity of the desired development and the characteristics of the adjacent land use. Areas designated as more urban will normally require smaller setbacks from parcel boundaries. While a maximum building height may be stipulated, the permitted height of a structure may be governed by performance criteria. A common method of control focuses on maintaining a relationship of building



**FIGURE 12.18** Example of comparable floor area ratios.

height to peripheral yard setbacks. If an acceptable angle of bulk plane from the top of building to a given property or parcel line is designated, the setback can increase proportionately as the building height increases. For example, a required 40° angle of bulk plane might require a 17-ft setback for a 20-ft high building and a 34-ft setback for a 40-ft tall building. Other jurisdictions may express a minimum yard setback based on a given building height and simply stipulate that for every additional foot of building height the setback dimension will increase a set distance. Figure 12.19 illustrates the concept of angle of bulk plane for regulating building height.

**Parking, Loading, and Service.** Criteria governing minimum parking, loading, and service are normally expressed in relationship to the floor area of the proposed building program. Typically, the zoning ordinance stipulates this standard as a minimum number of parking spaces to be provided on-site, although in some cases a portion of the parking may be provided off-site or in combination with adjacent uses. Jurisdictions vary in terms of how they express this relationship. It may be based on gross building square footage, net building square footage, net leasable or tenant area. The designer must understand the method of calculation, ascertain the minimum requirement based on public standards, and ensure that such a minimum provision has client and market acceptance. Unique program re-



**FIGURE 12.19** Illustration of angle-of-bulk plane.

quirements or marketing considerations often prompt a need for parking and loading space provisions to be in excess of the minimum zoning criteria.

**Landscape and Open Space.** For nonresidential uses, this is typically expressed as a percentage of the total site area that is not encumbered with the building, vehicular circulation, and parking. While it is normally considered the residual portion of the site remaining for landscaping, in some localities it may be more conservatively delineated to exclude pedestrian walkways, patios, and similar site improvements, which are deemed impervious surface treatments. Landscape requirements are generally categorized as two main types.

Peripheral yard and buffer area requirements may be stipulated for nonresidential projects, which are deemed incompatible with surrounding land uses. These may be prescribed as a fixed dimensional width with certain *pro forma* landscape treatments or as a performance criteria requiring a combination of plant material and/or structural elements that provide an equal or improved buffering condition.

A second set of standards focuses on minimum landscape improvements required given specific development program components. These may deal with shielding or screening of parking areas, service and loading zones, minimal streetscape standards, foundation planting, parking lot landscaping, or other specific concerns of the local jurisdiction.

## RETAIL

Retail development may be loosely categorized according to the size and physical characteristics of the proposed facility. The principal types of retail locations include freestanding, strip, center, and mall arrangements of sales and accessory uses.

### Freestanding

The freestanding retail establishment has taken on reduced import in the suburban market, given trends focusing on the aggregation of retail uses and the benefits associated with collective marketing, expanded visibility, and customer capture associated with more encompassing retail or mixed use projects. Where a freestanding retail use does exist, it is generally positioned to respond to several sets of location criteria. It may be an establishment, sited on a separate recorded lot or parcel, that is part of a larger community of like uses, such as a town or village center or in a linear arrangement along a major road or highway. It may constitute a modest forerunner retail service in a geographic area of insufficient population concentration to warrant more expansive retail development. It may represent a national or local business with a sufficiently unique market to generate patronage at a location apart from other retail trade or whose functional requirements are best satisfied as a freestanding structure.

In today's suburban retail market, freestanding retail establishments are commonly associated with locations within or proximate to larger retail or shopping center. Establishments such as restaurants, banks, movie theaters, bowling alleys, gasoline stations, office buildings, and similar singular operations may be located on separately recorded lots, sometimes referred to as pad sites, within the parking area or along the road frontage of larger retail centers.

The size and functional characteristics of such establishments vary considerably. However, building visibility, the number and convenience of customer parking, and the size and location of loading and service requirements generally represent the primary site design criteria that establish the pattern for such uses. Where such a facility is contemplated at isolated locations, one can anticipate a greater need to ensure measures of compatibility with surrounding development. Integration of freestanding retail uses as an adjunct to neighboring strip, center, or mall retail development patterns generally presents less difficulty in ensuring community fit and may afford advantages to the extent parking, street improvements, and other infrastructure considerations may serve multiple users.

### Strip Centers

Strip centers literally represent the aggregation of retail uses in a linear arrangement, most notably with their front facades paralleling roadways or positioned in an "L" configuration at the intersection of major transportation routes.

Generally, the establishments are one store deep, share common interior walls, and are linked by a common pedestrian walkway across the storefronts. The bulk of parking is aggregated into one or more principal parking areas normally located between the store facade and the street. Strip centers rely on a strong visual relationship to the adjacent street frontage for identity and marketing. Strip retail arrangements can vary in size, however, once the linear distance across the collective storefronts exceeds  $400 \pm$  ft pedestrian circulation between stores begins to subside. Once a larger retail establishment, such as a major grocery store or drug store, is introduced into a linear arrangement, the distinction between strip and retail center becomes less clear. Figure 12.20 illustrates the typical strip center configurations.

### Retail Center

Finding definitive criteria to distinguish a retail center from a strip center can be difficult, given the negative connotation that the latter has acquired. Retail centers share in their reliance on common parking areas, major pedestrian circulation across storefronts, and maintenance of a reasonable relationship and visibility to adjacent roadways. A distinction can best be found in the number of potential anchor or larger retail establishments that are located in a retail center, an abandonment of sole reliance on a linear arrangement for all of the retail establishments, and the introduction of smaller pad site within the retail center site. Centers often are arranged in an "L" or "U" shape, with anchor stores occurring at the building extremities and/or central to the parking lot. These arrangements take advantage of parcel depth, and lining multiple sides of the parking area with stores reduces the distance between the parking stalls and a variety of retail establishments. The introduction of free-

standing retail establishments such as banks, gas stations, and restaurants at the center or along the roadway frontage gives a functional complexity to a retail center and requires careful attention to site details to ensure sufficient visibility, access, convenience of parking, and service and loading. Figure 12.21 represents a typical retail center design.

### Retail Mall

The distinguishing characteristics of the retail mall include the separation of major pedestrian movements from peripheral vehicular parking and circulation. Generally located between opposing store facades, an internal pedestrian walkway or mall may be either open or under roof. Both arrangements provide an opportunity to direct customers along expanses of intervening retail establishments.

A typical mall is presented in Figure 12.22. A normal convention in organizing on-site vehicular circulation is to position a service drive at the building perimeter and a perimeter road near the outer edge of the major field of parking. Major approaches to the mall tend to be aligned with main store anchors. In many instances, freestanding

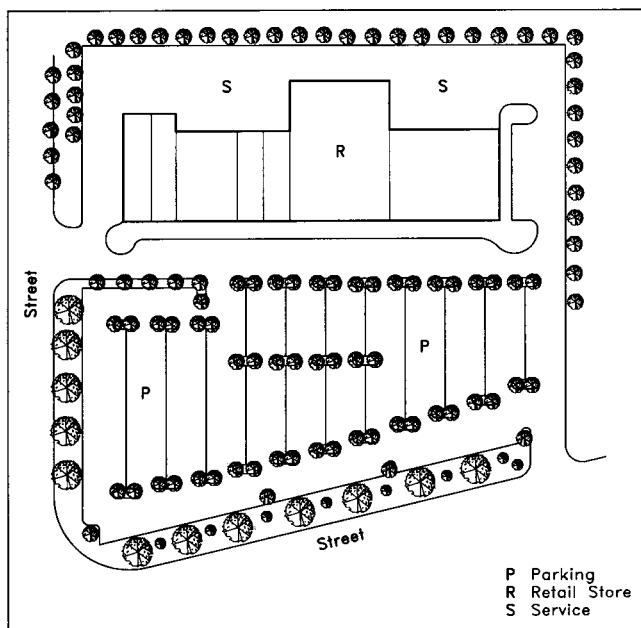


FIGURE 12.20 Typical strip center arrangement.

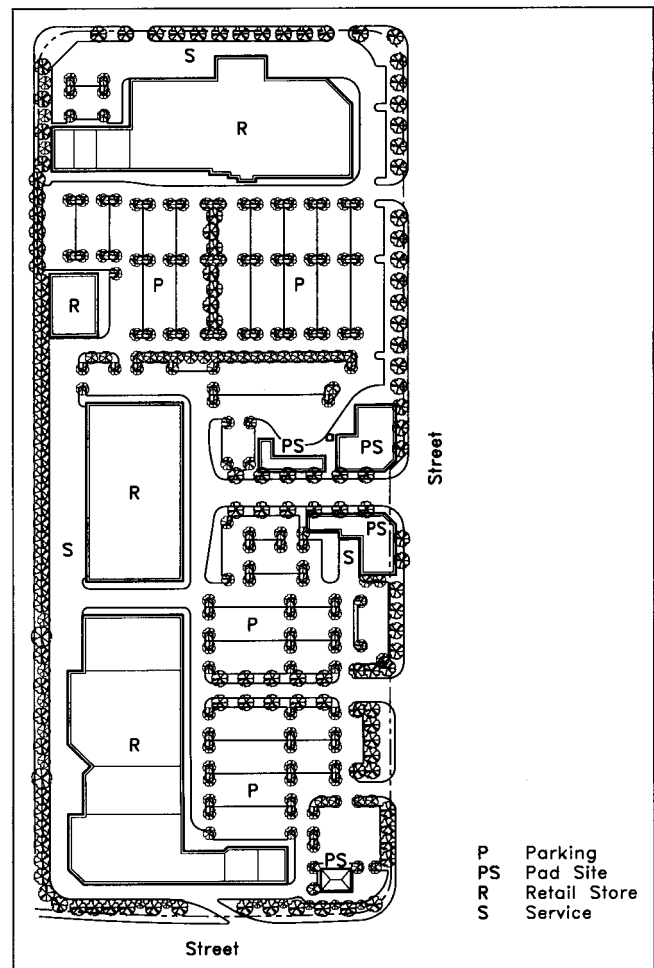


FIGURE 12.21 Typical retail center arrangements.



**FIGURE 12.22** Retail mall arrangement.

uses (pad sites) may occupy acreage between the outer mall loop drive and the perimeter street network.

Figure 12.23 illustrates the front-sided multilevel parking arrangement employed at the Tysons II retail mall in Fairfax, Virginia. In the background (right rear), parking decks have been constructed over former surface parking lots to accommodate the expansion of neighboring Tysons I.

There is a considerable range in the size of retail malls, with major regional serving facilities often exceeding 1 million ft<sup>2</sup> of leasable area. Increased reliance on structured

parking can assist in reducing distances between parking areas and retail stores. The resulting site solution often entails the bridging of peripheral service drives around the center perimeter, and in the case of multilevel malls may result in pedestrian walkways that connect to various levels of the center.

### **Retail Center Design Considerations**

Site design for retail centers should be predicated on overall customer ease in identifying and locating the individual tenants within the center. Convenience of vehicular access, ad-





**FIGURE 12.23** Retail mall with structured parking. (Photo by Larry Olsen)

equacy and placement of customer parking, and safety of pedestrian movements between parking areas and retail establishments are all fundamental site planning considerations. A secondary set of criteria focuses on the internal servicing and operation of the center, particularly in regard to separation of service, delivery, and loading facilities. Typically, reasonable convenience of parking areas (to the retail establishment) imposes a limit on the maximum acceptable distance between the most peripheral parking space and the center itself. This distance generally approximates  $400 \pm$  ft, the equivalent of a two-minute walk.

Beginning at the site perimeter, design considerations for retail activities should include the following:

**Center Orientation to Surrounding Road Network.** Retail establishments should be positioned to take advantage of visibility to abutting public rights of way. Leasing agents maintain the greatest market asset of a retail center is unobstructed visibility from adjacent roadways. In expansive centers, the major anchors are usually singled out for this elevated position, with smaller flanking or interior store locations dependent on foot traffic generated by the anchor. There are ample examples of successful retail centers that

do not rely on excessive road exposure. It should be cautioned, however, that many of these more introspective centers are located in planned communities, governed by site and architectural guidelines, and operate within an environment where the location of competing establishments is stringently controlled.

**Sizing.** The distinctions among freestanding, strip, center, and mall arrangements are related to the architectural massing and organization of retail space. With the exception of freestanding retail establishments, whose size and location may be a function of numerous unique circumstances, the size of the retail project and the profile of resident tenants relate to their intended service population. Normal conventions include a distinction among neighborhood centers, community centers, and regional centers. Although jurisdictions may provide precise definitions for each in their respective zoning documents, general definitions of each are as follows:

- *Neighborhood centers* generally service a population of 2,500–40,000 or a 1.5-mile radius. They may range from 3 to 10 acres in size, approximate 40,000–100,000 gross leasable square feet, and include a supermarket or drug store as the lead tenant. Other establishments focus on convenient consumer access to frequently purchased goods and services.
- *Community centers*, located on 10 to 40 acres, range from 100,000–400,000 gross leasable square feet and service a three- to five-mile radius and a population of 20,000–50,000, or the equivalent of several neighborhoods. The range in uses may include a supermarket, drug store, general merchandise, or similar offering, as well as attendant smaller retail establishments.
- *Regional centers* generally include retail assemblies sufficient to provide a full range of retail goods and services. They may range in size from 400,000 to over 1,000,000 ft<sup>2</sup>, service areas approximating an 8 to 10-mile radius, or 20 minute drive, and populations of 100,000+. They generally contain specialty offerings and full-scale department stores, as well as supporting establishments. Sites may range from 40 to over 100 acres.

**Vehicular Circulation.** Transportation circulation is extremely important to retail site design. Access and egress points should be well articulated and smoothly transition to the adjacent public road system. Ideally, multiple access points, sufficiently distanced to promote safe alternative approach routes, should be secured. Traffic routes should attempt to separate service vehicles from customer traffic. A separate site entry/egress point for service traffic is ideal but may be impractical in many instances. Customer vehicle routing should provide as near a continuous drop-off zone along the facade of retail establishments as possible. A perimeter circulation route, ring road, or street network to

access and exit the parking area can assist in reducing reliance on travelways nearer the building and minimize the potential conflict between through traffic and the discharging of passengers or loading of merchandise. In more conventional retail arrangements, depending on the depth of the parking bays and the size of the retail center, intermediate travelways midway between store facade and perimeter boundaries can assist high traffic volumes in navigating parking areas and assist overall traffic flow.

Service traffic should be directed to separate loading areas and service docks. Normally this occurs to the rear of the retail stores, where it can be appropriately screened and refuse containers can be shielded from public view. Loading bay approaches and service drive design should conform to appropriate geometrics for the size and number of vehicles necessary to serve the development program adequately. Entry drives should be of sufficient length for vehicles to enter the site adequately prior to contemplating major turn movements. Exits should provide sufficient stacking space to accommodate safe transitions into adjacent roadways.

**Parking.** The minimum number of parking spaces required to support retail activity is normally specified in the zoning ordinance, as is the required number of loading spaces. Criteria are usually expressed as a ratio of parking spaces to the square footage of the development program. The standards may vary depending on the service characteristics of the retail establishment. More spaces may be required for enterprises with higher customer occupancy, such as restaurants, than for stores with more extensive display and stock storage space, such as a furniture store. The required number of parking spaces is normally between four and six spaces per 1000 ft<sup>2</sup> of gross leasable area. The local zoning ordinance should be consulted for specific standards. Requirements for certain retail uses may require provision of additional spaces based on the number of employees or, in the case of restaurants and theaters, on seating capacity.

Although freestanding and smaller centers may accommodate parking stalls parallel to the pedestrian walkway immediately in front of the retail store, larger retail centers generally preclude parking from the vehicular drop-off and pedestrian approach zone. In these larger retail centers, the parking area normally begins beyond the frontage service drive and drop-off aisle. While the configuration of the parcel may warrant a different arrangement, parking aisles are generally positioned to run perpendicular to the store facade. This arrangement promotes ease and safety in pedestrian movements from their parked vehicles to the store entries.

Retail parking normally locates the major parking area between the retail establishment and the major frontage road. In larger multisided centers, such as shopping malls, parking generally encircles the center. In planned communities, where stringent design standards are in place, parking areas may be required to be buffered from public view or located to the rear of the retail area. Such an arrangement

minimizes the visible expanse of parking as well as promotes the retail building form in a forward position on the site to assist in defining the street edge. This latter approach is key to traditional design principles, which give heightened importance to pedestrian circulation, the use of building architecture, and massing to enhance street definition.

### Other Considerations

The sizing of retail use for a particular site and the establishment of the building floor plate should be done in consultation with both client and retail marketing professionals. While there are general rules of thumb associated with the provision of retail space, rapid changes in marketing practices and the unique circumstances associated with many individual retail establishments often prompt a tailored response that may vary from the norm. As a foundation, however, several general planning guidelines are appropriate for conceptual retail site planning.

- Single-story retail programs generally result in an effective floor area ratio of 0.18 to 0.25, depending on the configuration of the parcel and unique site conditions. Multilevel malls with structured parking can result in a floor area ratio exceeding 0.5.
- Conventional retail parking requirements of  $5 \pm$  per 1000 ft<sup>2</sup> of building area generally entail a parking area approximately 2 to 2.5 times the gross area of the building floor plate.
- Individual store depths generally range from 40 to 150 ft, while widths may approximate 20 to 150+ ft. Often the ability to partition individual tenant space is dependent on the method of construction and dimensions imposed by modular structural components. The clear span between such modular supports is often referred to as a building bay.
- Mall widths generally approximate 30 to 40 ft along pedestrian corridors. The distance may increase to 60 or more ft at intersecting corridors, or where feature courts or common areas are introduced along the pedestrian spines.
- In enclosed malls, 15 to 20% of the total gross leasable area may be included in common areas such as corridors, courts, and common mall area.
- Seven hundred ft is a reasonable walking distance between major retail anchors in an indoor mall. Four hundred ft is a maximum desirable distance from parking areas to retail entries.
- Large center retail layout is dependent on the number and positioning of anchor stores. Anchors are normally located at the building end or building intersections to provide each establishment with multiple facades and entrances for maximum building exposure and accessibility. In smaller single anchor centers,

the anchor should command a fairly central and visible location, with smaller stores flanking each side. This arrangement assists in promoting market identity, places the larger user central to the field of parking, and positions the smaller establishments to benefit from the pedestrian circulation generated by the larger store. In multiple-anchor malls or centers, the expanse between the major stores is most valued.

- Large retail centers may reasonably support a multilevel building program. In large retail centers, the internal vertical circulation within multilevel anchor stores augments core mall escalators and elevators in directing pedestrian movements to smaller stores on upper levels. Smaller neighborhood and community centers, dependent on ease of access and short duration of visit, seldom include second story retail space. If land costs or unique market circumstances warrant a second level, it is often occupied by office or service-related uses.
- In a large mall, a major anchor generally exceeds 100,000 ft<sup>2</sup> of gross leasable area. In neighborhood or community centers, the principal food market or drug store anchor tenant may range from 25,000 to 50,000 ft<sup>2</sup>.
- Design principles for power centers or big box users generally conform to those for comparably organized retail centers, be they standalone, strip, center, or mall arrangement. The size of the individual establishments, ranging from 40,000 to 100,000 ft<sup>2</sup> and up, however, requires additional attention to potential vehicular and pedestrian circulation conflicts and the size of the parking field.

### OFFICE USES: OVERVIEW

Suburban jurisdictions have experienced a tremendous amount of office development over the last several decades. The scale of this activity has evolved from tacit provision of small-scale office and administrative space serving locally based municipal, medical, and professional services to increasing accommodation of a major share of the employment base of metropolitan areas. The type, scale, location, and arrangement of these facilities vary considerably. They may house single or multiple tenants and may be occupant-owned or leased. Construction may be prompted by the specific program requirements of the occupant or be initiated on a speculative basis in response to existing or anticipated market demand.

In office development, zoning regulations most frequently focus on building bulk and site characteristics. Specific controls generally include criteria detailing acceptable limits for building site coverage, floor area ratios, building height, and impervious cover allocations. Typically, minimum standards include provisions for parking and loading, building setbacks and yard requirements, lot size, and open

space provisions. Additional criteria may refine the quality of the open space in specifying the extent to which it must be landscaped to qualify as landscaped open space or green space. These controls vary according to the intensity and character of existing development within the community. Controls may be less stringent in more densely developed areas surrounded by compatible land uses.

Four office types or configurations are commonly found in suburban locations: the freestanding office building, the office park, the garden office complex, and the townhouse office.

### **Freestanding**

Freestanding office buildings generally include a self-contained building on an individually recorded lot or parcel. The building may contain single or multiple users or tenants. Space may be rented or owned by the tenant or tenants. Its occupancy of a separate recorded parcel or lot and need to singularly satisfy appropriate zoning criteria is perhaps the only generalization that may be made of the free standing office structures. They can range from multi-floor multitenant structures with relatively high floor area ratios and heavy reliance on structured parking to small single-tenant buildings serviced entirely by surface parking.

The location and context of the surrounding land use factors heavily into the nature and execution of site design for office buildings. A more urban siting, with higher land costs, may confine the size of the building footprint and prompt a taller structure to secure the desired square footage. Such buildings may demand stronger street presence to satisfy pedestrian access, increased reliance on structured or below-grade parking, more intensive landscape development associated with modest residual open space, and greater restrictions on the number and location of access points.

Buildings in less urban locations benefit from lower land costs and may result in larger building floor plates, lower building height, and more expansive site improvements, including reliance on surface parking. Building siting may or may not demand a strong street presence. Institutional and corporate clients may prefer a more secluded setting in which the building is buffered from surrounding uses for purposes of privacy or security.

### **Office Park**

The office park requires coordination and control of a property assembly suitable for multiple office buildings and support services. Not unlike its residential counterpart, the planned community, the office park seeks to locate individual freestanding office structures in a setting enhanced by open space and landscape amenities. In addition to in-place jurisdictional controls, supplemental guidelines and covenants generally ensure that all development within the park will meet certain predetermined standards. The internal street system and infrastructure are normally planned and developed by a single entity. This results in the availability

of sites suitable for sale for individual building construction. The availability of appropriately zoned office acreage with in-place infrastructure is attractive to prospective owners or tenants who may not have the time, resources, or inclination to subject themselves to time-consuming front-end requirements necessary to transform raw land for office development. The proximity of similar and related uses coupled with the shared infrastructure and amenity packages, which may not be economical for any single user, are inducements for location within a well-planned and executed office park. Shared infrastructure elements may include both on- and off-site utility and roadway improvements as well as on-site amenities. Amenity provisions can range from extensive woodland preservation areas and water features to common recreation, athletic, and social facilities. Depending on the location and scale of the park, hotel, select retail, and day care facilities may also be included. Figure 12.24 presents an office park parcel plan.

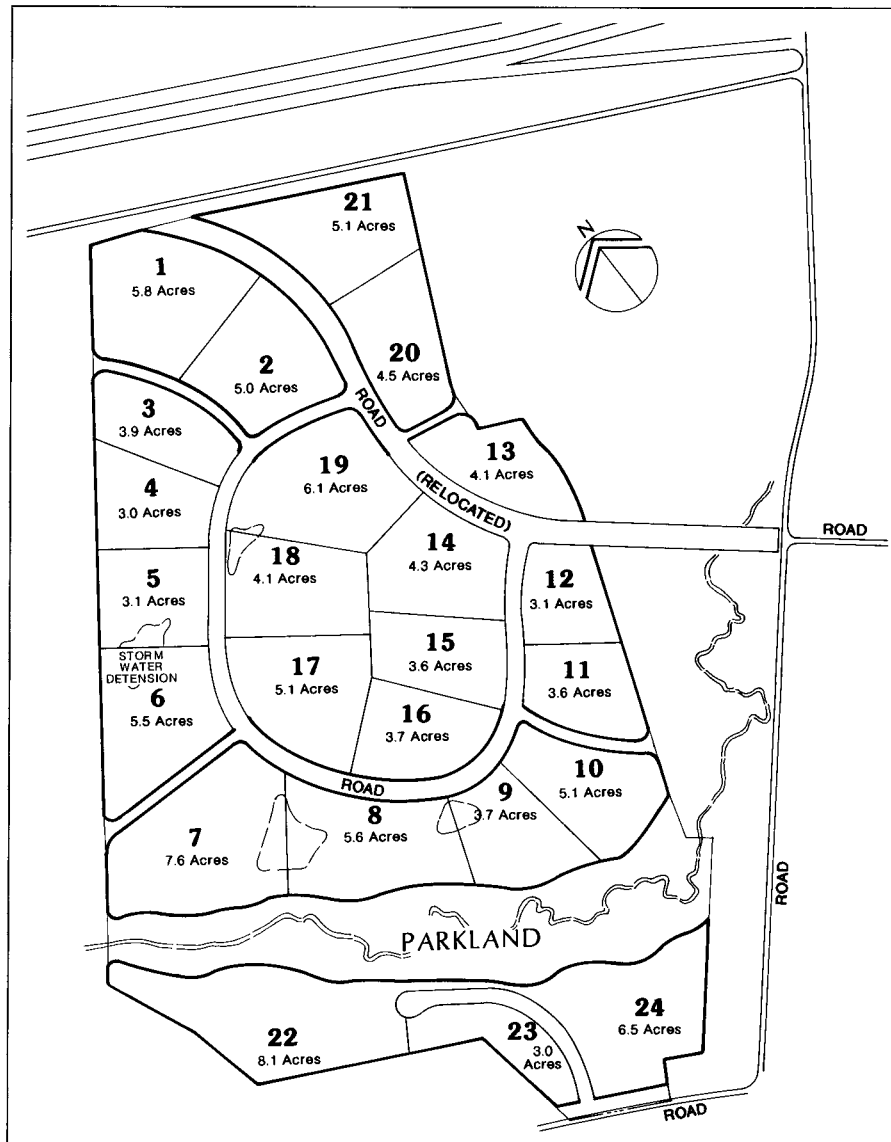
Site development in office park design, as with any complex multiuse building program, requires careful consideration of existing site features, criteria imposed by both local jurisdictional controls, prospective tenant requirements, and sound engineering and design practices. Site systems, including vehicular and pedestrian circulation, must be developed to balance the overall needs of the park with those of individual building sites. Parcelization plans require dimensional accuracy and flexibility in ensuring sales and marketing. The likelihood of surface, structured, or below-grade parking has bearing on the relative development intensity that may be anticipated on any single site within the park, as well as the aggregate development potential for the overall facility.

### **Garden Office**

The garden office generally provides for multiple tenancies in a low-rise assembly of buildings. It is particularly attractive to professionals and service-oriented businesses that benefit from proximity to surrounding residential areas. Its scale is generally conducive to locations near residential development, and the architectural style and scale may closely resemble garden-style multifamily residential construction. Ranging from one to three stories, the tenant space is normally divided horizontally by floor, with individual entry often open to common stairwells. Multilevel structures may be equipped with elevators. Buildings may be arranged linearly with common parking areas to the front, or clustered around parking areas or pedestrian courts. Where the terrain permits, entry may be gained at different levels of the building, providing the additional benefit and convenience of having associated parking at the same level as tenant space.

### **Townhouse Office**

Low-rise office space compartmentalized vertically may be classified as townhouse office. Similar to its residential counterpart, each unit is organized to access directly to the



**FIGURE 12.24** Example: office park parcel plan. (Photo by Larry Olsen)

exterior at grade level. The individual units may be single-story or multiple levels. While normally a single tenant occupies each unit, smaller businesses may occupy separate floors of the same unit.

Townhouse office, like garden office, benefits from a scale that is conducive to a location near residential areas. Its tenant profile focuses heavily on occupations and professional services that seek that proximity. Typical layouts vary from linear to cluster patterns, and topography may facilitate use of the underside of the unit to accommodate a separate tenancy.

### Office Design Principles

As with retail design, there are a few governing principles that are appropriate for office parcel layout and design. Foremost in this checklist are the specific requirements of the local jurisdiction. Zoning conditions may encompass building setbacks, buffering and screening requirements,

and minimum parking counts, as well as govern the height, bulk, and size of the building program. Municipal controls may reflect *pro forma* criteria attributable to the parcel zoning or tailored to the unique circumstances of a site relative to surrounding land uses and/or on-site characteristics.

Design of these elements requires attention to the details of site organization and function. Considerations generally include:

- Sensitivity in orchestrating an appropriate external image given potential roadway and land use adjacencies
- Site and building access that is legible and avoids congestion
- Efficient floor configurations that are conducive to leasing and occupancy requirements
- Sufficient parking given tenant requirements

- Parking that is convenient to both occupants and visitors
- Site development and amenity provisions that are tailored to meet the requirements of developer and tenant
- Service and loading design that avoids conflict with normal building operations and is adequately screened from occupant and community view

The range of land use intensity attributable to conventional office use can vary considerably. Floor area ratios in excess of 1.0 are not uncommon in proximity to suburban mixed-use centers. However, suburban office intensities for conventional freestanding buildings more commonly range from a 0.25 to a 0.4 FAR, which results in building heights of one to four stories. The 0.4 FAR is normally considered the threshold at which an office building may be surface parked while maintaining reasonable distances between the building and outer perimeter parking areas. An FAR greater than 0.4 normally requires significant reliance on subsurface or parking structures.

Although traditional parking ratios for office use averages four spaces per 1000 ft<sup>2</sup> of net leasable space, the requirement may be lessened at locations close to public transportation or mass transit. In some circumstances, parking demands may be lessened if a disproportionate amount of building space is to be utilized for operations that are not employee-dependent. On the other extreme, tenancy that requires high visitor turnover, such as medical office uses, may necessitate higher parking ratios. Similarly, anticipated building occupancy by tenants employing smaller per employee work areas, such as telecommunication call centers or 24-hour operations with overlapping work shifts, may require parking that exceeds conventional office criteria.

## INDUSTRIAL

Local development standards governing industrial activity have often augmented traditional zoning criteria that establish controls on maximum land use intensity, minimum lot size, setbacks, and building bulk and height, with specific performance standards that serve to categorize not only permitted types of industrial activities, but minimum standards associated with their operation. Performance criteria may be tied to standards governing air pollution control; fire and explosion; radiation hazard; electromagnetic radiation and interference; liquid and solid waste discharge; and noise, vibrations, and light source considerations, among others.

The nature, extent, and complexion of industrial land use has changed dramatically in the last several decades. The expansive single-purpose facilities focusing on raw mineral extraction, product manufacturing, and assembly have been overshadowed by the provision of more flexible space that focuses on product distribution, research, and service industries. This transition has had a profound impact on the location requirements of industrial uses and the

physical characteristics of industrial facilities. It has also lessened community resistance to a genre of uses that do not pose the level of incompatibility historically associated with industrial activity, including potential air pollutants, hazardous waste byproducts, acoustical and vibration impacts, and heightened fire and safety considerations.

While heavy industrial, extractive mineral, and expansive processing facilities can be found in suburban locations, escalating land values combined with new technologies have often prompted many of these types of industries to relocate to areas where land cost and labor supply and use controls are more conducive to their operations. In their stead, activities focusing on research and development, smaller-scale electronic assembly, data processing, telecommunications, product distribution, and warehousing have become more significant industrial components of many metropolitan areas. The noxious characteristics of traditional industrial activity located proximate to railroad lines have given way to expansive industrial and technology parks whose locations are governed more by convenient access to interstate highways, airports, and population centers.

Building programs associated with industrial activity generally rely on a strong building-to-ground relationship. This is in keeping with their dependence on truck transport as a principal means of product delivery and assembly operations, which rely on horizontal line arrangements and on-grade structural conditions to support machinery and expansive storage and warehousing requirements.

Sensitivity to the diversity in industrial uses of both historic and contemporary facility types has prompted most jurisdictions to adapt their zoning ordinance to provide varying standards for select classes of industrial activity. Classifications normally differentiated among light, medium, and heavy industrial uses. The gradation was strongly related to the potential level of land use incompatibility anticipated between the subject industrial operations and adjacent land uses. In recent years, finer distinctions have been made to accommodate the less onerous industrial characteristics associated with research and development (R&D) activities. In R&D parks and similar institutional environments promoting scientific research and product development, training and offices manufacturing or product assembly may be either nonexistent or only a diminutive part of the overall facility operation. The potential land use conflicts and levels of incompatibility characteristic of traditional industrial activity may be negligible.

Building programs that reflect some of the more typical industrial arrangements found in the suburban market are illustrated below.

### Heavy Industry

Precedent activity, proximity to raw materials, natural resources, power sources, and location at major transportation hubs and population centers are several factors that prompt the need for heavy industrial activity in otherwise urban and suburban markets. Generally, these activities fos-

ter a litany of potential use conflicts. Zoning criteria normally focus on minimum dimensional standards and performance criteria to reduce or contain the impacts of such uses. Facilities may include intense manufacturing operations; heavy equipment, construction, and fuel yards; mining and quarrying; major transportation terminals; and other equivalent concentrations of potential noxious uses. Heavy reliance on truck transport favors industrial locations with reasonably convenient access to major arterial or collector roads or, in the case of interstate trade, major freeways and highways. Reliance on rail and water access continues to be important to certain types of industrial activity.

Heavy industrial uses tend to be land-consumptive in terms of building program or exterior space requirements for vehicular movement and storage. Floor area ratios of 0.5 to 1.0 with surface improvements as high as 80 to 90% of the usable site area are common. Site design issues focus on adequacy of site entry and egress; internal site circulation with regard to potential separation of visitor, employee, service and facility operating needs; and screening of exterior storage and loading areas from external site views.

### Medium Intensity and General Industrial Use

Product fabrication and assembly operations, storage, and similar industrial operations in suburban locations tend to be categorized as light manufacturing. They offer reduced levels of noise, smoke, glare, and other environmental pollutants normally associated with the more intense or heavy industrial activity. Development considerations and site design issues do not differ substantially from those previously cited. Characteristics associated with a smaller scale of operation or activity that has a relatively less invasive impact on surrounding land uses often accommodate mitigation of concerns within the confines of the site

### Light Industrial

The light industrial classification affords more lenient standards, given the more compatible nature of permitted uses. Uses include activities considered least obtrusive and operating under high performance standards relative to off-site impacts. Floor area ratios may be less expansive, ranging from 0.4 to 0.5, and operations including storage may be required to be entirely under roof.

### Special Industrial Use Patterns

Contemporary development patterns, evolution in building technology, and opportunities or requirements for the containment of noxious impacts have fostered a radical alteration in industrial prototypes over the last several decades. This has resulted in refinements in standard land use controls, which are sensitive to trends in the aggregation of

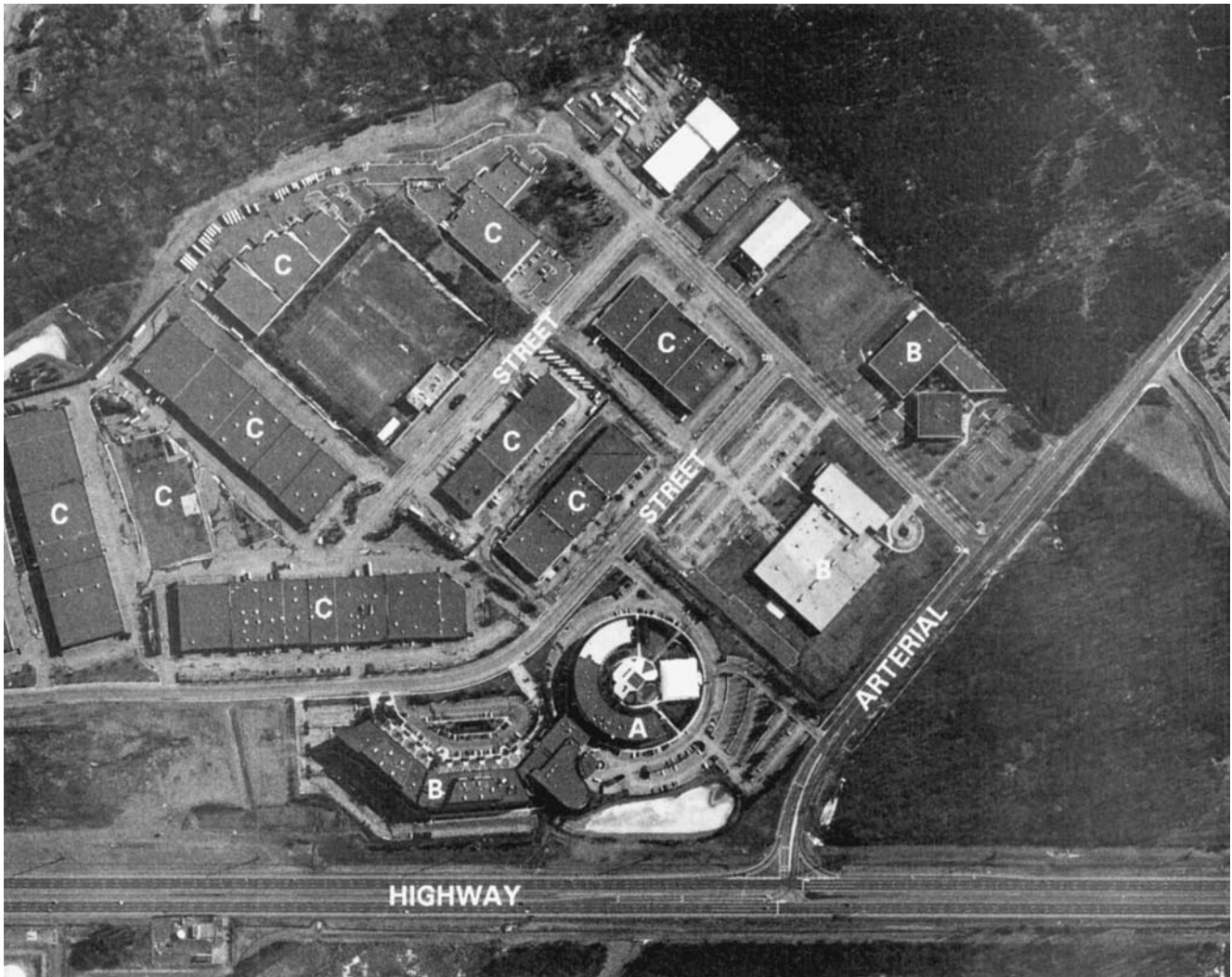
compatible industrial activities in planned industrial parks. The emergence of zoned industrial districts qualified as institutional, research, technology, warehouse, and distribution centers reflects unique sets of operating requirements, an affinity for like-kind uses to collocate, and market-driven pressure to elevate the site aesthetics of select industrial land use activities.

Local public controls may afford opportunities for the creation of an industrial district that caters to uses that can demonstrate compatibility with surrounding land activities. Minimum land assembly, larger minimum lot requirements, increased setback, screening and buffer requirements, increased landscape and open space standards, reduction in allowable building profiles, more restrictive building bulk criteria, and allowable land use intensities as low as 0.25 FAR assist in ensuring reduced visual impact on surrounding areas for uses that can demonstrate no adverse off-site impacts. Positioning more compatible activities at perimeter locations often assists in creating an overall planned industrial development that affords considerable variety in tenancy and use opportunities. A typical industrial park is presented in Figure 12.25.

### Industrial/Office/Retail Hybrid Building Forms

One of the more notable and popular building forms that has emerged in suburban development markets is the flex-tech or office showroom prototype. The overall building shell is designed to allow the compartmentalization of separate units, depending on the spatial needs of individual tenants. Generally, unit or bay sizes are tied to the modular dimensions of the building structure (the roof span between vertical support elements). Common modules range from 20 to 45 ft. As in construction techniques employed in conventional retail centers, the overall length of the building may be divided by interior walls, providing each occupant an exterior exposure on both the front and service side of the building. The front exposure is generally devoted to pedestrian access and proximate parking. Limited retail sales, showroom, or administrative offices may be located at the front of the tenant bay. The utility or service side of the bay is accessed from a vehicular service drive. Service drive pavement dimensions are sized relative to the type of service vehicle required for the targeted tenant group. While smaller delivery trucks can be accommodated in as little as 45 ft, docking and circulation widths generally range from 85 ft to 130 ft for larger vehicles. Individual or shared loading docks or service doors generally extend along the service facade proximate to interior bay storage, fabrication, or assembly operations.

While buildings are normally single-story on slab construction, full or partial second stories (mezzanine levels) may be incorporated into the building. Upper levels are commonly devoted to administrative uses and often are limited to the front module of the building, leaving the areas above the utility portion or rear of the building with a



**FIGURE 12.25** Example of industrial park; Renaissance Park, Fairfax, Virginia, is an example of a hybrid industrial park with (a) hotel located on a major highway frontage, (b) traditional office/research and development building types with separate parking areas and limited loading facilities at more visible locations, and (c) flex industrial building types with visitor/employee parking along the building frontage and a continuous service/loading area running the rear building length. (Courtesy of Air Survey Corp.)

higher ground-to-ceiling height, normally 14 to 20 ft. The type and flexible size of tenant space, up-scale facade treatments, and overall site finish associated with facilities of this type are attractive to a diverse range of tenants, which may include such uses as bulk warehouse, industrial/office, light manufacturing, office/showroom, research and development, and retail service centers.

Lot coverage and floor area ratios vary according to local regulations and size and bulk requirements of principal user groups. Generally, larger users command higher coverage and FAR capacity owing to the magnitude of desired floor plate and associated reduction in employee and customer parking needs. A reasonable range in smaller flex-tech and office showroom combinations may be between 0.24 and 0.35 FAR. Figures 12.26 and Figure 12.27, respectively, il-

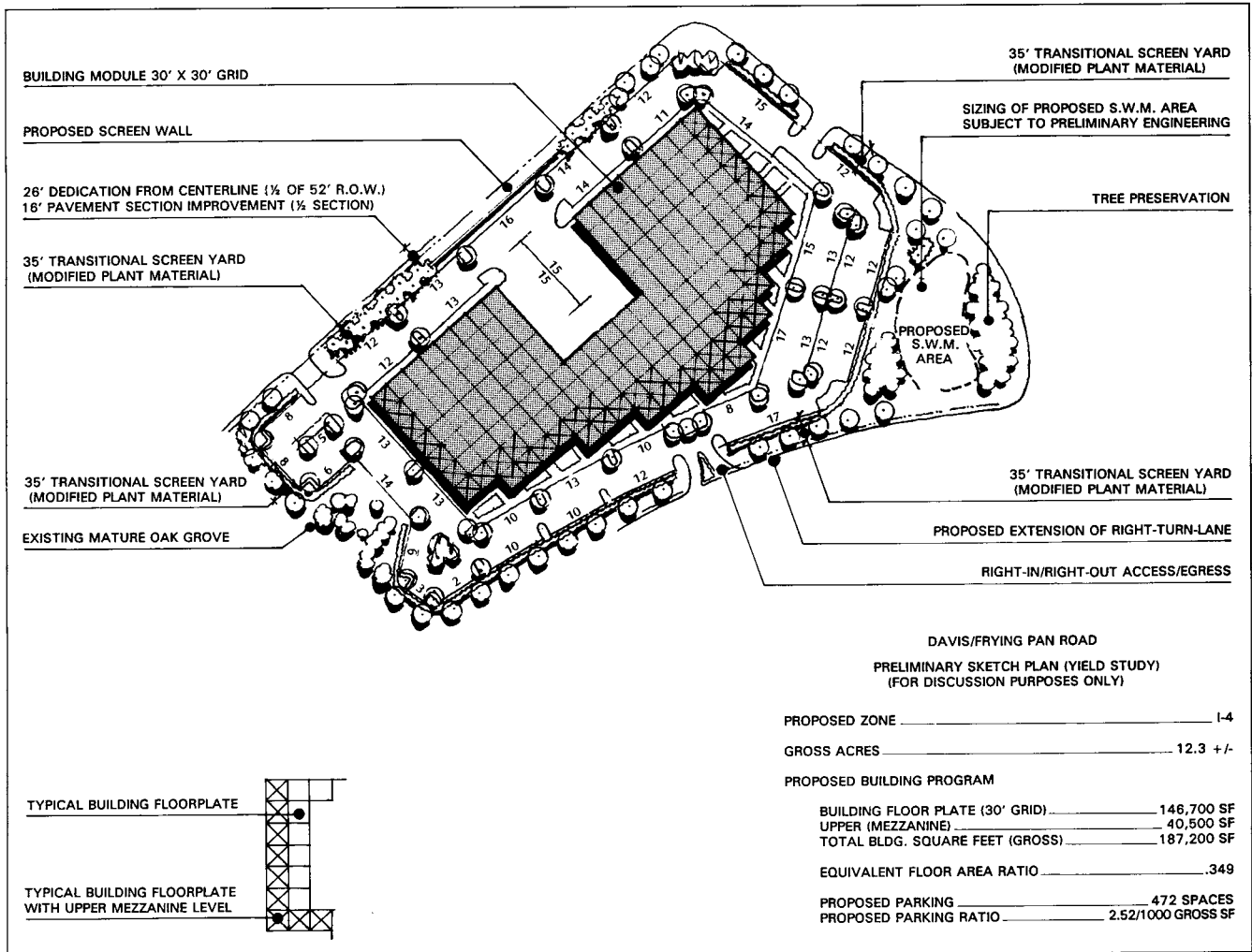
lustrate the flex-tech building and portion of a flex-tech park.

## CONCLUSION

Response to contemporary development trends, particularly in the suburban context, has fostered much introspection in regard to both the quality of life and the environment. While the development of land will in all likelihood not abate, greater integration of uses internal to individual developments or within the larger community context is increasingly evident in our built environment.

New market opportunities and community planning initiatives that seek to better direct and control the location, content, and quantity of development have fostered this attitude.





**FIGURE 12.26** Example of flex-tech building.

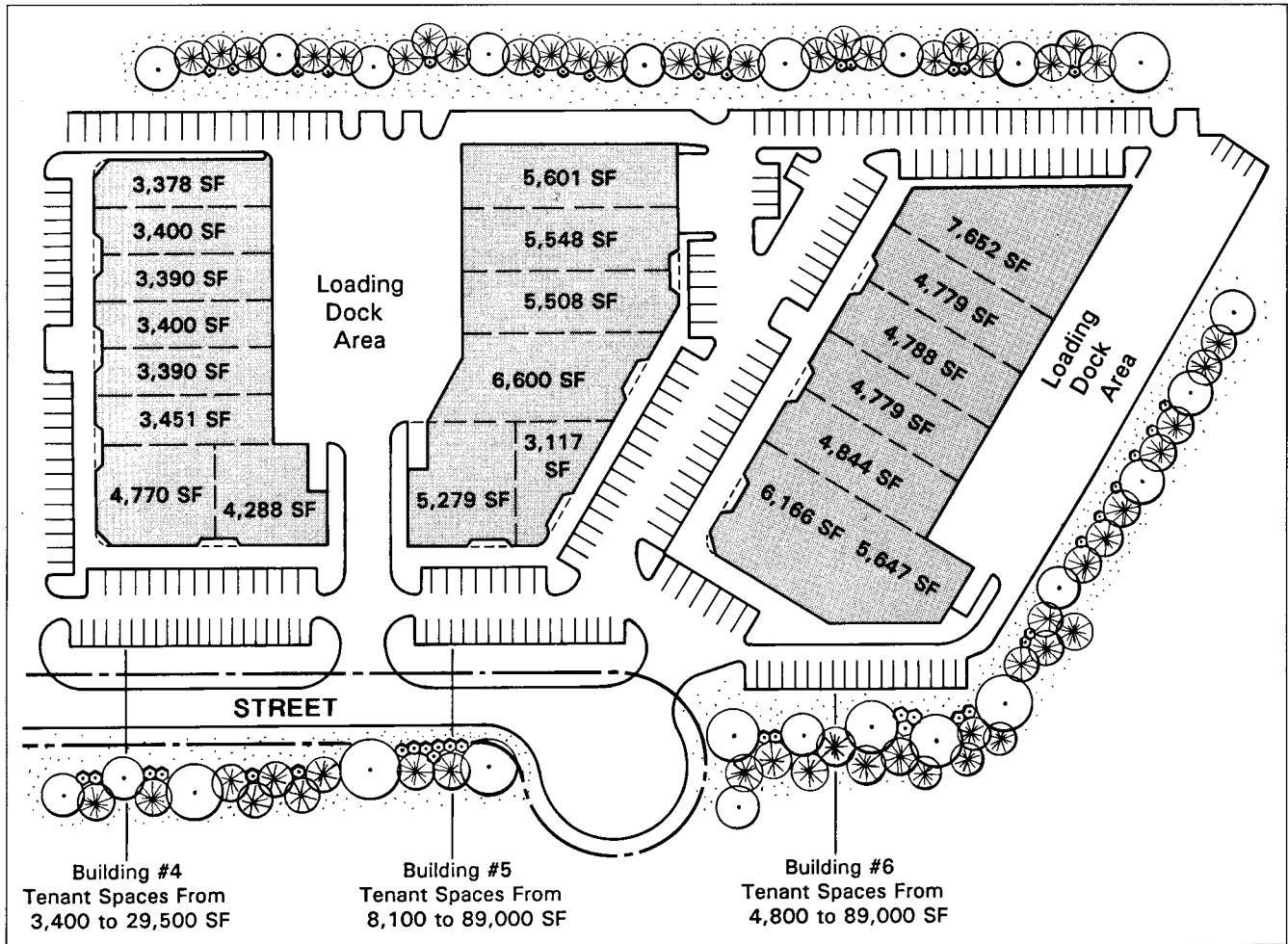
Numerous factors have occasioned a renewed look at development market opportunities and in turn development composition and project content. Increasing land costs have promoted an increase in development densities. The level of vehicular congestion has prompted more deliberate evaluation of development opportunities proximate to mass transit facilities or in closer proximity to supportive land uses, which might minimize reliance on the automobile or shorten travel time. Increased popularity in a greater variety of leisure activities, coupled with shifting demographics toward smaller families, empty nesters, and single-occupant households, prompts evaluation of requisite yard requirements, the venue of project amenity provisions, and proximity to existing and proposed community institutional and cultural offerings.

Public initiatives, in attempting to come to terms with the public infrastructure required to support future development activity, have similarly focused on efforts to mitigate

the current dependence on the automobile as the principal mode of transportation. Efforts to promote mass transit, concentrate development density near existing and planned transportation corridors, and integrate a variety of residential, retail, and employment uses with greater reliance on pedestrian travel all attest to the prospect for less insular, more integrated land development patterns.

Emerging trends on the content and context of development patterns and principles include:

- A greater focus on the integration of uses, as might be exhibited in a reduction in distances between development types, or vertical integration, as with first floor retail and upper level office or residential uses
- A finer grain in mixing uses, from the standpoint of an increased variety of residential types given the market diversity of residents, and greater integration with nearby retail and employment activities



**FIGURE 12.27** Example of flex-tech park.

- A renewed focus on the connectivity within and between developments in ensuring adequate and alternative means of vehicular movement as well as greater attention to pedestrian opportunities
- A reassessment of the location of open space with recognition of its role in place making and elevating it beyond the unusable or leftover ground within a development
- Increased attention to building placement, massing, and scale as tools in defining space, directing views, and encouraging diversity in the pedestrian experience
- A reassessment of the nature, magnitude, and placement of parking in terms of (1) potential for joint use and parking reductions owing to use adjacencies, (2) increased reliance on integral parking or parking structures in response to increased land cost, and (3) potential relegation of parking from forefront to rear yard as buildings move street-side

- Continued evaluation of the hierarchy and configuration of street networks, recognizing the need for safe and efficient vehicular movements as well as enhanced pedestrian opportunity and experience

#### ADDITIONAL READINGS

- De Chiara, Joseph, and Lee E. Koppelman, *Site Planning Standards*, McGraw-Hill, New York, 1978.
- Engstrom, Robert, and Marc Putman, *Planning and Design of Townhomes and Condominiums*, Urban Land Institute, Washington, D.C., 1979.
- Jensen, David R./HOH Associates, *Zero Lot Line Housing*, Urban Land Institute, Washington, D.C., 1981.
- Lynch, Kevin, and Gary Hack, *Site Planning*, MIT Press, Cambridge, Mass., 1984.
- Maitland, Barry, *The New Architecture of the Retail Mall*, Van Nostrand Reinhold, New York, 1990.
- National Association of Home Builders (NAHB), *Cost Effective Site Planning: Single Family Development*, NAHB, Washington, D.C., 1982.

National Association of Home Builders (NAHB), *Land Development 2*, NAHB, Washington, D.C., 1981.

Rathbun, Robert Davis, ed., *Shopping Centers and Malls 3*, Retail Reporting Corporation, New York, 1990.

Tomioka, Seishiro, and Ellen Miller Tomioka, *Planned Unit Developments: Design and Regional Impact*, John Wiley & Sons, New York, 1984.

Untermann, Richard, and Robert Small, *Site Planning for Cluster Housing*, Van Nostrand Reinhold, New York, 1977.

Urban Land Institute (ULI), *The Community Builders Handbook*, ULI, Washington, D.C., 1968.

Urban Land Institute (ULI), *Industrial Development Handbook*, ULI, Washington, D.C., 1975.

Urban Land Institute (ULI), *Office Development Handbook*, ULI, Washington, D.C., 1982.

Urban Land Institute (ULI), *Project Infrastructure Development Handbook*, ULI, Washington, D.C., 1989.

Urban Land Institute (ULI), *Residential Development Handbook*, ULI, Washington, D.C., 1990.

Urban Land Institute (ULI), *Shopping Center Development Handbook*, ULI, Washington, D.C., 1977.

## PART IV

# SCHEMATIC DESIGN

Dennis Couture, A.S.L.A., R.L.A.

### INTRODUCTION

Schematic design focuses on the development and refinement of conceptual design and results in an interim blueprint for subsequent detailed design efforts. During this phase of study, a selected conceptual development plan is subjected to more detailed analysis in terms of usable area and potential yields. Activity may include a formal presentation to public bodies and regulatory agencies to obtain concurrence with the site design strategy prior to detailed site design activity. This phase represents a major benchmark in the design process in its testing and confirmation of development intent. In view of the increasing complexity of the public review process and the costs associated with more detailed design, public review at this phase of the land development design process is extremely important. It is a sound business practice to keep public decision-makers abreast of development proposal whenever practical. If there are to be political or procedural obstacles to the plan, it is best for them to surface early in the design process. Jurisdictions increasingly require various types of interim submissions prior to the completion of final development site plan documents, and schematic level design instruments are not uncommon requirements for public sector review and approvals.

### STEP 4: SCHEMATIC DESIGN

The schematic or master development plan is often the design document that begins to establish many of the controlling standards and design character for actual project level development. Extensive studies are normally completed to ensure that the proposed development program is realistic in terms of development density, product yield,

functional prerequisites, economic, and political feasibility. Furthermore, the relative placement of land uses and infrastructure considerations begins to prescribe the ultimate project character. The additional level of specificity provided in the schematic level of study may include:

- *Traffic and transportation:* Program use distribution and projected yields can be modeled to provide a check on the adequacy of existing and proposed road system.
- *Utilities:* Preliminary sizing and routing of utilities can be located based on the distribution and density of program uses.
- *Stormwater Management:* Based on the impervious cover projected for specific land uses components, preliminary sizing and location of stormwater management areas can be determined.
- *Environmental specialist:* The more detailed schematic plan allows for an assessment of possible impacts on previously identified sensitive areas.

This refined level of information offered by other development team members affords the designer the opportunity to perfect the schematic plan in terms of the allocation of areas suitable to accommodate infrastructure requirements of the prescribed program as well as to resolve any conflicts which have been identified during this review and assessment activity.

The end product of the schematic design phase is a scaled representation of the development program and circulation framework (see Figure IV.1). Major elements normally include:

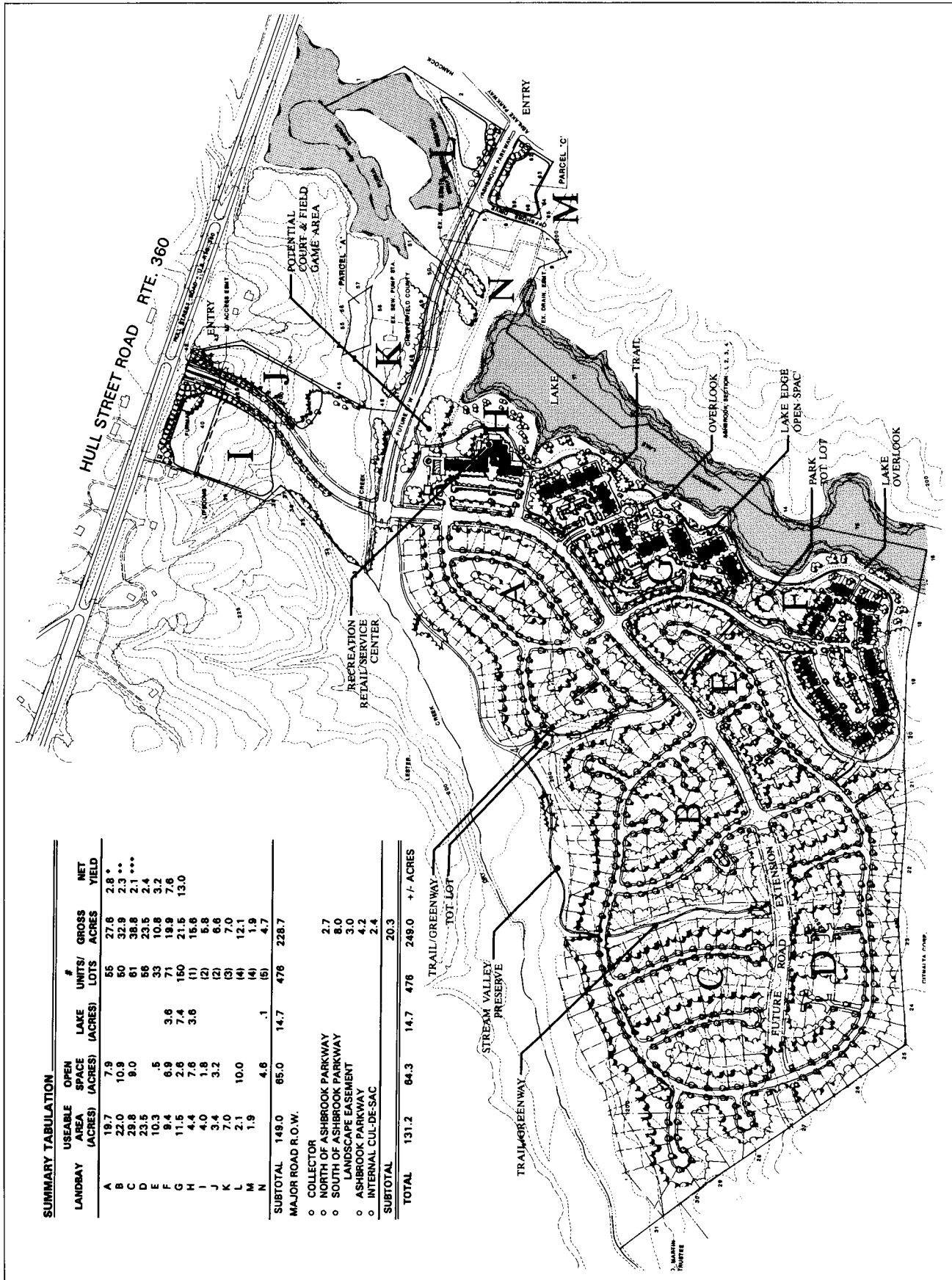


FIGURE IV.1 Example of schematic development plan

- Major vehicular circulation systems depicted in conformance with the design standards of the local jurisdiction and current public comprehensive or master planning considerations.
- The allocation of land uses annotated by use and area. Area is normally noted as the gross acreage of the individual land units commonly referred to as “land-bays.”
- Identification of areas subject to development constraints, including floodplains, wetlands and environmentally sensitive areas, buildings or grounds considered historically significant, and other acreage that by virtue of unique circumstance has been mandated by public policy to be subject to development limitations.
- Appropriate delineation of areas for public facilities, including schools, public safety, libraries, parks and recreation facilities, day care centers, church sites and similar facilities that have been determined necessary or desirable support components to the initial development program.



# PART IV.A BASE MAP PREPARATION

## CHAPTER 13

# BOUNDARY SURVEYS FOR LAND DEVELOPMENT

Gary Kent, L.S.

### INTRODUCTION

#### Property Surveying

Land surveying is the art and science of measuring distances and running lines on the surface of the earth. One of the purposes of land surveying is to determine the boundaries and ascertain the areas of tracts of land. Fixing the position of lines, monuments, and boundaries requires that the surveyors use a variety of instruments and correctly apply a variety of principles of law.

Reproducing previously established boundary lines and corners in their intended legal and physical positions is a matter of determining the intent of the original survey or other written documents used to first establish them. This seems trivial at a glance, but upon examination, such a task requires the experience, perseverance, and reasoning of an expert detective. Research is the key for a successful boundary retracement. That research includes gathering evidence, interpreting that evidence in the light of conditions at the time of the creation of the boundary, and finally recreating the location of the boundary on the ground as it exists today. Consideration of written evidence is of utmost importance, but physical evidence and oral testimony related to interpreting the written evidence are crucial to complete research.

Research for surveys prior to land development projects begins with gathering property descriptions, tax maps, roadway plans, zoning maps, easements and other related information. Gathering of this information begins when the client and the engineering or surveying firm enter negotiations for the future development of a tract of land. The prospective developer is the first source of information in a process of research necessary for such a survey. Often he

or she will have a copy of the deed or deeds and possibly other information related to the property. At a minimum, the developer will have knowledge concerning the present owner.

Important depositories of research information include local engineering and surveying firms. Local utility companies often can provide valuable maps and information. State and federal agencies maintain valuable land information. Realtors often maintain files on land ownership. Title insurance companies play a critical role in providing deed, easement, and other record title information and often previous surveys of the subject property or adjoining property.

Existing maps or plats of the land provide visual support for reaching conclusions that ultimately result in recreating boundary locations. Maps may be found during the search of land records and from the evidence found during the field survey. Plotting individual deeds, patents, grants, and tax records provides pieces of a jigsaw puzzle. When the pieces of the puzzle fit together and form a logical composite map, this often provides the necessary clues to interpreting land ownership and boundary location successfully.

The two primary systems that form the basis of land descriptions in the United States are the metes and bounds system and the public lands survey system. These systems have an impact on interpretation of evidence gathered in the research process.

Where surveys are for land development additional information needed for such purpose is obtained while survey crews are in the field. This includes topographical information to be used in determination of grades, street design, and drainage and for constructing structures and construct-



ing or extending sewer and water lines. These types of surveys are discussed in a subsequent chapter.

### Functions of the Property Surveyor

Determining the location of boundary lines receives high priority on the list of steps in planning and implementation of land developments. The professional surveyor thus becomes a major part of the team of professionals necessary for carrying land development projects to successful completion. His or her knowledge and skills must be broad in scope yet particularly specialized in knowledge of surveying.

The surveying professional must be versed in real property law. In resolving boundary locations, the surveyor is required to interpret and apply legal principles. The surveyor must have knowledge of engineering to provide, through the survey, the proper information for designing the utilities such as water and sewer systems. He will work closely with land planners to supply their needs for the planning process. In short, he must understand the entire land development planning process.

For many development projects that lie away from urban areas, the surveyor will fill the role of many professionals, much as a doctor who is a general practitioner fills different roles. Most often, land development takes place in the urban setting, but this situation does not completely relieve the surveyor of assigned general responsibilities in the project. In this situation, the surveyor's greater role, however, becomes that of specialist. The tools are those afforded by the latest in technology. Computers, electronic measuring devices, global positioning systems, aerial photogrammetry, and total station equipment have all but replaced the slide rule, transit, and steel tape of but a few years ago.

Mathematics, statistics, and electronics are becoming increasingly important to the professional surveyor today. The surveyor must keep in touch with his past because the very act of boundary surveying requires that he follow in the footsteps of his predecessors. To accomplish this he must, in essence, wear the shoes they wore. The Jacob staff, compass, perches, arpents, varas, poles, rods, and Gunter's chains must remain a part of his working vocabulary.

The surveyor must use his knowledge and training to perform the duties associated with the profession. These include duties to the public and duties to the client. The first category of duties to the public includes maintaining his qualifications and abilities through training, as well as limiting his practice to those areas of expertise where he is competent.

It is the surveyor's duty to avoid creating or propagating boundary disputes if possible. Often, through discussions with adjoining owners and the client that he represents, the surveyor can arbitrate disputes and bring parties to equitable solutions while avoiding costly legal actions. Of course, the surveyor should be cognizant that professionals with legal training (i.e., licensed attorneys) should be consulted on questions of law.

The surveyor must keep his duties to the public and to his client in perspective. Most important in this regard is the surveyor's advice to the client. The client should be made aware of the function and limitations of the surveyor. This discussion should include the process of research of documents related to the survey. The client should know that the surveyor must consider and inquire into all the facts of both record and existing in the field. The surveyor should advise the client that neither the opinion nor the survey of a surveyor is conclusive in the determination of boundaries. The client should be advised that it is the surveyor's duty to gather facts and interpret them based on the rules of evidence, but that the final disposition as to boundary location and property ownership lies with the courts.

Before proceeding with the project, it is advisable to reach a clear understanding with the client on the terms under which the work will progress. A proposal should be prepared for each project before beginning work. It should include a detailed scope of all services that are to be performed and should provide for means of payment.

When the client understands the role of the surveyor in the development process and agreement is reached concerning a contract to proceed, the actual research begins.

The date of the last transfer of the land must be determined. Often the client can furnish the names of adjoining landowners. All information obtained at this stage will reduce the time spent in searching records later. This information will form the basis on which future research and analysis of land ownership will rest.

If available, obtain an abstract of the property that is to be included in the proposed land development project. The abstract summarizes the record history of the documents affecting the title or land ownership. The abstract includes, in chronological order, all recorded grants, deeds, easements, wills, and other information such as deeds of trust, tax liens, assessments, and any other encumbrances of record affecting the property.

Likelihood of development and the additional survey requirements typically associated with development projects greatly increase the detail that must be obtained during the boundary survey. The surveyor should point out to the client that the necessity for greater detail, precision, and accuracy may directly increase survey costs. Because of the greater liability brought on by land development surveys, the surveyor should realize the importance of additional care and research that is more thorough.

### Legal Descriptions

The part of the deed devoted to the physical location of the real property is known as the legal description. A general definition of a legal description can be stated as those words and maps or plats that uniquely delineate the tract from any other. The description must be written in such a manner that it will stand any test under law and litigation.

Such descriptions can be formed in various ways. Words describing lines composed of bearings and distances and

calling for monuments of one type or the other are “metes and bounds” descriptions. Some descriptions refer to tracts that are recorded in the public records. Other types of descriptions are used for particular situations such as strip descriptions for rights of way for roads, power lines, and other utilities. The thread common to all of them is that each describes a specific parcel of land that cannot be applied to another parcel. The surveyor should write a legal description that is as brief as possible while maintaining clarity, completeness, and accuracy. Incomplete, inaccurate, and unclear descriptions will result in title disputes later.

The description is composed of three parts: the caption, the body, and the qualifications such as exceptions and reservations. In some cases, augmenting clauses are added. Each part of the description serves a definite purpose.

The caption establishes the general placement of the subject property and limits title in the remainder of the description to that general area outlined in the caption. A typical caption in a description could read as follows: “that certain tract or parcel of land, situated at or near City, District, County, State and being more particularly described as follows:”.

The body of the description generally follows the caption. A typical body in a colonial state deed description might read as follows: “Beginning at a set stone at a corner of rail fence on the old baggage road, and running with the said road to the Noah D. Johnson line, and with said line to the John C. Williams line, thence with the Williams line back to a set stone corner on said John C. Williams line, thence a straight line to the beginning, containing three (3) acres, more or less.” Describing parcels within land subdivisions employs the use of reference to a subdivision map of record. Such a description follows:

Being situated at or near City, County, State, and being more particularly bounded and described as follows, to-wit:

Lot No. 98 of the Forest Hills Subdivision, as the same is shown and designated on a map or plat thereof which is of record in the office of the Registrar of Deeds of (County), (State) in Map Book 8, at page 67.

The third part of the description, the qualifying clauses, does not follow any particular arrangement. These clauses may except some part of the conveyance or reserve some part generally in the grantor. Qualifying clauses often follow the body of the description. Such a clause might read, “There is excepted and reserved from this conveyance a 10 foot lane, which lies on the western boundary of the property herein described, which serves as access and egress to Donald Thompson and this reservation is for the benefit of the owners of the property presently owned by Donald Thompson and to the successors in title to Donald Thompson.”

For a complete discussion on writing descriptions as well as a study of the various types of descriptions encountered by surveyors, see Wattles (1976), a classic on this subject.

## SURVEYS FOR BOUNDARY LOCATION

The land description contains the written instructions that demonstrate the original intent of the conveyance. Except in explicit situations, any retracement of the property must be based exclusively on the record description.

When the legal description of the property is prepared from a survey preceding that description, that survey is called the *original survey*. Any surveys made after the writing of the description are called retracement surveys. If the description of the site was prepared without benefit of a survey, then there can never be an original survey. The first survey made after a description is written without benefit of survey is called a *first survey*.

Original surveys should not be confused with First surveys. Although all original surveys are first surveys, a first survey is not necessarily an original survey. First surveys are not given the same standing in law as original surveys. This is because the intent of the parties to the transaction creating the description comes from the document containing the description. A first survey is only an *interpretation* of the original document. When original surveys exist, the intent of the parties is presumed to have come to the description through the survey. Few rules in real property law are more certain than “Original called-for monuments control over course and distance.”

The surveyor’s responsibility when making a resurvey is to recover or determine the location of the original monuments (if there were any) and, if there were none, to interpret the intent of the record description based on the best available evidence, then establish the lines and corners of the tract accordingly.

A boundary survey does not end with the establishment of the lines and corners of the surveyed parcel. The surveyor must also relate the surveyed lines to occupation and possession lines.

## Encroachments and Gaps

Visible evidence of potential encroachments of any kind must be located and the history of the encroachment documented if possible. An encroachment may be defined as a trespass or the commencement of a gradual taking of possession or rights of another. Encroachments may take the form of walls, fences, buildings, or other structures representing occupation or possession that extend onto another’s title rights. Encroachments may lead to claims of unwritten rights such as adverse possession or acquiescence when the party whose land is encroached upon allows the encroachment to go unchallenged. Such an implied consent can cause loss of title.

Often land contemplated for development is composed of several smaller tracts that are joined together to form a larger, more developable tract. In such a situation, it is important for the surveyor to confirm the contiguity of the parcels involved or identify the existence of any gaps between the parcels. This is critical because the record title to

such a gap may lie in the hands of some former owner rather than in the hands of the owner of the parcels being surveyed. Gaps can be as troublesome as encroachments if not identified and quantified before development actually begins.

### Importance of the Survey in Land Development

In the land development process, the determination of tract location and geometry is of utmost importance. Surveys resolve whether the written description and the physical possession on the ground agree. All future work related to development hinges on the initial survey work. Should an error occur in the survey, it will most likely manifest itself in later stages of the development process. Errors in the initial survey could conceivably lead to improper location of streets and other improvements and to incorrectly placed interior boundary lines. All errors such as these could bring on project delays, possible project shutdown, and redesign, which will inevitably cost both time and money.

Errors undermine the client's confidence and can cause the loss of valuable clients. The results of poor judgment associated with boundary line location cannot be overstated.

Definite procedures, including such quality control tools as check-off lists, are invaluable in helping to ensure consistency. No procedure or checklist, however, can substitute for common sense, intimate knowledge of survey law, and diligent fieldwork.<sup>1</sup> Both the client and the surveyor must recognize that the results of any boundary survey can be challenged, as it is the province of the judiciary ultimately to determine boundary location. The surveyor supplies much of the evidence upon which the court decision will be based.

When the development involves one or more loans, the lender typically will require certain assurances to be confident that its investment is as risk-free as possible. One of the tools to minimize risk is the ALTA/ACSM Land Title Survey. This type of survey is defined by a strict set of standards adopted and periodically revised by the American Land Title Association and the American Congress on Surveying and Mapping. The current version, as of the date of this publication, is the 1999 standard (see Technical Appendix).

The Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys outline the responsibilities of the surveyor in conducting a survey that will be used by the title company and lender in conjunction with the closing of a commercial loan on real property. The standards also outline the responsibility of the client or her representative with respect to providing complete sets of documents from the title research and as may otherwise be required by the

surveyor to conduct a proper and complete survey. In addition, there are optional check-off items in Table A that allow the client or lender to request of the surveyor additional information related to certain specific issues such as availability of utilities.

The ALTA/ACSM Minimum Standards call for the surveyor to resolve the boundary of the property in addition to providing comprehensive documentation of any facts on the ground or in the provided records that may be evidence of otherwise unknown or undisclosed title problems. Such facts might include, for example, gaps or overlaps with adjoining, potential encroachments, and uses of the property by other than the owners. The disclosure of these facts allows the title company, lender, and buyer to weigh the associated risks and negotiate or formulate appropriate resolutions to those problems.

The ALTA/ACSM Standards provide for a nationally defined and accepted set of requirements for conducting surveys for use in commercial real estate transactions. Today they are required in conjunction with most such transactions, and the surveyor and other interested parties should be familiar with them.

### Research of Land Records

The actual process of conducting research varies widely across different regions of the United States. In the metes and bounds states (being, generally, the original 13 colonies and the lands claimed by those colonies) surveyors are often expected to conduct their own complete title searches, whereas in the public lands states great reliance is placed on title companies (in keeping with the ALTA/ACSM Standards) to provide the appropriate record document information and any title searches.

In either case, the surveyor must seek out all information from public and private records that may be of benefit in performing the survey. Ideally, this information is gathered and evaluated prior to commencement of the fieldwork. If not, additional trips to the field will be required as new pieces of evidence from the records dictate further field investigation.

Research information comes from a variety of sources. Typically, a visit to the county assessor and/or auditor (the name and function of county offices differs in various parts of the country) will help in the search for the record descriptions of the subject property and its adjoining. Most county tax maps are geographically inaccurate, but they are useful in identifying the juxtaposition of subject and adjoining properties, roads, streets, and other features.

Once the names and property transfer dates are determined, a copy of the actual transfer document can normally be found in the county registry of deeds, county recorder's office, or clerk of the court. This office of public record varies from state to state.

All states allow for the recording of deeds when the transfer of real property takes place and almost all such

<sup>1</sup> "It must not be assumed that a boundary is missing because it is not at once visible" (Breed and Hosmer, 1908).

transfers by deed are recorded. These recordation statutes provide a means of giving notice of ownership of the estates disclosed in the recorded instrument. Unrecorded deeds or other instruments are generally valid only between the immediate parties. These recordation statutes are not applicable in cases of title acquired by unwritten means such as adverse possession.

The surveyor must make use of these depositories to obtain the deeds of adjacent property and check previous transfers of the subject property. Intentions as to boundary location must be obtained from the information at the time the lines and corners were created. Comparison of the deeds of the subject tract must be made with those of the adjoining property to determine if conflicting calls exist.

The type or method of physical recordkeeping varies from county to county and state to state. Many continue to maintain their records as they have for 200 or more years, in transfer books, grantee/grantor index books, and deed record books, while many others have converted to microfilm or microfiche records. With the advent of affordable hardware and software for Geographic Information Systems (GIS), many more populous counties are converting their current and new records to digital format which is often available on-line in the courthouse or, in some cases, even remotely.

However the physical records are kept, the depositories contain the index of all deeds and most of the other legal instruments or writings affecting real property. To use this information, one must be familiar with the system used to index these many thousands of documents. Generally, the indexes are composed of two interdependent parts, the grantor (or vendor) index and the grantee (or vendee) index. The grantor is the party who sells the land; the grantee is the buyer.

Listings in these indexes are chronological by date of recording. Most states use this method of indexing and filing deeds. Although the details may vary somewhat from jurisdiction to jurisdiction, the basic information required for a title search is generally consistent no matter what the system. The surveyor needs to know the name of either the party who bought the property or the party who sold the property. Knowing both buyer and seller is even better, and this information is available from the transfer books mentioned earlier. In addition, knowing the approximate date of the transaction will shorten the search process.

The actual process of checking the title begins with the name of the present owner in the grantee index. The grantor of this conveyance was the grantee of the previous conveyance. Looking each previous conveyance up in this manner will result in the development of a chain of title for the property. Using this chain, a person familiar with reading and interpreting title documents can determine if each person in the chain took title to the property in a regular manner without defect. The grantor's name, the type of instrument and the book and page of the actual recorded

document involved in the particular transaction are listed with each entry. In some instances a brief description and location of the property is included. A copy of a partial page from a typical grantee index book is shown in Figure 13.1. A grantor book is identical except that the index is on the name of the grantor rather than the grantee.

### The Title Search

As noted above, the title research conducted by surveyors themselves varies widely in different regions of the country. In the public lands states, surveyors generally rely upon reports prepared by title companies providing title insurance to the lender and/or buyer. The simple fact that a title company is providing such insurance based on its own search of the records can drastically lessen, if not eliminate, the burden on the surveyor to assemble the chain of title and conduct a title search. On the other hand, surveyors in some parts of the original 13 states (and the lands claimed thereby) often conduct their own title research as part of the normal standard of care.

If a search is conducted, the title should be searched back in time until the property lines of the tract involved originated. From the surveyor's viewpoint, the search is to determine intent of the parties at the time of the original survey or, if there was no original survey, the intent of the parties based on the original writings. The search should establish a line of unbroken ownership in the subject property.

When the search back in time is concluded, the reverse process, called a forward search, is then conducted.

The forward search begins with the name of oldest grantee from the search described above. In the subsequent conveyance, this name becomes the grantor. Using the grantor index and starting with this name, title can be traced forward in time to the present. This provides the surveyor with the chain of grantors. By reviewing each conveyance, it can be determined if any owner in the chain of title has encumbered or impaired title to the land with, for example, easements, other sales, mortgages, or liens. This search will also reveal the creation of servient estates (for example, providing access to the real estate across another property), reservations, and exceptions.

This chain is composed of links. A link is a connection or transfer of property between consecutive owners of the property. These links are not always in the form of a deed but can be a will, the records of an intestate estate (an estate left by a deceased without a will), or a court order. In the absence of a will, the actions of the court must be documented. In order to place the various documents in their proper chronological order and context, a brief summary of each may be prepared. See Figure 13.2 for an example of such an information sheet. The surveyor should be particularly concerned with the property description in each conveyance. He must look for discrepancies in the current deed and may check the current description against those in

## GRANTEE General Index To Real Estate Conveyances - Forsyth County, N.C.

To Locate Names, Open at SURNAME INITIAL TAB and Refer  
To Buff Sub-Index Sheet For Page Reference

Made By The Cott Index Company, Columbus, Ohio  
Sold By Observer Printing House, Charlotte, N.C.

DATE OF INST.	GRANTEES				GRANTOR	KIND OF INSTRUMENT	RECORDED		BRIEF DESCRIPTION, LOCATION, ETC.
	SURNAMES	GIVEN NAMES ABCDEFGH	GIVEN NAMES IJKLMNO	GIVEN NAMES PQRSTUWXYZ			Book	Page	
1987	Gilder		J.	Roger	James M. & Margret Lipsitt	Deed	974	107	George D. Lester Property
1987	DO		J.	Roger	Larry S. & Judy Smith	Deed	976	426	Block 18, Lot 7-Greenhills
1988	DO	Frank	L.		Mark N. & Sally Johnson	Deed	981	331	Thomas Morrison Tract
1988	DO		J.	R.	Robert L. Martin & Wife Paula	Lease	984	274	Easement
1988	DO	Frank	L.		P. L. Bandcroft	Deed	985	67	Harris #3 Tract
1988	DO	Frank	L.		Kenneth & Mary Spillman	D of T	987	493	Lot 41, Block 3-College Park

**FIGURE 13.1** Typical index page for grantees.

prior legal instruments. If plats of survey exist, the deed descriptions should be checked against the plats for inconsistencies.

If the surveyor was responsible for conducting the title search, a post-survey discovery that the real estate in question lacks legal access or that the title to the property is in question is both embarrassing and problematic for the surveyor and the client.

A break in the chain of title occurs when a source reference to a prior conveyance is not found. If this should happen, then a search beyond the record books must be made to determine if a land title was actually conveyed. Such a transfer can occur by some means such as a will, and therefore no deed would be recorded. This makes the search more difficult and time-consuming.

Wills are often kept by the clerk of the court of the local county or city where the subject property is located. For estates settled without wills, one would have to resort to the probate records. Sometimes birth and death records can aid in determining time frames to limit searches for wills and the decisions reached in estate settlements.

### Check of Adjoining Property

Adjoining deeds are checked using the most recent descriptions. Boundary lines of the subject tract are compared with those of the adjoiners to determine if any discrepancies exist. If conflicts are revealed, the associated adjoiner's deed might have to be checked back in time to determine the intent of the original parties when the line was created. As with the title search on the subject tract, this check is accomplished using the grantee or grantor index book to trace the adjoiner's description to the point in time when the line in question originated.

When the calls on the same lines in adjoining deeds do not agree, it may be necessary to establish senior rights for the line in question. Senior rights are those rights gained by virtue of being the first parcel created out of a tract. The law provides that if a grantor conveys land to a grantee, he cannot later convey that same land to someone else. The first deed in such a case would be the senior deed. Therefore, the written intentions of a later deed for property previously conveyed would only convey that portion of property reaching to the land of the senior title holder. While the senior rights may be researched by the surveyor, it is important to remember that senior rights are related to the integrity of the title that a title company is insuring. Thus, when there is a title company insuring the conveyance, the issue may not be "Which property is senior?" as much as "What is the title company going to insure?" Typically, a title company will not insure the land in an overlap area, regardless of which property is shown senior.

### ANALYSIS OF INFORMATION AND PREPARATION OF COMPOSITE MAP

Having either assembled or been provided with the descriptions necessary to perform the survey, the surveyor should determine if these descriptions do, in fact, cover the proposed development area and that the acreage as called for in the document of conveyance is confirmed by the computed quantity.

It is the surveyor's job to interpret the intent of the description and to place the lines and corners it describes on the ground in accordance with accepted principles of law.

Much emphasis is placed on the intention of the parties at the time a monument or boundary originates, so the surveyor must know how to find that intent in a manner

<p><b>Grantor:</b></p> <p><i>John Smith &amp; Wife Jill</i></p> <p><b>Grantee:</b></p> <p><i>David Doe &amp; Wife Dawn</i></p>	<p><b>Instrument</b> <u><i>Deed</i></u></p> <p><b>Book</b> <u>1037</u> <b>Page</b> <u>401</u></p> <p><b>Date</b> <u>9/13/90</u></p> <p><b>Acknowledged</b> <u>9/15/90</u></p> <p><b>Recorded</b> <u>9/15/90</u></p> <p><b>Warranty</b> <u><i>Fee Simple</i></u></p> <p><b>Considerations</b> <u>\$ 50,700.00</u></p> <p><b>Stamps</b> <u>\$127.000</u></p> <p><b>Liens</b> <u><i>None</i></u></p> <p><b>Released</b> <u>---</u></p> <p><b>Restrictions</b> <u><i>Deed Book 1035 Pg 207</i></u></p> <p><b>Acknowledgement</b> <u><i>O.K.</i></u></p> <p><b>Execution</b> <u><i>O.K.</i></u></p>
<p><b>Granting Clause</b></p> <p><i>Smith &amp; Wife -- Grant &amp; Convey -- Doe &amp; Wife joint tenants / right of survivorship</i></p>	
<p><b>Description:</b></p> <p><i>Being all of lot 14, Block 9, Section 4 Pine Hills Sub-division</i></p>	
<p><b>Source of Title Deed Book</b> <u>1032</u> <b>Page</b> <u>140</u></p>	

**FIGURE 13.2** Information sheet for title search.

that will be upheld in a court of law should his interpretation be questioned. Except in specific instances, intent is to be garnered exclusively from the writings, which are considered to represent the intentional acts of the parties. Only when the writings cannot be interpreted without additional evidence or the description is so ambiguous as to beg explanation can the surveyor seek clarifying evidence from outside the deed.

The acts and intentions of the parties are to be considered under the circumstances existing at the time of origination. Thus, one frequent reason for seeking outside evidence is to clarify what those conditions may have been. For example, a deed call "to the right-of-way line" of a street

that has been widened numerous times will require research of additional documents outside the deed itself to determine which right-of-way line was being contemplated by the description.

Once all of the records pertinent to the subject property have been obtained, the work of examining those records for determining boundaries can begin.

The starting point for the resurvey or retracement must be the same point described as such in the record description. In the public land states, this point is typically a section, quarter section, or quarter-quarter section corner, and the primary challenge is to determine where that corner was established originally. In the metes and bounds states, the

intent may be less clear, and the goal is to identify the point most closely in agreement with the description as written and the evidence of actual ground possession.

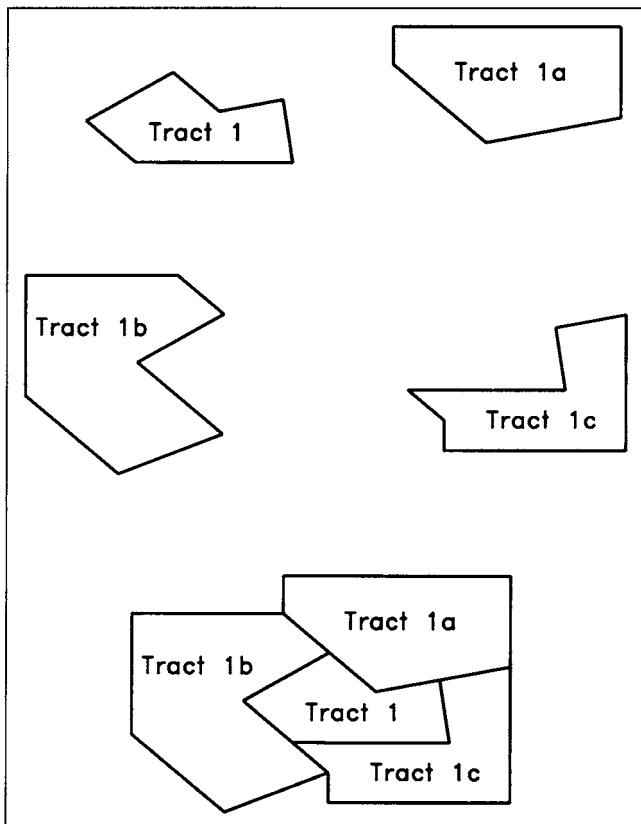
The analysis of the boundary continues using all appropriate information gathered during the research phase. A composite map of the subject tract and adjoining tracts is typically prepared. Figure 13.3 is an example of a composite map. Each description from the various tracts of land is plotted at the same scale and in reference to the particular tract being surveyed, much in the same manner that the parts of a jigsaw puzzle are assembled.

The composite map will include the bearings, distances, and calls for every line and corner represented. The composite map thus becomes the surveyor's guide for determining where evidence of the boundaries might be found during the field survey. It is also helpful in identifying potential conflicts between the subject tract and that of the adjoining tracts.

The surveyor must realize that evidence must be viewed considering the rules of law. Some of these rules of law will be covered in more detail in a subsequent chapter on the legal aspects of surveying. It is, however, necessary to consider them now for understanding of the analysis process.

### Monuments

Before the surveyor can begin making decisions related to the use and application of existing monumentation in de-



**FIGURE 13.3** Composite map.

termining the location of lines, he must understand the distinctions among existing corners, obliterated corners, and lost corners. Existing corners are defined as “those whose position can be identified by verifying the evidence of the monument or its accessories, by reference to the description in the field notes, or located by an acceptable supplemental survey record, some physical evidence, or testimony” (Bureau of Land Management, 1973).

An obliterated monument is one where no visible evidence of the work of the original surveyor remains to support the location as an original monument. Its location has, however, been preserved by acts of the landowners or is otherwise recoverable based on other supplemental evidence. The location can also be established from the memory of those who knew of the location of the original monument.

A lost corner is a point of a survey whose position cannot be determined beyond a reasonable doubt. When no traces of original marks are found and no acceptable evidence or testimony is available that supports the location of the corner, it is considered lost. The position of a lost corner can only be restored by reference to other independent corners.

Monuments control in the decision process when they meet the standards stated in the law. The law requires that a specific call must exist in the deed or other instrument for a point to have the standing of a controlling monument. An example of a specific call is as follows: “thence North 35° 15' West, 1357.8 feet along a fence line.” The words “along a fence line” give the fence the standing of monumentation and the fence takes precedence over the bearing. Monuments not called for in the deed are usually not controlling factors in determination of boundaries in a court of law. The monument must have characteristics that make it identifiable as the original monument. Evidence of the monument's disturbance will weaken its standing.

### Discrepancies

The surveyor must check for consistency of identifying calls for monuments by adjoining documents with those of the subject property. Any differences should be noted and reconciled if possible.

Discrepancies in metes and bounds descriptions are often the result of variation in magnetic north between the time the legal description was constructed and the present reference to north. Variation in distances can also exist. Human error and improvements in technology are both factors in discrepancies between calls in different documents.

Frequently the surveyor must search the records for a longer time than required for title purposes. There are two reasons for this extended search. As previously discussed, when senior rights are involved in the determination of ownership, such a search may be mandated. The second situation is required because bearings are often copied from older deeds. The origin of the bearing would be the date of the survey that produced the bearing. The date of the

origin of the bearing must be known so corrections for magnetic declination can be properly made. This is necessary in the proper preparation of the composite map. References to direction in different documents having various dates can be adjusted to a common north system of reference. This common reference could be true north, State Plane Coordinate Grid north, or some other local system that might be prescribed.

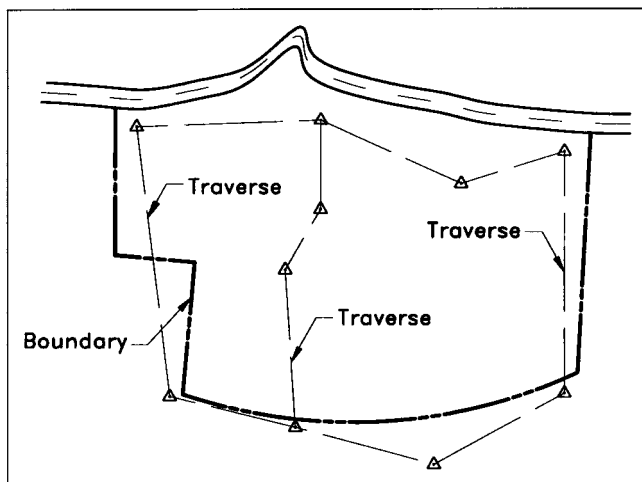
Differences in distances can result from a failure to convert units used in original surveys to those of the adjoining property or those in use at present. Reconstructing an old survey is not a simple matter. The surveyor must be familiar with the units of measurements used at the time the original survey was made. Often these measurements had more than one interpretation.

Early land grants were often described using differing lengths of chains and varying sizes of acres. The best example of this is given by the variation in the acre in different parts of England. In Devonshire and Somersetshire, the acre was 160 perches of 15 ft, or 36,000 ft<sup>2</sup>. In Cornwall, the perches were 18 ft and yielded 51,840 ft<sup>2</sup>. In Lancashire, the perches were 21 ft and the acre was 70,560 ft<sup>2</sup>. When the early settlers of this country began to survey, the land they often used the measure with which they were familiar.

Courts throughout the United States, particularly in the Western states, have found that many of the original government surveys of parts of the public lands were imprecise. The reasons for these inaccuracies included low land value, difficulties encountered during the surveys, and, in some cases, outright fraud. This phenomenon is also true for those early surveys of the original grants in this country.

### Preparing of the Field Map

Typically, a comprehensive map (see Figure 13.4) is prepared for use by the survey crew in conducting the fieldwork on the survey. The map should include information on those points and lines mentioned in the various deeds



**FIGURE 13.4** Field map.

and documents. Positions of monuments called for in adjoining deeds should be indicated and labeled. Horizontal traverse control to be used for survey is shown and identified.

Tax parcel numbers, names of adjoining owners, deed book, page numbers, and acreage are given for each of the adjoining tracts. Assembling a file for the survey project in an orderly manner such as that outlined below will provide a valuable reference for the field crew and for future reference.

1. A tax map with subject tract outlined in color and a tabulation of ownership information derived from assessment office
2. The chain of title and other research notes properly arranged in chronological order
3. All deed descriptions with tax assessment parcel numbers and deed book and page numbers identified
4. Any other miscellaneous plats, aerial photographs, or related information.

### APPLICATION OF INFORMATION TO THE BOUNDARY SURVEY

It is important to realize that office work and field work progress jointly. The two functions are interdependent. Office work requires continual input of field data to move forward efficiently and effectively. Continued fieldwork requires office computations and analysis of data previously supplied.

### Standards

Boundary surveys for land development differ from the typical rural or farm boundary survey. In addition to gathering of evidence for property line location and for use by the title company in evaluating potential title problems, these surveys require the compilation of features that will affect the later development.

For a description of what must typically be located in the field and shown on such a survey, refer to the Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys in the Technical Appendix.

In addition, most states have a set of standards adopted by their Boards of Registration. These standards vary in detail significantly from state to state, but all outline minimum requirements for the practice of surveying in each state and registrants must meet these standards in addition to any other site- or client-specific requirements.

### Fieldwork

The field phase begins with reconnaissance to locate corners or other calls referred to in descriptions and pertinent to the survey. The reconnaissance also serves as a basis for planning the subsequent fieldwork. When appropriate, it provides an opportunity to notify adjoining landowners of



the impending survey. This notification might also be done in a more formal manner. Some states require written notice to adjoining owners before the fieldwork begins. The field survey will proceed more smoothly if the cooperation of adjoining landowners can be secured.

### The Traverse Survey

Once the standards under which the work will be undertaken have been established, a random traverse of the subject property is typically run. The traverse may be conducted using conventional equipment and procedures, such as measuring angles and distances with a total station or theodolite and an electronic distance measuring device (EDM), or the surveyor may employ the use of Global Positioning System (GPS) receivers to establish locations using satellites. Frequently a combination of procedures and equipment is used. Terrain, improvements, vegetation, and traffic often dictate the most logical equipment and procedures to employ.

No matter what procedures and equipment are used, the lines and points on the traverse should be located on the subject property when at all possible to prevent damage to trees and shrubs belonging to adjoining owners. Stations on the survey should be placed at the most advantageous locations and frequencies, considering they will be subsequently used to locate features on the property and possibly for other functions later in the development process. These uses include topographical surveys, construction stakeout work, and control surveys for the development. If points are destroyed, the configuration of the initial traverse should allow for easy replacement.

The traverse control points serve as the basis for locating visible property corners, fences, tree lines, and other features relevant to the title lines and corners in addition to those improvements called for in the state or ALTA/ACSM standards or as may be required by the client.

Cross-ties between nonadjacent points of the traverse should be made as frequently as is practical. This affords checks on the survey work and allows for better survey adjustment of final results. These cross-ties also provide additional stations for control and stakeout in later stages of development. Survey markers set on this traverse, such as rebar, pipes, or concrete monuments, should be of a length and diameter to be secure after installation. Alternative objects such as PK nails or scribe marks can be used when they are more suitable for the conditions.

All traverse stations should be referenced for ease of future recovery. These reference ties should, when possible, be at 90° to each other, which provides for a more sound geometric solution in recovery. A minimum of three references is desirable. The references should be accurately described and recorded in the field notes. See the illustration in Figure 13.5 for an example.

Field crews must be sure that the information to be located in relation to the boundary traverse is properly tied to the survey traverse line using methods that afford redundancy in measurement better to ensure the integrity of

the data. This is particularly important when locating markers and monuments, which will be integral to the resolution of the boundary. See Figure 13.6 for an example of points to be tied to the traverse.

Blunders must be eliminated by providing sufficient tie measurements. Following correct procedures to prevent human errors such as misreading equipment and errors in notekeeping is an absolute necessity. A few simple additional control measurements allow for field determinations and an immediate rectification of such errors where required. Extra trips to the field can be avoided by using a pocket calculator to check measured data against a mathematical fit.

### Locating Features and Improvements

The lines of the boundary traverse should follow the record lines when possible since the traverse will serve as the basis for locating evidence, which will determine the position of the actual property line.

All iron pipes, concrete markers, stakes, and other objects used to mark corners and points on-line are evidence and must be located. The locations of tree lines, old fence lines, walls, ditches, and other features on or near the division lines must be determined and described in detail, including the material from which objects are made and the condition of the objects. Photographs of the evidence for future reference will be helpful; they can provide excellent supportive evidence should the survey result in litigation. Points found during the survey and called for in the deed should be used as traverse stations when they are accessible.

Existing streets may have an impact on the boundary line determination and later development. The locations of pavement, right-of-way lines, and centerlines should be recorded in the field notes. When no pavement exists, the traveled way should be located. Access roads on or across property, whether in use or not, should be located and shown. Such roads might be used for access to surrounding property and as such might be included in an easement. Cuts or embankments, tree lines, or fence rows, which might indicate long-abandoned roads, should be located and shown.

Drainage ditches, streams, creeks and rivers must be recorded in the traverse survey notes. Details about these features are most important in the land development process. Locate these features if they are mentioned as a boundary or if they cross a boundary line. Information on the size and condition of waterways should be included in the field notes.

A complete explanation of the conduct of the fieldwork is beyond the scope of this book. Surveyors unfamiliar with generally accepted standards and procedures should consult appropriate sources of information and their peers for assistance.

### Easements

Many land and title features may be located within easements. In addition, easements needed to install or maintain

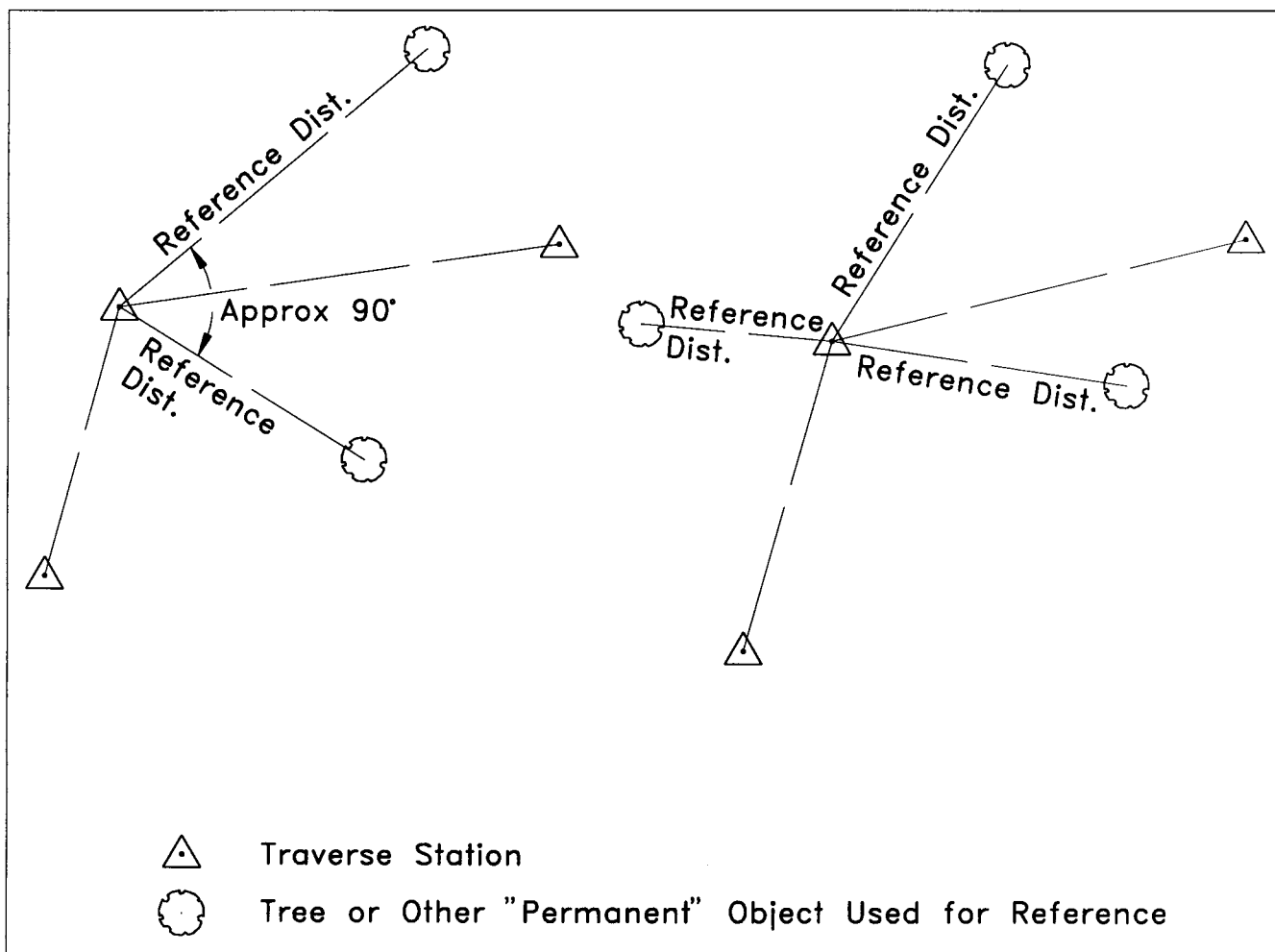


FIGURE 13.5 Methods to reference traverse stations (traverse ties).

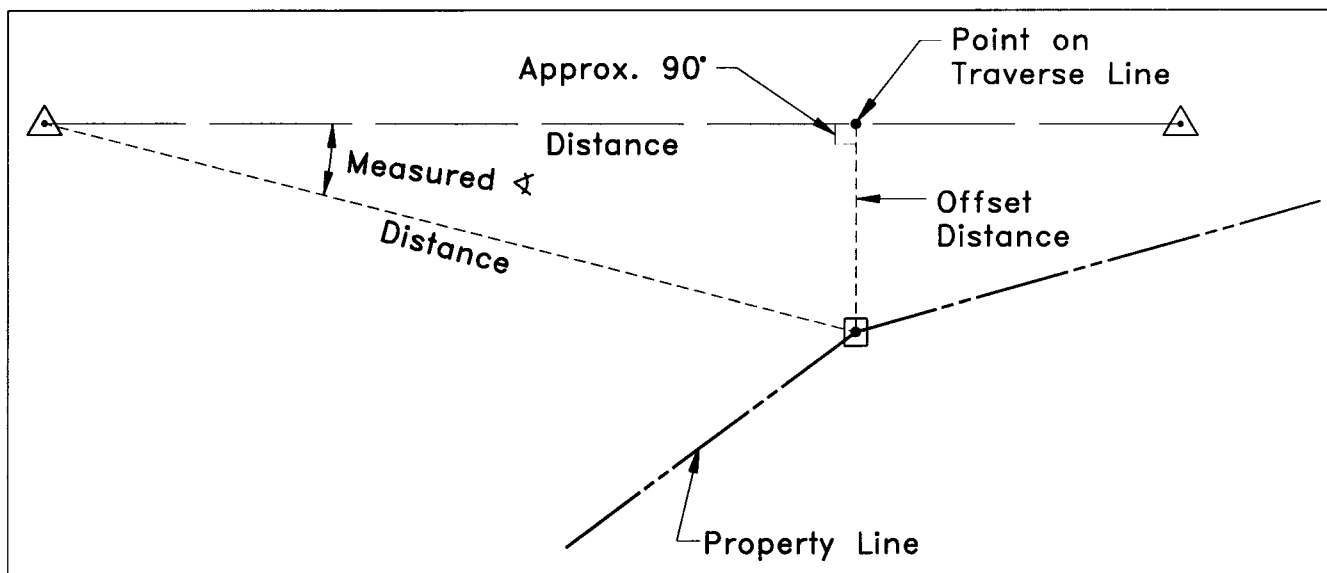


FIGURE 13.6 A boundary point is tied to the traverse using a minimum of two references. Shown here is (1) a measured angle and distance directly to the point and (2) measured distance to POL and offset distance to the point.

the infrastructure of the land development must be established. These include easements for streets, wells, drainfields, stormwater runoff, storm and sanitary sewers, gas lines, and power lines. Because of their importance, the nature and location of easements that already exist and those that are to be created must be carefully considered. Often, proposed easements will connect to existing easements.

A simple general definition of an easement is a nonpossessory interest held by one person in land of another. The person holding that interest is accorded partial use of the burdened estate for a specific purpose. An easement restricts but does not abridge the rights of the burdened estate fee owner to the use and enjoyment of his land.

Two types of easements, express and implied, can lead to land disputes. The law provides for express easements. The law also recognizes that easements may exist or be created that are not express. These are implied and prescriptive easements.

Easements fall into three broad classifications: surface, subsurface, and overhead.

It is important that the surveyor determine if any easements exist at the time the boundary survey is being conducted, since they may interfere with later development plans. This is why locating existing and potential easements is such an important part of the ALTA/ACSM Minimum Standards. Should any possibilities of easements exist, they must be reported to the client at the time of the boundary survey. It is not the surveyor's responsibility to determine whether an easement has been created in fact but instead it is his responsibility to report its possibility.

Many times old logging roads or field roads exist on the property to be developed. These can usually be easily seen on an aerial photograph or visually located by ground survey. This type of road could be evidence of an existing easement created by prior use or by necessity. If by chance the adjoining land became landlocked because of the closure of such a road, an easement by necessity could be required across the burdened property.

Some states have statutes to provide for easements of various widths for ingress and egress to public highways over intervening lands. There are many purposes for granting these easements. Some common examples are cultivation, cutting and removing of timber, mining operations of various types, and manufacturing plants.

Locations of future easements that will be required for development should be considered while the boundary survey is in progress if the development plans are far enough along to give such consideration. This can be done by referencing service lines of different types that adjoin or cross the subject property. It is the surveyor's duty to report to his client all possibilities of easements found. In accomplishing this, he or she must apply that degree of skill usually exercised by a person of ordinary prudence engaged in the same work. Because of their nature as a nationally rec-

ognized standard, it is likely that a court would determine that the ALTA/ACSM Standards define that degree of skill.

### Grave Sites

It is critically important to locate any evidence of the existence of grave sites. The ALTA/ACSM Minimum Standards emphasize this importance. In some jurisdictions, grave sites by law are granted protection against disturbance even if no easements or other records concerning these sites exist. Aerial photos are useful in location of deserted graves and cemeteries. Burial plots are often near groves of trees or the tops of knolls. Time spent locating these sites on a photograph and a field trip to examine these areas are necessary to avoid problems that occur when a grave site is overlooked. During this closer inspection the surveyor should look for grave markers (both head and foot markers) and depressions in the ground.

### Evidence of Possession

The surveyor should make careful notes of all visible evidence of possession or occupation, such as the cutting of timber, farming, and construction. Any evidence that could be construed as possession of the land must be investigated and identified. The ALTA/ACSM Minimum Standards outline the surveyor's responsibility in this regard in careful detail. Possession can ripen into a fee right that could destroy prior title rights. When land development is pending, failure to discover claims related to such possession can lead to long and costly delays and even work stoppage as the project progresses. The possibility also exists that injunctions over such matters might stop future land sales.

### Office Work and Computations

Meetings between the party chief in the field and the office surveyor (compiler and computer) should be held regularly. It is necessary that they work as a team. They must develop a joint feeling of accomplishment in a job well done.

Once the data-gathering phase of fieldwork is completed, the field notes are turned over to the person assigned to compiling and computing the information for the project. This is the time for the party chief and the person in charge of office computations for the project to coordinate their efforts. The party chief can relay any additional information known by him or her to the person compiling the data. Such information might include a variety of observations of conditions on the ground that, for whatever reason, did not find their way in to the field notes.

As this information is all compiled into presentable information, the surveyor can begin to make a determination of boundary locations. He or she should start by checking the field data. Aerial photographs may be used for comparison of data if possible. Field information on lines and points found can be compared with lines and points called for in the record documents. The composite map previously referred to is most helpful when the field data are added to

it. The map becomes the main source of comparison of field and record data.

Before the survey data can be plotted, the field notes must be reduced. Averages must be determined when multiple angles and distances were measured.

In confirming the mathematical integrity or closure of a traverse, angles are checked for error first. The field results can be summed and compared to the value that should exist depending on the particular geometric shape of the subject traverse.

After it is determined that the result of the angular error lies within the allowable limits documented in the specifications, the angles are adjusted. Using these adjusted angles, a closure can be calculated for the traverse to determine the linear error. If this precision is within prescribed limits, which may also have been set by specifications, then adjustments can be made to balance the error. Should either angles or distances lie beyond the limits allowed, office computations can be made to determine which measurements might be in error. The field crew must return to the field to remeasure suspect angles or distances.

The use and application of positional tolerance or positional uncertainty are becoming commonplace in federal and state standards and now also appear in the ALTA/ACSM Minimum Standards. This concept involves the quantification of probable errors in individual measurements and the computation of the resulting potential error in the locations of the points on the survey. Such a standard typically does not dictate procedures or equipment, but rather allows the surveyor latitude in applying proper procedures and employing his equipment and personnel in a manner that he sees fit as long as he can ensure that the allowable tolerances are not exceeded.

Where no specifications are otherwise required by contract or state laws, the Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys can serve as a guide for allowable errors in both angles and distances, although, as mentioned earlier these standards will most likely be a part of the contractual requirements. The surveyor should keep in mind that he or she must strive for higher standards than the minimum. Some work requires much higher precision, and the surveyor should be acquainted with the local requirements of precision for development.

Once the integrity of the control survey points has been assured, the coordinate values of all points included in or located from the traverse are computed. This includes fences, pipes, and monuments in addition to all other points of reference. All of these points are plotted in their respective positions relative to each other.

These location data are used as the basis for the evaluation of the field evidence in conjunction with analysis of the evidence contained in the various record documents and deeds. Using accepted principles of law, the various pieces of evidence are given their appropriate weight and the locations of the missing corners are computed.

The relationships of these points to their nearest traverse stations are determined, allowing the field crew to make an efficient and more complete field search for these points.

The person assigned to making boundary determinations should walk all lines of the survey, searching for points called for but not found and looking for fences, utility lines, encroachments, roads, streams, or other items inadvertently omitted from the survey. Before a monument is reported as destroyed, it must be assured that that destruction has occurred. This is as important as the location of objects that were found. A buried monument or a bound called for but slightly out of place and found too late can be a source of serious future problems. This trip will facilitate decisions related to the final boundary placement. The surveyor must be sure that nothing affecting the location of the boundary is overlooked.

### Boundary Determination

The surveyor has no legal power to establish boundaries. Her function is one of identifying the most conclusive evidence about the existing limits and location of ownership of real property. To carry out this task, two principles that guide the surveyor. The surveyor must follow both of them as the specific condition dictates.

The first principle is that the intentions of the parties at the time the boundaries and corners originated are to be construed exclusively from the description itself, unless extrinsic evidence is required to do so. The boundary lines and corners are then established accordingly. Many times the task of identifying intent will necessarily include analysis of field conditions to assist in understanding the initial intent.

As time progresses, the evidence of intent becomes less clear and incomplete evidence can lead to incorrect conclusions. This prompted the need in law for the development of a second principle. This basic rule is that lacking clear intent in the writings, the longstanding acceptance of a boundary location may be considered evidence of the originally intended location.

The application of this rule is probably more important where the land being surveyed is in the process of development. Much consideration should be given to the claims of adjoining landowners. If minor differences can be reconciled by agreement with adjoining, this is usually the most appropriate method of settling such disputes. If an adjoiner has been claiming a boundary and has possession of the claim, the fact that litigation is time-consuming and costly must be given weighted consideration.

In surveys where relocated corners do not agree with all calls of original documents or notes, attention must be given to the rules of construction for ambiguous descriptions. These rules simply set the priority for considering physical evidence. That order is as follows: (1) natural monuments, (2) artificial monuments, (3) calls for adjoining, (4) courses and distances, and (5) acreage. When the inconsistency is between course and distance, the distance is gen-

erally considered more reliable, although this depends on the state in which the property lies. This subject is discussed in more detail in the next section. The surveyor must understand that these rules of construction are subject to change if they result in absurd conclusions. The choice for construction is the one that brings the most harmony with the deed.

### LEGAL CONSIDERATIONS IN BOUNDARY DETERMINATION

Thomas M. Cooley, Chief Justice of the Michigan Supreme Court in the 1880s, was the author of the address titled "The Judicial Functions of Surveyors." This address covered the role of surveyors when they are performing relocation surveys of old boundaries. The concluding statement from this address was, "Surveyors are not and cannot be judicial officers, but in many cases they act in a quasi-judicial capacity with the acquiescence of parties concerned; and it is important for them to know by what rules they are to be guided in the discharge of their judicial functions."<sup>2</sup>

All deeds, wills, agreements, and evidence must be considered in making the final determination of a boundary location. Surveyors gather and analyze the evidence to support claims concerning the true location of corners and lines. The determination must be made considering the intentions of the parties at the time the boundary was first established. The laws governing surveying and real property also play an important part in the decision process and in the weighing of the evidence to support claims. Final boundary determinations rest with the courts, but fortunately, as stated earlier, most boundary locations can be resolved with confidence by competent surveyors.

### Conflicting Title Elements

In making decisions based on discovered evidence, the surveyor must be familiar with the order of importance of conflicting title elements. These elements were mentioned earlier and are discussed below. The surveyor must be familiar with cases enabling conclusions to be drawn based on precedence set down by the courts in similar situations.

### Possession

The first of these elements of importance is the right of possession. These are the rights not stated in writing but, as pointed out earlier, can become rights in fee. This fact is cause to give first and highest consideration to these unwritten yet visible claims to land ownership. The consideration of unwritten rights often involves adverse possession, although there are a number of other types of unwritten rights, such as acquiescence, estoppel, and oral agreement. Such possession is a matter often determined initially by a land survey and ultimately, in most states, by a court of law; the surveyor cannot make a determination of the legality of the possession. The more evidence the

surveyor gathers, the better able he is to inform his client as to the potential of an adverse claim.

Adverse possession is a means of gaining ownership of land based on possession. This form of obtaining ownership is one that can cause many problems for the land developer and the surveyor.

This doctrine began in feudal England with statutes of limitation. The limitation under English statutes was set at 20 years. The time was referenced to some royal event such as a coronation or death of a king. The feudal concept of ownership was based on possession.

The modern principle of specifying a uniform number of years as the limitations period for real property actions began during the reign of Henry VIII. Early American colonies and several states today have continued a limitations period of 20 years as it was set in the 1623 Statute of James I. Under this early statute, if a freeholder was dispossessed, he had the chance to regain possession and claim to the estate of freehold.

This is accomplished today by bringing an action similar to the ancient writ designed to recover real property interests. The dispossessed party must seek a writ of right or possession before the statute of limitations expires. Failure to comply with this requirement may result in the rights of the party that dispossessed becoming absolute. The modern principle of adverse possession differs because the record owner does not lose title or ownership when the adverse possession begins, but rather after the statute of limitations has run.

Gaining of title by adverse possession is dependent on a variety of factors in addition to meeting the statute of limitations. If those requirements are met, the record title owner may lose that title through later judicial processes and the person with adverse possession rights might become owner of the property.

The claimant's possession generally must be open and notorious, continuous, hostile, actual, visible, exclusive, and under a claim of title. The exact combination of requirements and the length of the statute of limitations vary from state to state. In any case, however, the statutory period necessary to defeat title and satisfy the required statute of limitations must be met (Brown, 1969, Sec. 2.34).

The elements of adverse possession are required because the record owner must have actual knowledge of the adverse claim against his holdings. The terms *open*, *notorious*, *actual*, and *visible* all raise the presumption at law of notice. The true owner being intentionally infringed upon is essentially given this notice by virtue of the claimant meeting these requirements.

Hostile possession is often misunderstood. A good example of its meaning can be explained by resorting to case law. The majority rule is that possession based on a survey mistake is sufficiently hostile. This is true as long as the person in possession is claiming the property as his own (Powell, 1987, ¶ 1013[2][f][i]).

The requirement of continuous possession means that the true record owner of the land has had suitable cause

<sup>2</sup> *Diehl v. Zanger*, 39 Mich. 601, 605 (1878).

(right to act) and time to bring legal action against the adverse claimant during the entire time of possession.

The requirement that the possession be exclusive might be characterized by a situation whereby anyone other than the adverse claimant must receive his permission to enter on or use the land being possessed.

### Senior Rights

The next element in order of importance for determining ownership is called “senior rights.” A brief discussion of senior rights was included earlier in this chapter. Senior rights have been stated as the rule of “first deed and last will” (Brown, 1969, Sec. 4.14) Put another way this statement simply means that one’s interest in land can only be transferred to another party or parties once. This prevents the same interest from being sold later at a date to another party or parties.

### Written Intentions of the Parties

After consideration of the preceding elements, the written intentions of the parties are the paramount consideration. These intentions are those expressed by the parties to a conveyance and put in writing in the document that brings about that conveyance. The written intentions of the description are used to consider the importance of the remaining conflicting title elements. The strongest expression of intent is a call for a survey.

### Calls for Monuments

Monuments are the next element to consider when a situation of conflicting elements arises. Natural monuments usually take precedence over artificial monuments. In some instances, artificial monuments can become so well identified that they obtain equal standing with the natural monuments. When an object built by humans becomes generally well known and recognized and is considered permanent, it may attain the characteristics of a natural monument

Monuments of record (sometimes referred to as calls for adjoiners) are often considered a third type of monument. Calls for record monuments become relevant when other calls result in gaps or overlaps with adjoiners. Record monuments most often gain importance in situations involving senior rights, although a call for an adjoiner does not in and of itself establish senior rights.

### Distance, Direction, Area, and Coordinates

After consideration of the monuments, elements such as distance, bearings, or directions called for in the description are next in importance respectively. Area does not have a high priority in the conflicting elements except in cases of wills. The intent in wills and in some descriptions is often expressed specifically by a certain number of acres, and in those cases area receives first consideration.

Coordinates are given little consideration. They are generated from information already mentioned. Therefore, they are subject to mistakes in computation. Parties to a con-

veyances rarely understand the relationship of coordinates to a field location, so they cannot represent intentions of these parties.

There are other legal possibilities that might affect the location of property lines. These include boundaries by agreement, acquiescence and estoppel.

### Boundaries by Agreement

Confirming that a boundary has been established by agreement is dependent on finding a mutual agreement between landowners as to where the common boundary is located. Courts often require that certain factors be present before boundaries can be established based on such agreements. Boundary agreements must involve lines or corners that are uncertain or disputed. These agreements must set out a specific line as the boundary. They must be executed by acts of the parties occupying adjoining land, and the agreement must include land to the agreed line. The final requirement is that the agreement be recognized for a considerable time. This period is often equal to that required for adverse possession.

### Boundaries by Acquiescence

Acquiescence is similar to boundaries by agreement. This type of ownership is unique, however, because it does not require proof that the boundary location resulted from an agreement. The justification for this ownership is similar to adverse possession. For this doctrine to be enforceable, occupation must be visible up to a definite marked line. Generally, a long period of time must have elapse before acquiescence exists, although one theory of acquiescence is that it is based on a prior agreement.

### Boundaries by Estoppel

Boundaries by estoppel develop when the true owner knowingly misrepresents his boundary and causes a neighbor to rely on the misrepresentation thereby incurring detrimental cost. In such cases, the boundary becomes that as represented. Estoppel is defined as “a man’s own act or acceptance stops or closes his mouth to allege or plead the truth.”<sup>3</sup> The concept of estoppel works to bar, stop, impede, prevent or preclude denial of a certain fact as illustrated by the following example. A property owner who knows that an adjoiner is making improvements along a line, which they have incorrectly believed to be the actual boundary line, may later be estopped from claiming a true line running through the improvements.

### Party Walls and Line Fences

Line fences are special cases when they divide properties. Known in law as division or partition fences, they are covered by statutes in all but six states. Such a fence is built on an equal amount of land of each of the adjoining owners. Fences built entirely on the land of one party are not

<sup>3</sup> *Caulfield v. Noonan*, 229 Iowa 955, 295 N.W. 466, 471 (1941).

division fences. In these cases, the boundary lines prove the fence location. Fences not called for in the description of lines when the lines originated do not prove the lines. A fence can become a line fence by consent of the owner of the land on which the fence was built.

Land development can also involve boundaries established by party walls. Party walls are located similarly to division fences. Party walls are usually built after formal agreement of the concerned owners. Only a few states and the District of Columbia allow the building of such walls without agreement of adjoiners.<sup>4</sup>

### Evidence

It is evident the surveyor must consider any factors that might impact future development. Evidence of construction on the land that might be grounds for liens must be reported. He or she must also determine from local authorities if there is any proposed construction around the subject tract. Such construction might cause changes in streets, sidewalks, water, sewer, gas, or other service lines that could have an impact on the property detrimental to the proposed development.

In addition to this basic knowledge of law the surveyor must be familiar with state statutes and local ordinances and have a knowledge of case law related to boundaries. Those cases related particularly to land development should be checked when a situation of conflict over a boundary develops.

Laws form the basis of land ownership. Proof of the limits of this ownership requires gathering evidence by surveying the boundary. The surveyor should remember that for each law there are many exceptions to that law. Each land survey is unique and different. This is because the physical characteristics of the land vary and the circumstances surrounding the conveyance govern the interpretation of the intentions of the parties.

The deeds located in the record search become the basis for locating and establishing boundaries. They are used to identify the true location of monuments and courses, as originally run, on the ground. This holds even if the measurements by the original surveyor were not correct.

All of this relates to what is called the rules of evidence. The definition of the law of evidence is stated as "the aggregate of rules and principles regulating the admissibility, relevancy, and weight and sufficiency of evidence in legal proceedings."<sup>5</sup> Evidence may be defined as "Any species of proof, or probative matter, legally presented at the trial of an issue, by the acts of the parties and through the medium of witnesses, records, documents, concrete objects, etc., for the purpose of inducing belief in the minds of the court or jury as to their contentions."<sup>6</sup> Evidence is not proof but the

consideration of it and the deduction reached from it produces confirmation as to what will prevail. Not all evidence carries the same weight in establishing proof.

Indispensable evidence, as the name implies, is that evidence absolutely necessary to prove a particular fact. In the case of land boundaries based on adverse possession, for example, the evidence must establish the fact of that possession such as enclosure. The surveyor would therefore locate fences, which would constitute indispensable evidence of such enclosure.

Evidence that cannot be disputed is called conclusive. The clear contents of a written instrument such as a deed cannot be changed by the oral testimony. Therefore, documents that are a part of a conveyance must be considered in high regard when trying to prove the location of a boundary.

*Prima facie* evidence is that which will stand as correct until it is rebutted by other evidence. A probated will granting a party ownership to a tract of land is *prima facie* evidence of that ownership. Proof of the rebuttal of the will would be necessary to contradict such evidence.

Other classes of evidence are primary evidence, secondary evidence, direct evidence, indirect evidence, partial evidence, and extrinsic evidence. Primary evidence is original or first-hand evidence. It is that evidence which is given priority for ascertaining the truth. A copy of a document is secondary evidence because it is considered inferior to the original document.

Direct evidence supports the existence of a fact without depending on proof of other facts. The testimony of an eyewitness to an event is considered direct evidence.

Indirect evidence, on the other hand, requires proof of a fact by proof of another. This evidence is also called circumstantial evidence. Evidence used to establish a detached fact is partial evidence. This type of evidence is subject to be rejected unless connected to the fact in dispute by proof of other facts.

Extrinsic evidence is that not contained in the body of a document for agreements or contracts. Such evidence is primarily for explanation of references in the written documents. This evidence can be used to clarify meanings of terms that vary within local context.

The surveyor must understand the types of evidence and realize that the law of evidence is not exact but relative. The rules referred to in the definition of the law of evidence can be found in the statute law of the state where the subject property exists. These rules deal with judicial notice, presumptions and burden of proof, and the relevancy, competency, and admissibility of evidence. Other considerations in the rules of evidence include real and demonstrative evidence. Documentary evidence, secondary evidence, and parol evidence are also included. In addition, expert evidence, opinion evidence, and hearsay evidence are discussed in the rules. Topics such as declarations and admissions, weight and sufficiency, and questions of law

<sup>4</sup> See, e.g., D.C. Code Ann. §§ 1-625 to 1-626 (1981).

<sup>5</sup> *Black's Law Dictionary*, rev. 4th ed., s.v. "evidence, law of."

<sup>6</sup> *Black's Law Dictionary*, rev. 4th ed., s.v. "evidence."

and fact are dealt with in the same chapter of the statute law.

The understanding of evidence is critical to the surveyor involved in land development since determinations of the location of property lines and corners are often immediately relied upon as a basis for work to progress. Often much or all of the development has been carried to completion before legal problems manifest themselves. Should the land under development become involved in litigation, the surveyor's interpretations will be admitted or rejected in court depending on his adherence to the rules governing the law of evidence.

It has been said that "everyone is presumed to know the law and the surveyor is no exception. If he agrees to monument a certain written conveyance on the ground, he also agrees to locate the conveyance according to the laws regulating the interpretations of written conveyances" (Brown et al., 1981, Sec. 1.3).

### Basic Rules for Guidance in Boundary Location

Many rules of case law are available to guide the surveyor in making determinations concerning boundary locations. These rules are too numerous for a complete list to be included here. They are being constantly added to and amended with each court opinion that is handed down. Some case law included here will make the surveyor aware of those that have more general application. Those covering more specific situations will require research in law libraries. It should be emphasized again that rules of law are often overturned. In addition, exceptions to the rules are commonplace. Rules are therefore only a guide in the decision process. The courts make the final determinations should boundaries fall into dispute. Following are a few of the primary rules established by case law:

*Rule 1:* In ascertaining boundaries from title papers, he who has the oldest title is entitled to take his courses and distances, go where they may.

*Rule 2:* Written descriptions of property are to be interpreted in the light of the facts known to and in the minds of the parties at the time. They should be construed with reference to any plats, facts, and monuments on the ground and referred to in the instrument.

*Rule 3:* Where the description of the boundaries in a deed is indefinite or uncertain, the construction given by the parties, and manifested by their acts on the ground, is considered the true one unless the contrary is clearly shown.

*Rule 4:* Every call in the description of the premises in a deed must be answered if it can be done, and none is to be rejected if all the parts can stand consistently together.

*Rule 5:* The certain description must prevail over the uncertain, in absence of controlling circumstances.

*Rule 6:* Where the boundaries mentioned are inconsistent with each other, those are to be retained which best subserve the prevailing intention manifested on the face of the deed.

*Rule 7:* When one part of the description in a deed is proved false, and by rejecting that part, a perfect description remains, such a false part should be rejected and the deed held good.

*Rule 8:* Where land is described as running a certain distance by measure to a known line, that line will control the measure and determine the extent of the grant.

*Rule 9:* Where a patent calls for unmarked lines of surrounding surveys, the position of which can be accurately ascertained, and there is no evidence as to how the survey was actually made such unmarked lines will prevail over courses and distances, in case of a conflict. (This rule does not hold if the line is obscure, not definitely fixed, marked or known, and therefore likely to be looked upon by the parties as less certain than the measure given.)

*Rule 10:* Where the description in a deed calls for land "owned and occupied," the actual line of occupation is a material call to be considered in locating the lines of the land bounded therein.

*Rule 11:* The use of the term "about" indicates that exact precision is not intended, rather where nothing more certain can be found to control the course and distance. Subsequently, the grantee is limited to the exact course and distance given.

*Rule 12:* Where lines are laid down on a map or plan, and are referred to in a conveyance of land, the courses, distances, and other particulars appearing on such plan are to be as much considered the true description of the land conveyed as they would if expressly recited in the deed.

*Rule 13:* In construing the description in a deed which bounds the land conveyed upon a street, river, or other monument having width, courts incline strongly to such an interpretation of the language as will carry the fee of the land to the center line of such monument, rather than to its edge only.

*Rule 14:* When land is bounded on a private way, it extends only to the side line of the way.

*Rule 15:* Where a survey is referred to in a deed for greater certainty, it legally forms a part of it and both should be construed together.

*Rule 16:* Extrinsic evidence is admissible only when required to explain the conditions existing as of the date



of a deed or to resolve an extrinsic ambiguity in the deed.

*Rule 17:* The mention of quantity of acres after a definite description by metes and bounds, or by the aliquot part of the section, is a matter of description only, and quantity being the least certain, does not control.

*Rule 18:* Where boundaries are doubtful, then quantity (area) often becomes a controlling consideration.

### INFORMING THE CLIENT

All interested parties should be apprised of the results of the survey by virtue of a plat or map of the survey and an accompanying written report. All facts affecting the boundary and future development of the land should be included, along with notes of all variations from the descriptions found as a result of the survey. The report should address gores, gaps, and overlaps if any exist. Issues of encroachments, easements, rights of way, and known violations of any restrictions or regulations should be addressed as well. In general, consider in this report anything affecting property location that might affect the future development.

### Prepare the Final Map and Description

A final plat or map should always be prepared. The ALTA/ACSM Minimum Standards and most state regulations require such a document. This plat provides a means of formalizing the determinations made from the survey. It should be prepared while the work is still fresh in the minds of those completing the survey and computations. If this work is delayed, you may omit information that would immediately come to mind if the work had just been completed. Eventually your client, his attorney, his lender or a fellow surveyor will require information from the survey.

It is good policy for the person preparing descriptions to sign them. Do not sign descriptions drawn from information that was not a result of a survey made by the signer unless the source of the information used in preparing the description is clearly identified. Reference descriptions to the map used in their preparation.

### SUMMARY

A sample final map is included to provide the reader an example of a boundary survey where land development will follow. Later chapters will include detail computations that

will also relate to this survey. To bring continuity to the land development process, illustrations will be based on this same subject property whenever possible.

### REFERENCES

- Breed, Charles B., and George L. Hosmer. 1908. *The Principles and Practice of Surveying*, 3d ed. John Wiley & Sons, New York.
- Brown, Curtis M. 1969. *Boundary Control and Legal Principles*, 2d ed., John Wiley & Sons, New York [contributions by H. Frederick Landgraf and François D. Uzes].
- Brown, Curtis M., Walter G. Robillard, and Donald A. Wilson. 1981. *Evidence and Procedures for Boundary Location*, John Wiley & Sons, New York.
- Willing v. Booker*, 160 Virginia 461 (1933).
- Bureau of Land Management. 1973 *Manual of Instruction for the Survey of the Public Lands of the United States*, Tech Bulletin 6, U.S. Government Printing Office, Washington, D.C.
- Powell, Richard R. 1987. *The Law of Real Property*, ed. Patrick J. Rohan, Matthew Bender & Co., New York.
- Wattles, Gurdon H. 1976. *Writing Legal Descriptions*, Parker & Son.

### ADDITIONAL READINGS

- Armstrong v. Daily*, 514 S. 2d 993 (1988).
- Ayers v. Watson*, 137 S.C.R. 584 (1890).
- Burby, William E., *Handbook of the Law of Real Property*, West, St. Paul, 1965.
- Flick v. Kramer*, 151 Ill. App. R. 3d 836 (1987).
- Gage v. Davis*, 104 Idaho 48 (1983).
- Galt v. Willingham*, 11 F.2d 757 (1926).
- Re Gielle and Wife*, 373 S. 2d (1979).
- Gilardi v. Hallam*, 636 P. 2d 588 (1982).
- Hale v. Ball*, 70 Wash. 435 (1913).
- "Historical Note," *West's Annotated California Codes*, Sections 725–900, 291, West, St. Paul, 1980.
- Hodgman, F. *A Manual of Land Surveying*, F. Hodgman Co., Climax, 1913.
- Humphrey v. Futter*, 169 Cal. 3d 333 (1985).
- Klinhart v. Mueller*, 166 S.W. 2d 519, S.W. 2d (1943).
- Piccirilli v. Alfred Groccia*, vol. 327 A. 2d 834 (1975).
- Robillard, Walter G., and Lane J. Bouman. *A Treatise on the Law of Surveying and Boundaries*, 5th ed., Michie, Charlottesville, 1987.
- Ruick v. Twarkins*, 171 Conn. R. 149 (1976).
- Slaughter v. Atlanta Coca Cola Bottling Co.*, 172 S.E. 723 (1934).

## CHAPTER 14

# CONTROL SURVEYS

Marc S. Cheves, L.S.

### INTRODUCTION

The backbone of any land development project is the control network. From inception to completion, the success of a project depends on accurate horizontal and vertical control. Project designs, as well as staking for construction, are based upon the boundaries and topography established by the survey of the property. If the accuracy of the horizontal and vertical control is poor, the consequences can be disastrous.

Because all measurements contain a certain amount of error, surveyors must apply corrections and adjustments to remove the inevitable misclosures in traverses and level loops in their survey measurements. Such adjustments allow surveyors to evenly distribute these inevitable errors found in survey measurements. The accuracy of the work is completely dependent upon how the surveyor deals with these errors in measurement. One method of evaluating the quality of a surveyor's work is to compare it to industry standards, which are based on detailed specifications. Such specifications also give surveyors the means to evaluate the quality of their survey equipment.

If the accuracy of the equipment is known, the adjustment of horizontal and vertical control work can be properly performed. If not, adjustments are applied proportionally or by a surveyor's intuition or experience. While these methods do not really provide a reasonable explanation for the manner in which closure errors are distributed, they are still widely used. Many surveyors are unaware of the relationship among standards, specifications, and adjustments. This chapter will demonstrate this relationship by covering the traditional methods and techniques used to make adjustments, as well as illustrating how standards and

specifications control the proper adjustment of fieldwork. This chapter will also cover the development of standards and specifications, errors and their sources, and the application of adjustments to account for errors. A short discussion on probability theory and the statistical aspects of survey measurements and equipment is also required to indicate how these three topics relate to one another. At the end, a method of adjustment will be discussed that will allow the land development surveyor to incorporate standards and specifications, errors, and adjustments to provide the best estimate of the true position of the objects being located.

### SPECIFICATIONS

#### USC&GS Third Order

In order to adequately evaluate and judge the quality of survey work, a set of standards must be developed. Standards are statements of required accuracies of survey measurements, such as traverse for horizontal measurements or level loop closures for vertical measurements. Specifications detail the necessary steps to achieve the required standard, such as the number of angle repetitions or the quality of the instrument. The National Geodetic Survey (NGS), formerly known as the U.S. Coast & Geodetic Survey (USC&GS), implemented the most widely accepted set of standards of accuracy and specifications to support these survey classifications. The NGS is a division of the National Oceanic and Atmospheric Administration (NOAA), which is part of the U.S. Department of Commerce.

Issued by the Federal Geodetic Control Subcommittee (FGCS), the *Standards and Specifications for Geodetic Control Networks* (September 1984, reprinted October 1990) in-

cludes the familiar Third-order, class I and II classification, which is now the basis for many civilian survey specifications. Even though this document still uses ratios (1:10,000, etc.)<sup>1</sup> to classify the different orders of surveys, the Global Positioning System (GPS) and the widespread availability of least squares adjustment software have caused NGS to consider the classification of surveys by positional tolerance.

As the name implies, this document was created primarily to support the development of geodetic control networks. However, previous editions of these standards and specifications state that the third-order requirements were designed for use on local mapping and engineering projects. To the land development surveyor, the specifications required for angular, linear, and vertical closure are of great importance.

These specifications state that the maximum angular closure for third order class I shall be  $10''\sqrt{N}$  and class II shall be  $12''\sqrt{N}$ , where  $N$  equals the number of legs in a traverse. Further, minimum position closures for third order are to be no less than 1:10,000 for class I and no less than 1:5,000 for class II. For level loop closures, the publication states that the third order accuracy requirement is  $12 \text{ mm}\sqrt{E}$ , where  $E$  equals the length of the level loop in kilometers. The vertical closure requirement can also be stated as  $0.05 \text{ feet}\sqrt{M}$ , where  $M$  equals the length of the level loop in miles.

The reader is encouraged to obtain copies of the previous editions, including *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (February 1974, reprinted January 1976) and *Specifications to Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (July 1975, reprinted May 1978) for supplemental information. These versions provide valuable procedural guidelines and specification details pertaining to the establishment of horizontal and vertical control. Tables from the 1974 and 1984 documents are included at the end of the chapter. Copies of these documents are available from NGS in Silver Spring, Maryland (301-713-3242).

For land development projects, unless the state board of registration has developed more rigorous minimum standards, the commonly implied standard is third order. Work performed for state or local government agencies is usually subject to specifications similar to third order. The exception to this is that for most land development projects, the objective is to make the civil improvements to the site operate properly. In other words, the objective is for the storm drain system to carry the runoff to the desired place, the sanitary sewer system to do likewise, and the curb and gutter for streets and parking lots to drain properly. In this sense, most land development surveying is performed to construction order. It is important to remember that it is

wasteful to perform fieldwork in support of land development to a higher order of accuracy than required.

### First and Second Order

For more accurate work, the USC&GS developed the first order and second order standards and specifications. The triangulation network that covered the United States was composed primarily of first- and, to a lesser extent, second-order stations. Previously, this network was based on the North American Datum of 1927 (NAD 27). NAD 27 contained significant relative positional errors because of the distortions caused by the additions of new observations and because these positions were developed mathematically, that is, by triangulation, and not by distance measurement. These observations affected the network similar to the way cross-loops can affect a traverse. Due to complaints from surveyors attempting to fit classical terrestrial surveys into the network, NGS decided to perform a massive readjustment of the NAD 27 to ensure that, relative to each other, all stations fit into the network. Known as the NAD 83, this readjustment effectively removed most of these distortions. However, because GPS is so accurate, small errors resulting from the readjustment have since been revealed. NGS then embarked on the creation of the High Accuracy Reference Networks (HARN) in response to continued complaints about the degradation of accuracy resulting from forcing GPS surveys into the NAD 83 network. HARNs are based on high-accuracy GPS stations established by NGS and have been created for many of the states. Surveyors using GPS can now tie their work to the HARN stations with assurance that the accuracy of their work will not become degraded.

A similar federal network exists for vertical control. Previously known as the National Geodetic Vertical Datum of 1929 (NGVD 29), it too has been readjusted and is now known as NGVD 88. Because this network does not rely on computed values in the same manner as the horizontal network, it is not subject to the same magnitude of relative errors due to the more stringent accuracy standards set for vertical work and the fact that vertical errors are much easier to detect.

### Global Positioning System (GPS)

The FGCS has developed the *Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques*. It created new survey categories, including AA, A, and B order. The corresponding relative position accuracies for the new orders are 1:100,000,000, 1:10,000,000, and 1:1,000,000. The traditional first, second, and third orders are now included in the C group in the FGCS document (as C1, C2-I and C2-II, and C3) with the corresponding accuracies being 1:100,000, 1:50,000 (or 1:20,000), and 1:10,000. As the name suggests, this standard and specification applies more to geodetic surveying and has no real application in GPS surveying of land development projects. Nonetheless, the GPS surveyor should consider these requirements in the design, execution, and adjustment of GPS work to support land development. The

<sup>1</sup> The ratio 1:10,000 means that for every 10,000 units of horizontal measurement, the tolerance or error will be no greater than 1 unit.

land development surveyor will most likely encounter this standard and specification if he or she is involved in the establishment of geodetic control for the creation of a Geographic Information System (GIS). As more local governments create GISs, the land development surveyor will use the horizontal and vertical control created by these systems. Some local jurisdictions have created specifications for the accuracy of ties to this type of control. Eventually, almost all land development projects will make use of GIS horizontal and vertical control, particularly in metropolitan areas.

In November 1997, NGS issued NOAA Technical Memorandum NOS NGS-58, *Guidelines for Establishing GPS-Derived Ellipsoidal Heights (Standards: 2cm and 5cm)*, Version 4.3. Authored by David B. Zilkoski, Joseph D. D'Onofrio, and Stephen J. Frakes, the guidelines established procedures for using GPS to obtain ellipsoidal heights at the 2-sigma (95% confidence) level. Not to be confused with orthometric heights, which are what is used in land development, the guidelines are based on extensive research by NGS, and require specific observation techniques. To obtain 2-cm accuracies, users are required to use dual-frequency receivers and fixed-height tripods, make certain ties to A- or B-order stations, make observations of a certain length in time, and set up the network such that baseline lengths do not exceed certain maximums. Furthermore, multiple sets of observations are required at different times during the day (twice, on two different days, to benefit from changes in satellite geometry). Other requirements include gathering meteorological data, setting the elevation mask on the receiver to  $10^\circ$ , and ensuring that VDOP values for the satellite constellation are within certain limits before occupation of the stations. Certain loosening of the specifications are allowed to obtain 5-cm accuracies. At the time of the publication of this Handbook, guidelines for obtaining GPS-derived orthometric heights from the ellipsoid heights had been developed but not yet published. Once these guidelines are released, the land development surveyor will be one step closer to using GPS to obtain the necessary vertical accuracies needed for land development work.

The use of GPS has caused a rethinking of control network concepts. NGS has discovered that with the addition of the Continuously Operating Reference Stations (CORS) into the National Spatial Reference System (NSRS), long-range horizontal accuracies in the 1- to 2-cm range are possible. NGS has also found that vertical accuracies in the two- to 4-cm range are possible, but require 12- and 24-hour observation times, respectively.

In 1998, the Federal Geodetic Control Subcommittee (FGCS) and the Federal Geographic Data Committee (FGDC) issued the Geospatial Positioning Accuracy Standards. More information can be found at <http://www.fgdc.gov/standards/status/textstatus.html>. This document represents a departure from the classification of control points based on the traditional order and class schema. Another change involves a move away from determining accuracy based on distance. For example, under the old

order and class system of classification, the accuracy of the relationship between two first-order horizontal points was determined by dividing the distance between them by 100,000. The accuracy of different types of geodetic spatial data was determined using different methodologies, such as classical horizontal (triangulation and traverse) and classical vertical (leveling). But NGS found that GPS accuracy does not degrade rapidly with distance. This means the accuracy between unconnected points between which a direct measurement had not been made is not affected as dramatically with GPS as with traditional, ground-based surveying methods. However, it is still good surveying practice to connect points directly that are very close to one another.

Another new concept is that of *local* accuracy and *network* accuracy. NGS defines the local accuracy of a control point as a number, expressed in centimeters, that represents the uncertainty at the 95% confidence level in the coordinates of the point relative to the coordinates of other directly connected, adjacent control points. The reported local accuracy will be an approximate average between the point and other adjacent points. Network accuracy is defined in the same terms, but with respect to the geodetic datum, in this case the CORS stations, are assumed to have no (or practically no) positioning error. NGS is in the process of converting all accuracy statements using this new concept.

Using first-order points as an example, network accuracy in NAD 27 was approximately 10 m, while local accuracy was 1 part in 100,000. With NAD 83, network accuracy was reduced to 1 m while local accuracy remained the same. But with the addition of GPS into the High Accuracy Reference Networks (HARN), the network accuracy improved to a tenth of a meter, and local accuracy for B-order marks improved to 1 part in a million while A-order marks improved to 1 in 10 million. Finally, with CORS, *both* network and local accuracies have improved to 1 cm.

Both standards are relative accuracy measures. Ellipsoidal/orthometric heights are also included in the new standards. The difference between horizontal control, discussed above, and vertical control is that horizontal accuracy will be expressed as 2D circular error, vertical control accuracy will be expressed as 1D linear error, and both will be expressed at a 95% confidence level. Additionally, these standards will meet the needs of GIS users. Once the new standards are totally implemented, the horizontal, ellipsoidal height, and orthometric height accuracy standards will be based on accuracy classifications (at the 95 percent confidence level) expressed in linear, terrestrial, units. The classification of geodetic control points by order and class methodology, based on distance, will gradually fade away.

### ALTA/ACSM Standards

For commercial property, another standard exists. Many times a lender is called upon to make a loan on property that is across the country. Furthermore, because there is such wide variation in the standard of surveying between different locales, clients had no way of knowing the degree

of accuracy of the survey work. Members of the American Land Title Association (ALTA), in cooperation with the American Congress on Surveying and Mapping (ACSM), realized that no national standard existed to judge the accuracy of land title surveys. So, in 1962, the ALTA/ACSM *Minimum Standard Detail Requirements for Land Title Surveys* were developed. Until recently, this type of survey was seldom used for raw land. However, as its use is increasing, an increasing number of lenders are requiring that these standards be followed when surveying vacant land. Furthermore, some state boards of registration have incorporated the ALTA/ACSM Standards into the accuracy requirements of their Minimum Standards.

The ALTA/ACSM standard has undergone a number of updates, starting in 1986, and most recently in 1999. The original rationale behind these standards was that the title insurance industry needed to be held responsible for the legal aspects of accuracy standards while the surveying community would be responsible for the surveying and accuracy aspects. The title industry has generally not concerned itself with the accuracy portion of the standards but instead has relied on ACSM to develop them. With the 1992 changes, ALTA and ACSM separated the survey accuracy specifications from the rest of the standards to allow for the adoption of necessary updates without having to revise and re-adopt the entire document. That portion of the standard is now called the Accuracy Standards for ALTA/ACSM Land Title Surveys, and an excerpt appears in Figure 14.1.

The 1962 ALTA/ACSM Standards called for the surveyor to make a statement concerning the positional accuracy of the property markers he or she had set. This requirement was widely ignored, primarily because it was extremely difficult and expensive to meet these standards while performing a survey using the equipment and procedures typically employed at the time. As a result, in 1986, the positional accuracy portion was replaced with a controversial set of specifications that outlined the exact procedures and types of equipment that were to be used in performing an ALTA/ACSM Land Title Survey. The controversy generated by these very specific requirements left little or no room for professional judgment by the surveyor. Also, although the requirements had a sound statistical basis, they did not provide any accommodation for the rapid improvements in equipment that were occurring even as the 1986 standards were being developed. These requirements were widely ignored or situationally applied because surveyors wanted leeway in applying what they felt were sound procedures and because surveyors were acquiring equipment that allowed them to measure more accurately than was required by these standards.

In the most recent version of the ALTA/ACSM Standards (1999), the organizations have reverted to a positional accuracy standard. This new standard allows the surveyor to employ appropriate procedures and equipment as long as a maximum allowable amount of positional uncertainty in

any corner location is not exceeded. The surveyor should perform a rigorous mathematical adjustment of the survey observations to ensure compliance with the standard. The terms of this document will be defined and further discussed under Types and Sources of Error.

### National Map Accuracy Standards

A third standard that the land development surveyor is likely to encounter is the National Map Accuracy Standards. These standards, developed primarily for the U.S. Geological Survey topographic quadrangle maps, specify the horizontal and vertical accuracy of the information presented on such maps. They are also commonly applied to maps prepared by photogrammetry. The surveyor's role in photogrammetry is the establishment of horizontal and vertical control to allow the photogrammetrist to rectify the aerial photographs from which the maps are prepared.

One of the National Map Accuracy Standards specifications states that 90% of the contours shown on the map will be within one-half a contour interval of their true position. Furthermore, the standard states that 90% of spot elevations on the map shall be within  $\pm 20\%$  of the contour interval. This means that for 5-ft contour mapping, 90% of all spot elevations shall be within  $\pm 1$  ft of their true vertical value. For 2-ft contour mapping, 90% of all spot elevations shall be within  $\pm 0.4$  ft. The use of GPS has become more prevalent for the establishment of the horizontal and vertical control for aerial mapping projects. With recent advances and by following certain guidelines, NGS feels that current vertical accuracies for GPS, under ideal conditions, are on the order of 2 to 5 cm. For land development surveying, the advent of real time kinematic (RTK) techniques makes GPS an ideal tool for data gathering and construction layout.

### Equipment Specifications

Angle and distance measuring devices are now evaluated on the basis of their accuracy and precision. Specifications created by equipment manufacturers commonly list the accuracy, as in the case of an angle-turning instrument, being reported as  $\pm x$  number of seconds. For EDMs, the accuracy is defined as being  $\pm x$  number of millimeters plus  $x$  parts per million (PPM). For example, a 1-km measurement made with the most commonly used EDM ( $\pm 5$  mm + 5 PPM) will contain a plus or minus uncertainty of 10 mm. Likewise, the accuracy of levels is listed as being able to achieve a closure of  $x$  times the length of a level run. These  $\pm$  amounts are supplied by the manufacturers and are the result of rigorous statistical testing involving thousands of measurements. For land development work, most surveyors use theodolites having an accuracy specification of  $\pm 5$  or 6 seconds and EDMs having an accuracy specification of  $\pm 5$  mm plus 5 PPM. Some surveyors will use more accurate instruments for the control network, like a "one-second" Wild T2, but "six-second" and "five-second" instruments are more commonly used for everyday land

## CLASSIFICATIONS OF ALTA-ACSM LAND TITLE SURVEYS

### Introduction

The degree of precision and accuracy necessary for a particular cadastral survey should be based on the intended use of the land without regard to its present use, provided the surveyor has knowledge of the intended use. If the surveyor has no such knowledge, the degree of precision may be based on the present use of the land.

Four general survey classes are defined using various state regulations and accepted practices. These general classes are listed and defined below.

### Survey Classes By Land Use

#### URBAN SURVEYS

Surveys of land lying within or adjoining a city or town. This would also include the surveys of commercial and industrial properties, condominiums, townhouses, apartments and other multi-unit developments, regardless of geographic location.

#### SUBURBAN SURVEYS

Surveys of land lying outside urban areas. This land is used almost exclusively for single family residential use or residential subdivisions.

#### RURAL SURVEYS

Surveys of land such as farms and other undeveloped land outside the suburban areas which may have a potential for future development.

#### MOUNTAIN and MARSHLAND SURVEYS

Surveys of lands which normally lie in remote areas with difficult terrain and usually have limited potential for development. Should these above cited specifications be in conflict with state laws, rules or regulations, the more stringent requirements must be followed.

The combined precision of a survey can be statistically assured by dictating a combination of survey closure and specified procedures for a particular survey class. ACSM and ALTA have adopted specific procedures for control surveys in order to assure the combined precision of a particular survey class. The statistical base for these specifications is on file at ACSM and available for inspection. The surveyor shall employ, in his or her judgement, proper field procedures, instrumentation and adequate survey personnel in order to achieve accuracies comparable to those adopted by ACSM for a designated class of survey.

## AMERICAN CONGRESS ON SURVEYING and MAPPING

### MINIMUM ANGLE, DISTANCE and CLOSURE REQUIREMENTS FOR CLASSES OF SURVEYS

(1)

SURVEY CLASS	DIR. READING OF INSTRUMENT (2)	INSTRUMENT READING ESTIMATED (3)	NUMBER OF OBSERVATIONS PER STATION (4)	SPREAD FROM MEAN OF D&R NOT TO EXCEED (5)	ANGLE CLOSURE WHERE N = NO. OF STATIONS NOT TO EXCEED	LINEAR CLOSURE (6)	DISTANCE MEASUREMENT (7)	MINIMUM LENGTH OF MEASUREMENTS (8), (9), (10)
URBAN	20° < 1' > 10"	5° < 0.1' > N.A.	2 D&R	5° < 0.1' > 5"	10° √N	1:15,000	EDM or Doubletape with steel tape	(8) 81m, (9) 153m (10) 20m
SUBURBAN	20° < 1' > 10"	10° < 0.1' > N.A.	2 D&R	10° < 0.2' > 10"	15° √N	1:10,000	EDM or steel tape	(8) 54m, (9) 102m (10) 14m
RURAL	20° < 1' > 20"	N.A.	1 D&R	20° < 0.3' > 20"	20° √N	1:7,500	EDM or steel tape	(8) 40m, (9) 76m (10) 10m
MOUNTAIN/MARSHLAND	1° < 1' > 1"	N.A.	1 D&R	30° < 0.5' > 30"	30° √N	1:5,000	EDM or steel tape	(8) 27m, (9) 51m (10) 7m

Note (1) All requirements of each class must be satisfied in order to qualify for that particular class of survey. The use of a more precise instrument does not change the other requirements, such as number of angles turned, etc.

Note (2) Instrument must have a direct reading of at least the amount specified (not an estimated reading), i.e.: 10° = Micrometer reading theodolite, < 1' > = Scale reading theodolite, 10" = Electronic reading theodolite, 20" = Micrometer reading theodolite, or a vernier reading transit.

Note (3) Instrument must have the capability of allowing an estimated reading below the direct reading to the specified reading.

Note (4) D & R means the Direct and Reverse positions of the instrument telescope, i.e., Urban Surveys require that two angles in the direct and two angles in the reverse position be measured and meaned.

Note (5) Any angle measured that exceeds the specified amount from the mean must be rejected and the set of angles re-measured.

Note (6) Ratio of closure after angles are balanced and closure calculated.

Note (7) All distance measurements must be made with a property calibrated EDM or Steel tape, applying atmospheric, temperature, sag, tension, slope, scale factor and sea level corrections as necessary.

Note (8) EDM having an error of 5mm, independent of distance measured (Manufacturer's specifications)

Note (9) EDM having an error of 10mm, independent of distance measured (Manufacturer's specifications)

Note (10) Calibrated steel tape.

FIGURE 14.1 Classifications of ALTA-ACSM land title surveys and minimum angle, distance, and closure requirements for classes of surveys. (Courtesy of ACSM)

development surveying. Levels that are capable of achieving a closure of  $\pm 0.01$  ft in 1 mile of double run levels are commonly used by today's surveyors.

It is a widely held misconception that the manufacturer's accuracy specification for a theodolite refers to the plus or minus uncertainty in *each angle*. This is not true. The *de facto* specification for instrument accuracy is the so-called DIN spec. Referred to as DIN 18723 and developed in Germany by the Deutsches Institut für Normung, this method of evaluation refers to the plus or minus uncertainty in each *direction* or *pointing*, a direction being composed of a direct and reverse pointing to a single target. An angle is considered to be the difference between two directions or pointings. The differences between these two terms and the DIN spec will be discussed further under Types and Sources of Error.

### ACCURACY & PRECISION

Before we can discuss the true significance and meaning of specifications like third order and the ALTA/ACSM standards, we must first define some relevant terminology, such as accuracy and precision. *Accuracy* is defined as "being close to the truth." In other words, for something to be accurate or to possess accuracy, the user or recipient of the work needs assurance that the product is true or correct. Another frequently used term in discussing specifications is *precision*, defined in a surveying sense as "being able to repeat a measurement over and over again and being able to obtain close to the same answer for each one." The classic example is that of a rifleman and a bull's-eye target. With reference to Figure 14.2, if the rifleman is able to cluster his shots together at the edge of the target, the results are said to be precise but not accurate. In other words, he was able to shoot close to the same mark but not near the center of the target. On the other hand, if his shots were spread out all over the target, the results would be neither accurate nor precise. An exception would exist if the mean of the shots were in the center of the target, in which case his shots would be considered to be accurate but not precise. If, however, his shots were clustered around the center of the target, the results would be said to be both accurate and precise.

Relating the above to survey measurements, if a theodolite or EDM is accurate it will allow the operator to turn better angles or measure better distances such that the measured quantity is hopefully close to the true site conditions. If the operator has a good eye, turns the angles carefully, and controls any systematic errors, that operator is described as having the capability of producing accurate angles. So-called reading and pointing errors can contribute a great deal of inaccuracy. Any errors caused by the estimation needed to read an angle on an optical theodolite can often contribute more inaccuracy than pointing errors. The precision of an instrument is classified not only by its ability to accurately turn angles, but by its ability to turn accurate angles repeatedly. The land development surveyor will en-

counter survey standards and specifications in two forms. The most familiar is the standard that dictates horizontal and vertical closures of a traverse or a level run, while the less familiar is the process of establishing the accuracy of a piece of equipment (see Types and Sources of Error for more detail). The discussion of precision and accuracy will play a part in our further discussion of standards and specifications and adjustments.

### TYPES AND SOURCES OF ERROR

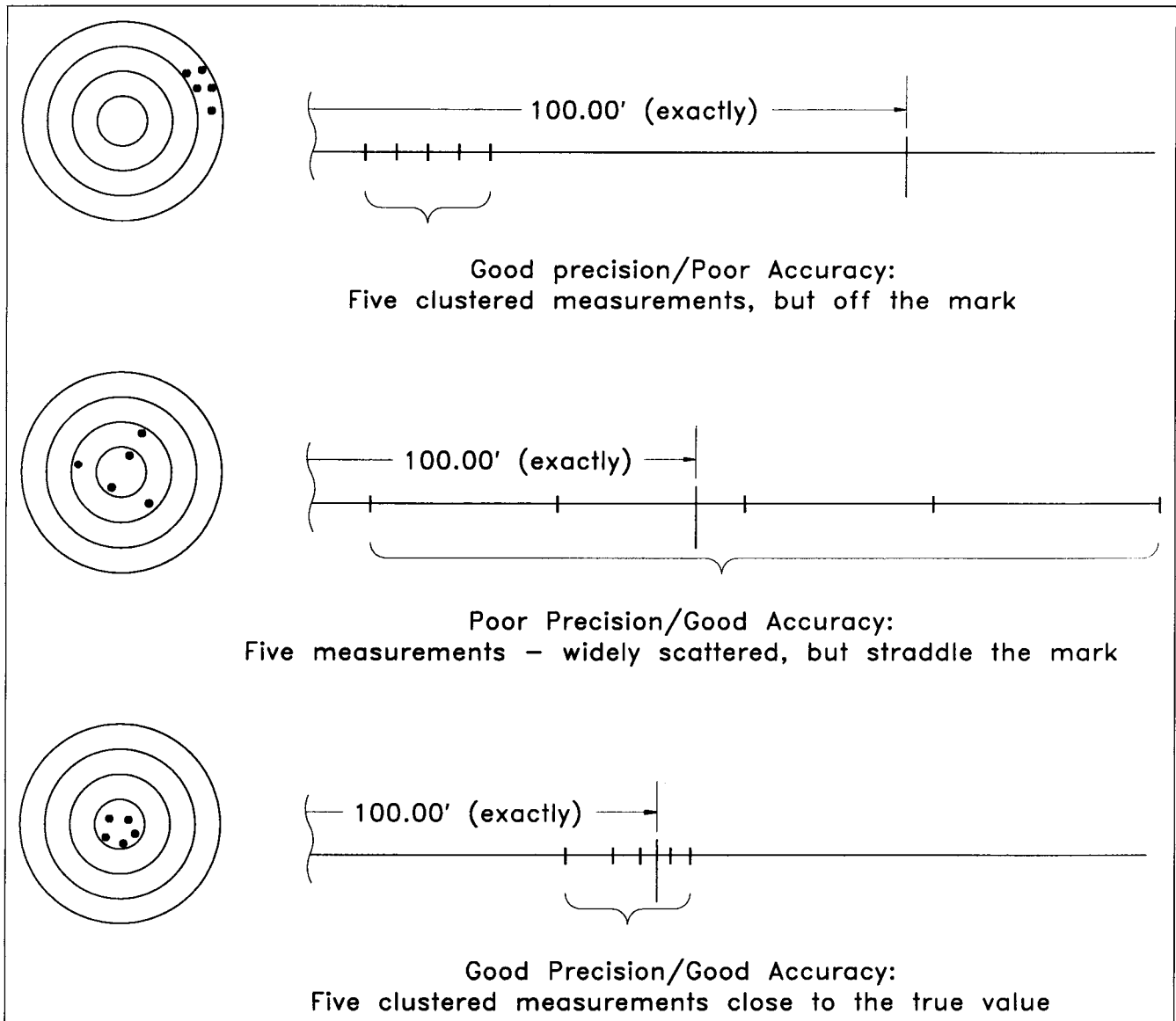
As mentioned previously, all measurements contain errors. The first type of error is the *blunder* or *mistake*. An example of a blunder would be recording an angle as  $29^\circ$  instead of the instrument reading of  $92^\circ$ . For purposes of our discussion, it is assumed that the surveyor has eliminated the outright mistakes or blunders in the work. The other errors that will be dealt with in this chapter fall into two categories: systematic (both equipment and natural) and *random* or *accidental*.

#### Systematic Errors

Systematic errors are considered either *constant* or *variable*. Constant systematic errors can, to a certain extent, be corrected. Examples of constant systematic equipment errors are chains that measure too short or too long or instruments and EDMs that are not properly calibrated. These errors generally have the same magnitude and the same algebraic sign. Using a chain that is too short will consistently provide distance measurements that are too long. An EDM that is not accurately calibrated will consistently measure distances incorrectly, i.e., short or long. Temperature corrections for measured distances (chained and EDM) would be applied to eliminate systematic errors. Other sources of constant systematic error include prism offsets, adjustment of prism pole, and optical plummet adjustment. Equipment errors can normally be corrected by accurate calibration or by following the proper methods and procedures. Differential heating of a tripod by sunlight, movement of a tripod or instrument due to wind or vibration, and tripod settling are all variable systematic errors that can be alleviated by following proper methods and procedures. An example of such a procedure would be to perform observations over a short period of time to eliminate the possibility of differential tripod heating.

#### Natural Errors

Natural errors include such things as temperature, barometric pressure, humidity, and refraction. Surveyors are familiar with the correction for the first two because the PPM setting on an EDM is dependent upon temperature and barometric pressure. Of the variable errors, refraction is probably the single greatest source of angular error. It is affected by humidity, and it is hard to detect and even harder to predict. However, it can be avoided or reduced by making observations over a longer period of time and by being conscious of how it can affect angular measurements. In peri-



**FIGURE 14.2** Accuracy and precision for a bull's-eye and the relationship to measurements.

ods of low or no wind, care must be exercised due to the creation of air bubbles, or air strata of differing density. Air strata are generally of greater density near the ground and lie roughly parallel with it. Over sloping terrain these strata of different densities are not horizontal and a ray of light passing through them will be bent horizontally as well as vertically. A line of sight passing near a building or a tree may suffer a change in horizontal direction. Likewise, a line of sight passing close to the ground may suffer a change in vertical direction. The nearer the cause of the disturbance to the observer, the greater the angular distortion. A line of sight that passes over cool areas and then hot areas will bend. Similarly, a line that passes over a body of water is likely to incur horizontal refraction. Refraction will cause a light ray to bend away from low density and towards high density, away from lower pressure and towards higher pres-

sure, and away from lower temperature and towards higher temperature. Making observations at night can alleviate refraction caused by heatwaves (shimmer).

### Random Errors

Most random errors are caused by a surveyor's inability to aim an instrument correctly, read angles or measure distances accurately. Unlike systematic errors, which can be corrected, random errors have magnitudes and signs that are unpredictable. The theory of probability states that random errors tend both to be small and to distribute themselves equally on both sides of the true value. That is, random errors will manifest themselves as observations that are greater (plus) and lesser (minus) than the true value. It can be shown that, given a sufficient number of observations, random errors will follow predictable mathematical



rules. These errors can be plotted on a graph and will normally fall within the familiar bell-shaped curve, known as a *probability curve*. If the observations are plotted on a bar graph, known as a *histogram*, the resulting bars from the graph will approximate the probability curve (see Figure 14.3). Properties of this curve, also known as the *normal distribution curve*, can be described by mathematical parameters.

### Probability Theory

The average, or *most probable value*, of these readings is obtained by simply adding them up and dividing by the number of readings. This is known as the *arithmetic mean*. After systematic errors and blunders are removed, the arithmetic mean is accepted as being the most probable (or true) value. If the readings represent a valid sample, the observations will fall on both sides of the mean. That is, some will be greater than the mean and some will be less. The variances, i.e., the mathematical difference between the measured value and the mean, are known as the *residuals* (see Figure 14.4).

The term used to define the plus or minus uncertainty in a measurement is *standard deviation* or *standard error*. These terms are used to express the accuracy of a group of measurements or the accuracy of a piece of equipment. They are used for two reasons: (1) to analyze measurements to see if they meet the specification requirements and (2) to establish procedures and specifications to ensure that the work will meet the anticipated standard.

The equation for the standard error is

$$\sigma = \pm \sqrt{\frac{\sum v^2}{n-1}} \quad (14.1)$$

where  $\sigma$  equals standard error,  $\sum v^2$  equals the sum of the residuals squared, and  $n - 1$  equals the number of observations minus one. It can be said, statistically, that any one observation has a 68.3% chance of falling within the range of the mean  $\pm \sigma$ . To illustrate this principle we will use the following example.

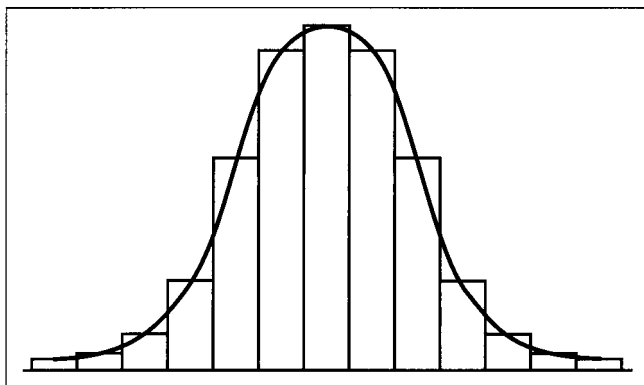


FIGURE 14.3 Histogram.

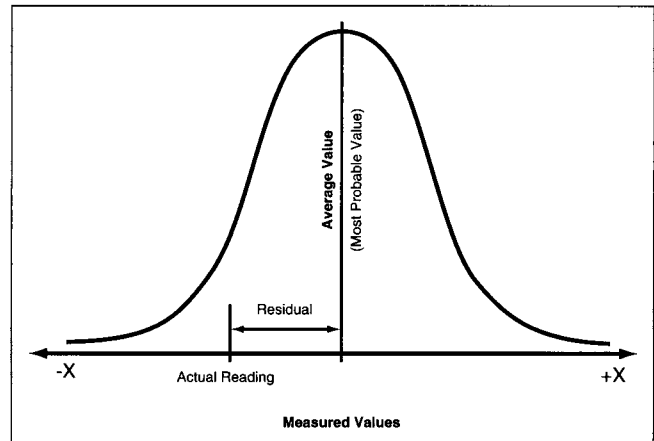


FIGURE 14.4 The residual is the difference between the most probable value and the actual reading.

**EXAMPLE** Twenty-five reading and pointing observations made with a Wild T-2 (all readings are in seconds) are listed in Table 14.1. In the example, the term for the readings is  $x$  and the term  $\sum x$  is the sum of all the measurements and  $n$  is the total number of measurements.

The first step is to determine the mean by dividing the sum of the readings by the total number, or in this case,  $\sum x = 1130.6/25 = 45.2$ . The second step is to determine the residuals by subtracting each reading from the mean, observing the signs. For example, the residual for observation #1 would be  $45.2 - (+46.7) = +1.5$ . The third step is to square each of the residuals and then sum the squared values. For the example,  $\sum v^2$  equals 41.88. By inserting these values into the formula, it can be seen that the standard error for any single observation equals  $\pm 1.32$  seconds. It is important to remember that this formula applies to random errors and is true only if all systematic errors and blunders have been removed from the sample. It can be seen that 17 (or 68%) of the observations fall within this range. Whereas the formula for computing standard error applies to a single observation, the following formula can be used to determine the *standard error of the mean*.

$$\sigma_m = \pm \frac{\sigma}{\sqrt{n}} \quad (14.2)$$

For the example, it can be seen that although the standard error for a single observation is  $\pm 1.32$  seconds, the standard error of the mean is  $\pm 0.26$  seconds. This demonstrates the principle that multiple observations should produce a value that is closer to the truth. (Note: Some textbooks define standard deviation by Eq. (14.1) and standard error by Eq. (14.2). For purposes of this discussion, standard deviation and standard error are assumed to be defined by Eq. (14.1) and standard error of the mean to be

**TABLE 14.1 Observations for Example 1**

Obs. #	$x$	$v$	$v^2$
1	46.7	+1.5	2.25
2	45.2	0.0	0.00
3	45.9	+0.7	0.49
4	45.2	0.0	0.00
5	46.2	+1.0	1.00
6	47.4	+2.2	4.84
7	43.0	-2.2	4.84
8	45.2	0.0	0.00
9	43.2	-2.0	4.00
10	44.2	-1.0	1.00
11	46.2	+1.0	1.00
12	43.9	-1.3	1.69
13	46.7	+1.5	2.25
14	43.0	-2.2	4.84
15	46.2	+1.0	1.00
16	44.1	-1.1	1.21
17	45.7	+0.5	0.25
18	46.2	+1.0	1.00
19	45.3	+0.1	0.01
20	44.3	-0.9	0.81
21	44.7	-0.5	0.25
22	45.7	+0.5	0.25
23	43.2	-2.0	4.00
24	47.3	+2.1	4.41
25	45.9	+0.7	0.49
	$\Sigma x = 1130.6$		$\Sigma v^2 = 41.88$

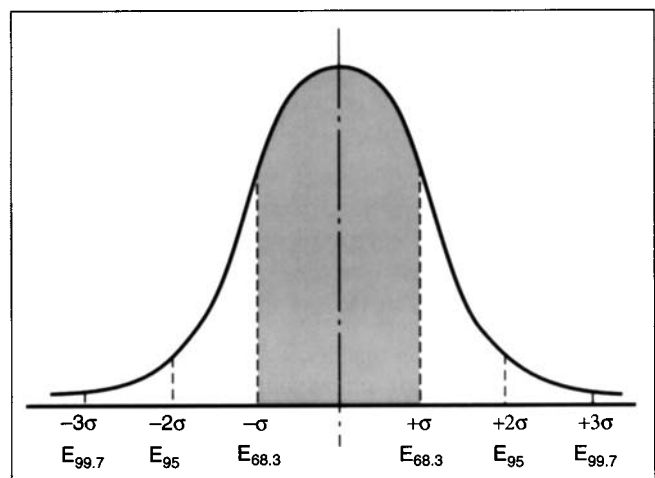
defined by Eq. (14.2).) The standard error of the mean can be used to select an instrument capable of producing the required accuracy. It can also be used to determine the number of observations (repetitions) that will be required to achieve a given certainty in the measurements. The establishment of specifications for a particular standard is be-

yond the scope of this chapter. For more information consult Buckner (1983). Although it would be unusual for twenty-five observations to be taken at a single setup, the example provides a good illustration of statistical analysis. Furthermore, the least squares adjustment to be discussed under Least Squares effectively treats all the measurements as a large sample and simultaneously applies the corrections for a best fit.

Figure 14.5 illustrates the different levels of probability. Even though the standard error derived by these formulas is expressed as being 68.3%, surveys generally require a higher degree of probability. Most surveys will require a certainty or confidence level of 95% or  $2\sigma$ , also known as 2 sigma. Depending on the circumstances, some surveys will require a 99% certainty ( $2.5\sigma$ ) or even 99.7% which is  $3\sigma$ , known as 3 sigma (see Figure 14.5).

### The Din Spec

Based on the knowledge of these specifications, instruments can be chosen that will meet the requirements. The specifications will take into account the accuracy and precision of a given instrument and specify the number of repetitions for each angle. Error budgets can be established that will give the surveyor an idea, based on statistical knowledge, of the type of instrument necessary to meet a certain specification and the procedures that must be followed to achieve the desired result. As mentioned earlier, the DIN spec relies on specific procedures, including the number of targets to be sighted and advancing the circle between sets, etc., to establish the accuracy of a given instrument. The result of these procedures is the standard error of the mean of a pair of face I (direct) and face II (reverse) readings to a single target. If the surveyor wishes to work with angles as opposed to directions, the manufacturer's stated standard error should be multiplied by  $\sqrt{2}$ . This is the recommended NGS procedure and is stipulated because angles are the un-correlated result of the subtraction of directions.



**FIGURE 14.5** Levels of probability.

### ACSM Classification and Specifications for Cadastral Surveys

The previously mentioned ALTA/ACSM Classification and Specifications for Cadastral Surveys were developed at the 99% ( $3\sigma$ ) confidence level and were based on an attempt to equate angular precision with distance measurement precision. For an urban class survey, the normal 1:15,000 traverse closure was multiplied by 3 to arrive at the  $3\sigma$  level and rounded off to 1:50,000. Furthermore, this specification dictates that no instrument with an estimated reading greater than 6 in. may be used. The specification takes into account instrument and target miscentering and reading and pointing errors. Obviously designed for optical instruments, the specification does not apply towards the use of total stations, which are, in effect, self-reading. Also included are similar error sources for EDMs. The document states that

Once the [required] closures were determined, angular and distance measurement precisions of equal weight were computed. Finally, with the instrumental precisions established, the procedures for observation were derived using normally distributed statistics. The number of positions for each type of instrument and survey class were determined by dividing the angular standard error root mean square by the square root of the number of repetitions required to reduce the standard error root mean square to a value equal to or less than the angular precision specified for each survey class.

As stated above, this specification has proved to be very controversial, but has a sound statistical basis if it can be agreed that these types of surveys should be performed at the 3 sigma (99.7%) level of accuracy.

### ADJUSTMENTS

As stated above, the inevitability of errors in surveying measurements requires a means to eliminate them. As we have seen, random errors should follow a predictable pattern and surveyors can use adjustments to remove these errors from the control network. If measurements are carefully made and if systematic errors are removed by calibrating the equipment and employing correct methods and procedures, the resulting random errors reveal themselves as angular and linear misclosures. If the misclosures are within tolerance, customary practice dictates that adjusting the original observations the removal of these misclosures will cause the work to mathematically close upon itself.

#### Horizontal Adjustments

Adjustments of this type are made by surveyors for any or all of the following three reasons:

1. To distribute errors so that the measurements will mathematically close
2. To arrive at a set of horizontal positions that are closer to the truth

3. To determine the accuracy of each individual horizontal position

Different philosophies exist within the surveying community concerning the need for horizontal adjustments. Some say that because today's instruments are so accurate and precise, no adjustments are needed. Certainly the advent of EDMs removed one of the largest sources of horizontal error. Likewise, the introduction of the self-leveling level eliminated most of the vertical errors caused by mis-leveled instruments. With increase in property values, the three reasons stated above become increasingly important to ensure that a mathematically sound strategy for traverse adjustment exists.

If the traverse consists of a simple loop, the horizontal misclosure is small, there is no desire to be closer to the truth, and the value of the accuracy for horizontal position is not desired, a case can be made for not making an adjustment. However, most traverse control for land development projects consists of multiple cross-loops within the main loop for either data gathering or construction layout. Any surveyor who has attempted to compute traverse closures to fit cross-loops into a main traverse loop can testify to the difficulty of achieving good closures. Once again, this is usually due to the random errors in both the main loop and the cross-loops.

#### Angular Adjustment

For traverses, the traditional adjustment process contains two steps, angular and linear adjustments. A closed traverse, by definition, is one that closes upon itself or on a line of known azimuth or bearing. Because traverse misclosures manifest themselves as both angular and linear misclosures, most surveyors start the adjustment process by apportioning the angular misclosure. For a traverse that closes upon an external reference of known azimuth or bearing, the angular error equals the observed azimuth or bearing minus the known azimuth or bearing. For a traverse that closes upon itself, the angular error is determined by the following formula: the sum of the interior angles equals  $(n - 2)180^\circ$ , where  $n$  equals the number of traverse angles in the loop. As an illustration, the sum of a five angle traverse equals  $540^\circ 02'$ . The formula states that the sum should be  $540^\circ$ . Therefore, the angular misclosure equals  $02'$ .

This formula is commonly used by field personnel to check their angular closure before leaving the field to ensure that their angular measurements do not contain a blunder. Intuitively, it can be seen that if the sum of the angles in the above example equaled  $550^\circ 02'$ , it would be possible for one of the angles to contain a  $10^\circ$  blunder, with the other  $02'$  resulting from random errors distributed throughout the traverse. Most surveyors divide the amount of angular misclosure by the number of setups and then equally proportion this amount to each angle. This is referred to as

balancing the angles. As stated above, this method provides no assurance that the errors are being properly assigned, but nevertheless it is commonly used. Furthermore, it makes no provision for error relative to the geometry of the traverse, such as the fact that short traverse legs contain more angular error.

### Error of Closure

Since the advent of modern computers, traverses are computed using coordinates as opposed to latitudes and departures. The reciprocal of the amount of linear misclosure divided by the length of the traverse is called the *error of closure* and is stated as a ratio. For example, if the linear misclosure of a traverse is 0.484 ft and the length of the traverse is 9,551.45 ft, the error of closure will be expressed as 1/0.00005, or approximately 1:19,740. This is commonly stated as 1 in 19,740 or 1 part in 19,740 parts. Simply stated, this means that, due to the accuracy with which the traverse was performed, if the traverse were 20,000 ft in length, there would be a horizontal error of 1 ft.

Before any angular corrections are applied, the traverse should be run up (computed) to determine the amount of linear misclosure. If the distribution of the angular errors causes the linear misclosure to increase, then the angles should not be balanced but rather adjusted by one of the following methods to remove the misclosure. Consider the following hypothetical traverse (Table 14.2) using angles turned to the right:

This traverse contains an angular misclosure of 39". If the angles are balanced proportionately, i.e., 13" per station, the traverse closure would be approximately 1:19,740. If, however, the angles are not balanced before the closure is computed, the closing line would be 0.194 ft and the traverse closure would rise to approximately 1:49,300. A better course of action in this example, which suffers from poor angular geometry and does not even meet third-order Class II angular closure requirements, would be to examine the fieldwork for blunders because the difference between balanced and nonbalanced closures indicates that something is probably wrong with the fieldwork.

**TABLE 14.2 Hypothetical Traverse**

Reference Azimuth = NORTH	
Trav. station 1-2	293° 50' 12"
Trav station 2-3	356° 59' 50"
Trav station 3-1	249° 09' 19"
Closing line:	0.484 feet
Total dist. traversed:	9551.45 feet

### Traditional Methods of Adjustment

Traditionally, surveyors have made traverse adjustments using the *arbitrary method*, the *compass rule*, the *transit rule*, or the *Crandall method*. As the name implies, the arbitrary method allows the surveyor to adjust the traverse by distributing the angular and/or linear errors among the setups where he or she feels they were most likely to have occurred. For instance, if a portion of a traverse consists of good setup conditions, that is, good angular geometry, and long and clear sight distances, and a portion of the traverse is down a creek with short backsight and foresight distances or other poor setup conditions, the surveyor, using discretion, can distribute the misclosure errors by adjusting the observations where he or she feels the errors were most likely to have occurred. In this case, adjustments are made to the setups along the creek. Many surveyors favor this method because it allows for the use of professional judgment. Even so, it does not provide any assurance that the corrections were applied to the needed stations. But under some conditions this can be the correct method.

### The Transit Rule

The transit rule, like the compass rule, applies proportional adjustments and is theoretically better for surveys where angles are measured with greater accuracy than distances, such as stadia surveys. The adjustment is preferred by some surveyors because it does not change the directions of east-west and north-south lines. However, it is seldom used because it is only valid when the traverse lines are parallel with the grid system used for the traverse computations, and thus different results can be obtained for every possible meridian.

### The Crandall Method

The Crandall method, proposed by Professor Charles L. Crandall in 1912, uses a nonrigorous least squares formula and relies on probability theory to determine the most probable lengths of the traverse legs after the angular misclosure has been removed. The adjusted angles are then held fixed and all the corrections are applied to the traverse lengths. Because the assumption is made that each angle was measured with equal precision, angle balancing is usually performed on an equal basis for each traverse angle. This adjustment does not acknowledge that the accuracy of bearings derived from these adjusted angles is not constant. Also, because of the previously discussed desirability of being able to place the error correction at the point where it is most likely to have occurred, the Crandall method is not a preferred method of adjustment.

### The Compass Rule

By far the most common traverse adjustment in use is the compass or Bowditch method. Proposed by Nathaniel Bowditch in 1908, the compass method assumes that angles and

distances are equally accurate and applies proportional corrections based on the length of each traverse leg. The formula is stated as

$$\begin{aligned} &\text{Correction to northing coordinate} && (14.3) \\ &= \left( \frac{\text{distance from the starting stations}}{\text{total length of traverse}} \right) \end{aligned}$$

× total error in northing closure

and

$$\begin{aligned} &\text{Correction to easting coordinate} && (14.4) \\ &= \left( \frac{\text{distance from the starting stations}}{\text{total length of traverse}} \right) \end{aligned}$$

× total error in easting closure

using the hypothetical traverse from the error of closure section above (angle convention = backsight-occupied station-foresight) (see Table 14.3).

The object of the adjustment is to apply the corrections (observing the signs) in such a manner as to cause the ending coordinate values to be the same as the beginning coordinate values. It can be seen that both the easting misclosure and the northing misclosure are negative, that is, the ending northing and easting coordinates are south of the beginning northing and easting coordinates. Therefore, the individual corrections for both the easting and northing coordinates are positive and would be applied as shown in Table 14.4. Note the accumulation of the traverse distances from the starting station.

### Vertical Adjustments

Much as with horizontal adjustments, the most common method of adjusting level loops is to proportion the closing

error equally to each turning point along the level run. However, the best method of adjustment requires knowledge of the distances between each turn. For this reason, the three-wire level procedure, because it supplies these distances through the use of stadia intercepts, is the easiest to adjust. The formula is

$$\begin{aligned} &\text{Correction to each turn point} && (14.5) \\ &= \left( \frac{\text{distance from the start}}{\text{total loop distance}} \right) \end{aligned}$$

× total vertical misclosure

The following level loop with a source benchmark elevation of 500.00 will illustrate this principle

As with the traverse adjustment shown in Table 14.3, note that the misclosure is positive in value, thereby requiring that each individual correction be subtracted from the unadjusted (or observed) elevations. Also note that this example does not meet third-order vertical closure requirements, which state that the allowable error shall not be more than 0.05 ft in one mile.

### LEAST SQUARES

Throughout this chapter, reference has been made to the fact that most traditional methods of adjustment rely on equal proportioning or intuition or experience to remove the inevitable errors that result from survey measurements. However, a better way exists, and thanks to powerful and affordable personal computers and, more importantly, easy-to-use and affordable software, there is no reason why the land development surveyor cannot make use of least squares adjustments. It is beyond the scope of this chapter to cover the complex mathematics involved in least squares. Suffice it to say, however, that with a rudimentary under-

**TABLE 14.3 Data for Compass Rule Adjustment**

STATION	ANGLE RIGHT	HORIZONTAL DIST.	NORTHING	EASTING
1			10,000.000	10,000.000
3-1-2	293°50'12"	4,694.20	11,897.069	5,706.213
1-2-3	356°59'50"	4,593.96	10,263.183	9,999.795
2-3-1	249°09'19"	263.30	9,999.884	9,999.845

Angular Error = 39" (angles not balanced)

Linear Error = 0.194

Northing closure error = -0.116

Easting closure error = -0.155

Distance traversed = 9,551.46 feet

Closure ratio = 1:49,300 ±

**TABLE 14.4 Traverse Corrections Using the Compass Rule**

STATION	DISTANCE	N. CORRECTION	E. CORRECTION	ADJUSTED N.	ADJUSTED E.
Station 2	4,694.20	+0.057	+0.076	11,897.126	5,706.289
Station 3	9,288.16	+0.113	+0.151	10,263.296	9,999.946
Station 1	9,551.46	+0.116	+0.155	10,000.000	10,000.000

standing of the workings of the adjustment and the answers it provides, any surveyor can be trained in its use.

For many years, surveyors have repeatedly heard about the desirability of using the least squares adjustment method. Yet very few practicing surveyors are actually using the method for everyday surveying. Besides being a superior adjustment method, least squares will allow the simultaneous adjustment of multiple loop traverses. This is a long-desired capability. In addition, it provides a statistical measure of accuracy based on positional tolerance that some believe is preferable to classifying a survey by closure accuracy. As the surveying community moves into the era of satellite surveying, it has become apparent that it will have to learn how to use least squares and a minimal amount of the rudiments of the statistical theory that goes along with the method.

True least squares for all the data (loops and cross-loops simultaneously) works by deriving a solution that minimizes the sum of the squares of the residuals. It does this by minimizing the variation between the field observations and the final adjustment. It also allows the surveyor to combine the known accuracy of the instrument and EDM with the field conditions that existed when the measurements were taken. Least squares will then use this information to provide an adjustment that closely reflects the actual position of the located points, all the while adjusting the coordinates the absolute minimum amount necessary.

Accuracy and precision should be the ultimate goal. While the compass method often introduces needless error, least squares allows statistical adjustment, thereby providing the best possible estimate of the true position of the points located, provided the blunders and systematic errors have been removed.

Even though the compass method has emerged as the most popular adjustment method for horizontal misclosures, it is highly unlikely that angular and distance errors will occur in an equally proportionate manner throughout a network. Most surveyors do not attempt to distribute angular or distance misclosure based on knowledge of field conditions. Rather, they simply “hit the compass button and go with the results.” Consequently, when they return to the field to perform stakeout, the angles and distances derived by the adjustment do not match the original angles or distances.

Traverses that contain more than one loop also present major problems when the compass method is used. Invariably, the cross-loops do not fit well with the main network, leaving the surveyor no choice but to force the cross-loop to fit the main network. This introduces a large amount of error into the cross-loop and causes the field people to lose confidence in their control. The obvious intent should be to perform an adjustment that makes the smallest possible changes to the angle and distance measurements, preserving the surveyor’s original observations as much as possible.

**TABLE 14.5**

STATION	ELEVATION	OBSERVED ELEV.	DISTANCE FROM START	CORRECTION	ADJ. ELEVATION
B.M.	500.00				
T.P.		505.15	2000 feet	-0.028	505.122
T.P.		511.23	3500 feet	-0.049	511.181
T.P.		502.87	4500 feet	-0.063	502.807
B.M.		500.07	5000 feet	-0.07	500.00
Misclosure		+0.07			

Classification of a traverse by closure is fine for an indication of the accuracy of the traverse as a whole, but it will not indicate where the traverse points really lie or what could be done to improve the traverse.

Least squares will provide a superior adjustment based upon a statistical analysis of the data and varying the coordinate values the least amount, resulting in a best fit of the data. It is hoped that the preceding sections have adequately presented the concept of the probability curve and that the reader understands the concept of having a certain level of confidence that their measurements will fall somewhere within that curve. The preceding sections should have also brought about an understanding that to achieve a higher level of confidence in where the objects being located really are, the surveyor must use a more accurate piece of equipment or employ more precise procedures to gather the measurements.

Least squares will provide positional tolerance information based on a 95% confidence level. This means that the surveyor can say, with a 95% level of certainty, that the points he or she has located fall within an ellipse of certain dimensions. With traditional adjustment techniques, the surveyor has no idea where the points he has located actually are. This is why cross-loops do not fit.

If a surveyor wishes to use the manufacturer's specifications for the plus or minus uncertainty in the mean of a direct and reverse measurement, he or she can do so. But if the surveyor wishes to change the weights (confidence) based on his or her knowledge of actual field conditions, the program will allow it. An example of a situation where the surveyor would want to weight an angle measurement might be the case of a poor (unstable) setup, a short backsight, objects on line that might bend the line of sight, or other field conditions that the surveyor would know about that would cause a particular measurement to be less accurate than others.

Redundant measurements to the same point from different setups will result in the most accurate adjustment. To take full advantage of this feature would require a fundamental change in the manner in which surveys are performed. Instead of simply establishing traverse points so that the backsight and the foresight can be seen, the proper use of least squares will encourage these points to be placed in such a way as to allow maximum intervisibility between all setup points. The redundancy provided by more observations will result in a better solution.

Least squares provides two additional tools for the surveyor. The preanalysis option will allow the user to input approximate coordinates of the proposed traverse stations. The program will then suggest additional stations and/or measurements that will result in a stronger, more accurate network. These approximate coordinates can be digitized from a quad sheet or other map or derived by scaling and protracting. The preanalysis option will detect geometric weaknesses in the proposed traverse. It will even tell the

user if he or she is doing more work than necessary to achieve the desired level of accuracy.

Another powerful feature is blunder detection. Least squares software will analyze the data from the traverse and detect possible blunders. In the event a survey fails to meet the minimum requirements for accuracy, this feature alone will indicate the stations where the blunders in measurement most likely occurred. Because the program makes a simultaneous adjustment and all of the stations are interrelated, it might tell the user to look at a station that is one or two traverse points away from the real culprit. How many times has a surveyor gone back to the field to re-turn angles, not having the slightest idea where the angle bust occurred? Even the technique of using the perpendicular bisector of the closing line does not always indicate where the error lies.

The 1962 ALTA/ACSM *Minimum Standard Detail Requirements* called for the surveyor to make a statement regarding the positional tolerance of his work. The 1986, 1988, and 1992 standards removed this requirement and reverted to classifying surveys by closure and procedure. As stated above, classification by closure is fine as an indication of the accuracy of the traverse as a whole, but it does not indicate where the errors lie, nor does it give any idea of the relative accuracy of any one point. Least squares, because it determines error ellipses, gives such an idea of each point.

Furthermore, for the reasons discussed above, the new requirements make the recommendation that EDMs not be used on urban class surveys for obtaining distances under 81 m ( $266 \pm$  ft). Again, this recommendation is based on the knowledge that statistically, every measurement that an EDM makes is going to have a plus or minus error based on the accuracy of the EDM and the length of the shot. Is it probable that surveyors will not use their EDMs when they believe that, even with the stated uncertainty of the measurement, an EDM distance is superior to a hand-chained distance? Incidentally, the standards do not allow hand-chained distances if they are under 20 m ( $66 \pm$  ft).

Some believe that the reason the requirement for a statement of positional tolerance was removed was because surveyors did not understand how the method of least squares worked and therefore were uncertain about using an adjustment procedure. Could these be the same surveyors who never give a thought to applying angular and distance corrections to their traverses based on their knowledge of field conditions, but rather simply hit the "go" button? Do these surveyors really understand how their EDM works? If not, does this lack of understanding prevent them from using their EDMs?

As the use of total stations for surveys becomes more prevalent, the use of these instruments to perform simultaneous horizontal and vertical surveys will increase. NGS has pioneered the use of total stations to perform first-order level loops, and even the less precise total stations will pro-

**TABLE 14.6 - a Standards for the Classification of Geodetic Control and Principal Recommended Uses**

CLASSIFICATION	HORIZONTAL CONTROL		
	FIRST-ORDER	SECOND-ORDER	THIRD-ORDER
Relative accuracy between directly connected adjacent points (at least)	1 part in 100,000	1 part in 20,000	1 part in 10,000
	1 part in 50,000	1 part in 20,000	1 part in 10,000
Recommended uses	Primary National Network. Metropolitan Area Surveys. Scientific Studies	Area control which strengthens the National Network. Subsidiary metropolitan control. National Network.	Area control which contributes to, but is supplemental to, the National Network. Local control surveys. General control surveys referenced to the National Network. Local control surveys.
<b>VERTICAL CONTROL</b>			
Relative accuracy between directly connected points or benchmarks (standard error)	0.5 mm $\sqrt{K}$	0.7 mm $\sqrt{K}$	1.0 mm $\sqrt{K}$
	0.5 mm $\sqrt{K}$	0.7 mm $\sqrt{K}$	1.0 mm $\sqrt{K}$
Recommended uses	Basic framework of the National Network and metropolitan area control. Regional crustal movement studies. Extensive engineering projects. Support for subsidiary surveys.	Secondary framework of the National Network and metropolitan area control. Local crustal movement studies. Large engineering projects. Topographic mapping.	Densification within the National Network. Rapid subsidence studies. Local engineering projects. Topographic mapping. Small-scale topographic mapping. Establishing gradients in mountainous areas. Small engineering projects. May or may not be adjusted to the National Network.

(K is the distance in kilometers between points.)



**TABLE 14.6-b Classification, Standards of Accuracy, and General Specifications for Vertical Control**

CLASSIFICATION	FIRST-ORDER			SECOND-ORDER	THIRD-ORDER
	CLASS I, CLASS II	CLASS I	CLASS II		
<i>Principal uses</i>	Basic framework of the National Network and of metropolitan area control	Secondary control of the National Network and of metropolitan area control	Control densification, usually adjusted to the National Net. Local engineering projects	Miscellaneous local control may not be adjusted to the National Network.	
Minimum standards; higher accuracies may be used for special purposes	Extensive engineering projects Regional crustal movement investigations Determining geopotential values	Large engineering projects Local crustal movement and subsidence investigations Support for lower-order control	Topographic mapping Studies of rapid subsidence Support for local surveys	Small engineering projects Small-scale topo. mapping Drainage studies and gradient establishment in mountainous areas	
<i>Recommended spacing of lines</i>	Net A: 100 to 300 km <i>Class I</i>	Secondary Net: 20 to 50 km	Area Control: 10 to 25 km		As needed
National Network	Net B: 50 to 100 km <i>Class II</i>				
Metropolitan control; other purposes	2 to 8 km As needed	0.5 to 1 km As needed	As needed	As needed	As needed
<i>Spacing of marks along lines</i>	1 to 3 km	1 to 3 km	Not more than 3 km	Not more than 3 km	Not more than 3 km
<i>Gravity requirement*</i>	$0.20 \times 10^{-3}$ gpu				
<i>Instrument standards</i>	Automatic or tilting levels with parallel plate micrometers; invar scale rods	Automatic or tilting levels with optical micrometers or three-wire levels; invar scale rods	Geodetic levels and invar scale rods	Geodetic levels and rods	Geodetic levels and rods

<i>Field procedures</i>	Double-run; forward and backward, each section	Double-run; forward and backward, each section	Double- or single-run	Double- or single-run
Section length	1 to 2 km	1 to 2 km	1 to 3 km for double-run	1 to 3 km for double-run
Maximum length of sight	50 m Class I; 60 m Class II	60 m	70 m	90 m
<i>Field procedures†</i>				
Max. difference in lengths				
Forward & backward sights per setup	2 m Class I; 5 m Class II	5 m	10 m	10 m
per section (cumulative)	4 m Class I; 10 m Class II	10 m	10 m	10 m
Max. length of line	Net A; 300 km		50 km double-run	25 km double-run
between connections	Net B; 100 km	50 km	25 km single-run	10 km single-run
<i>Maximum closures‡</i>				
Section; fwd. and bkwd.	3 mm $\sqrt{K}$ Class I; 4 mm $\sqrt{K}$ Class II	6 mm $\sqrt{K}$	8 mm $\sqrt{K}$	12 mm $\sqrt{K}$
Loop or line	4 mm $\sqrt{K}$ Class I; 5 mm $\sqrt{K}$ Class II	6 mm $\sqrt{K}$	8 mm $\sqrt{K}$	12 mm $\sqrt{K}$

\*See text for discussion of instruments.

†The maximum length of line between connections may be increased to 100 km for double run for Second-Order, Class II, and to 50 km for double run for Third-Order in those areas where the First-Order control has not been fully established.

‡Check between forward and backward runnings were K is the distance in kilometers.

**TABLE 14.6-c Traverse**

**TRAVERSE**

CLASSIFICATION	FIRST-ORDER		SECOND-ORDER		THIRD-ORDER	
	CLASS I	CLASS II	CLASS I	CLASS II	CLASS I	CLASS II
<i>Recommended spacing of principal stations</i>	Network stations 10–15 km Other surveys seldom less than 3 km.	Principal stations seldom less than 4 km except in metropolitan area surveys where the limitation is 0.3 km.	Principal stations seldom less than 2 km except in metropolitan area surveys where the limitation is 0.2 km.	Principal stations seldom less than 2 km except in metropolitan area surveys where the limitation is 0.2 km.	Seldom less than 0.1 km in tertiary surveys in metropolitan area surveys. As required for other surveys.	
<i>Horizontal directions or angles</i>						
Instrument	0".2	0".2	0".2	0".2	1".0	1".0
Number of observations	16	8	12*	6	4	2"
Rejection limit from mean	4"	4"	5"	4"	5"	5"
<i>Length measurements</i>						
Standard error <sup>1</sup>	1 part in 600,000	1 part in 300,000	1 part in 300,000	1 part in 120,000	1 part in 60,000	1 part in 30,000
<i>Reciprocal vertical angle observations</i> <sup>4</sup>						
Number of and spread between observations	3 D/R—10"	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of stations between known elevations	4–6	6–8	6–8	8–10	10–15	15–20
<i>Astro azimuths</i>						
Number of courses between azimuth checks <sup>7</sup>	5–6	10–12	10–12	15–20	20–25	30–40
No. of obs./night	16	16	16	12	8	4
No. of nights	2	2	2	1	1	1
Standard error	0".45	0".45	0".45	1".5	3".0	8".0
Azimuth closure at azimuth check point not to exceed <sup>8</sup>	1".0 per station or 2" √N	1".5 per station or 3" √N. Metropolitan area surveys seldom to exceed 2".0 per station or 3" √N	1".5 per station or 3" √N. Metropolitan area surveys seldom to exceed 4".0 per station or 8" √N	2".0 per station or 6" √N. Metropolitan area surveys seldom to exceed 4".0 per station or 8" √N	3".0 per station or 10" √N. Metro-politan area surveys seldom to exceed 6".0 per station or 15" √N	8" per station or 30" √N
<i>Position closure</i> <sup>8,9</sup> after azimuth adjustment	0.04 m √K or 1:100,000	0.08 m √K or 1:50,000	0.08 m √K or 1:50,000	0.2m √K or 1:20,000	0.4m √K or 1:10,000	0.8m √K or 1:5,000

\*May be reduced to 8 and 4, respectively, in metropolitan areas.

vide enough accuracy for triglevels to aerial targets and certain construction layout tasks. With this increased use comes a need for a method to adjust both the horizontal and vertical measurements of any traverse, applying the same statistically proven adjustment techniques to both.

Level circuits containing multiple loops can be easily adjusted by least squares. The user can assign different weights (standard errors), thus allowing the leveling of observations of varying accuracies to be simultaneously adjusted, as would happen if measurement were collected by different equipment at different times. The program will work with distances between turns or the number of turns to assign weights automatically.

Although the reputation of least squares may seem intimidating, the software removes the necessity for the surveyor to understand all of the complex aspects of the program. Simply inputting the same data as always will allow a superior adjustment to be obtained. And if the surveyor fully utilizes the capabilities of the method, field techniques can be learned that will result in higher-quality surveys.

### A Typical Least Squares Adjustment<sup>2</sup>

Under Adjustment Options, the user can select the type of adjustment desired, e.g., data check, full adjustment, blunder detect, etc. The user can also select whether the network is 2D or 3D. One of the most powerful features of the program is its ability to handle state plane coordinates. The user can input the state number and zone, an average project elevation (or elevations, fixed or free, at each traverse station), and the geoid separation value. The geoid separation value can be obtained from NGS or from the NGS State Geodetic Advisor. Using this information, the program will automatically compute the combined scale factor and process the input data. The field crew can use this combined scale factor when staking out, for instance, by inputting the value into a data collector. The data collector can then apply the factor to each distance measurement, thereby saving time for the field crew. The program will accept local coordinate systems, NAD 27 or 83 Mercator or Lambert state plane coordinates, and UTM coordinates.

The user next inputs the default standard errors for the equipment he or she is using. Many users simply input the manufacturer's stated accuracy for these values, e.g., 5 mm and 5 PPM for a standard EDM. As mentioned above under Types and Sources of Error, if the input file consists of angles (as opposed to directions) then the NGS procedure of multiplying the manufacturer's stated accuracy by the  $\sqrt{2}$  should be used. This will establish the *a priori* (before-adjustment) value for the observations and represents the user's estimate of the accuracy of the equipment.

The weight of an observation can be thought of as a measure of its precision. More precise measurements have larger weights and will have more influence in the adjustment. Generally, observations with smaller weights will be allowed to change more, that is, given larger residuals by the adjustment. As discussed above under Adjustments, the user can assign weights to the observations or make the assumption that all observations are equal with respect to their accuracy. Because the program will render an adjustment regardless of the input values, many users will hold the coordinates for two points (or one point and an azimuth) fixed and run what is called a *minimally constrained* adjustment. This trial adjustment can aid the user in identifying blunders in the observation. Next the user can fix the values for all points or azimuths that are to be held fixed and rerun the adjustment. Remember that this is the opportunity for the surveyor to express his or her level of confidence in the measurements by weighing the observations. Most users assume that all observations are equal in accuracy and allow the program to use the default values for weighing.

The user can control the number of *iterations* the program uses to perform the adjustment by specifying the convergence limit. If the change in the total sum of the squares of the standardized residuals is less than this convergence limit, the adjustment is said to have converged, and iterations will stop.

After the adjustment has run, the user is presented with a statistical summary containing some of the most useful and important information about the adjustment. The total number of observations defines the number of condition equations to be solved. All observations that have not been fixed are considered to be free, i.e., adjustable. The number of unknowns equals the number of adjustable coordinate components. Each 2D station contributes two unknowns as each 3D station contributes three. The *degrees of freedom* equal the number of condition equations minus the number of unknowns. Using the approach of giving the program all possible pieces of data, some users include every observation in the input file, e.g., foresight and backsight distances and foresight and backsight zenith angles. This approach actually inflates the degrees of freedom that affect the chi-squared test, and even though the final adjusted coordinates will be the same, error propagation will be influenced, as will the chi-squared test. The result of this approach is that otherwise good surveys might fail the chi-squared test because of too many degrees of freedom.

The next item the summary lists is the sum of the squares of the *standardized residuals* for each data type. The standardized (or normalized) residual is the result of dividing the residual for each observation by its standard error. This is done because although the residual is the variance between the observed value and the value that best fits into the network, the standardized residual takes into account the geometry of the survey network. Because of this, it is a

<sup>2</sup> The software referred to is Star\*Net, Starplus Software, Oakland, California.

good check for blunders. If the value is less than one, the data type actually fits better than estimated results. If greater than one, it is a poor fit. Any value greater than three is flagged as an indication that there is something wrong with that data type. The next part of the summary lists the *error factor* for each measurement type. The error factors are affected by the number of observations and are a good indication of how well each data type fits into the adjustment. The error factors should be roughly equal and should range from 0.5 to 1.5. If one is much greater than another, then a problem could exist with that measurement type. Remember that unless the user weights measurements, the adjustment will assume that the data types are equal in precision. Large error factors indicate an error in the input data, a systematic error, or input standard errors that are unrealistically small. Also, because of the interconnection of stations, a large error in one data type can influence the error factor of others. If the user has established the input standard errors correctly and has no systematic errors or blunders, the *total error factor* should be close to 1. This is an indication of correct input data assumptions. The total error factor is calculated by dividing the square root of the sum of the squares of the standardized residuals by the number of redundancies. The total error factor is also referred to as the *reference factor* or the *standard error of unit weight*.

The program also applies a test to the adjustment by comparing the total error factor against the assumed value. This test, called the *chi-squared test*, checks the validity of the weighing observation. If this value is significantly greater than 1.0, then there is some problem with the adjustment, either a systematic error, a blunder, or weakness of network geometry. The chi-squared test is performed at a 5% level of significance. This means that there is a 5% chance that the adjustment will pass the test with a total error factor that is greater than 1.0. If the value is significantly less than 1.0, then the input standard errors have been overestimated. While this may not affect the final coordinates, the result of the chi-squared test is an error propagation in that portion of the adjustment and will increase the size of the error ellipses.

The *error ellipses* station represent the size of the ellipse for which there is a 95% certainty that the point actually lies. As indicated by the program, the semimajor and semiminor values are ground dimensions across half of the ellipse. The absolute dimensions provide a measure of positional accuracy. Points with larger ellipses are weaker than those with smaller ellipses. The shapes and orientation indicate how the network can be strengthened. An elongated ellipse indicates that there is a larger error in one of the coordinates for that station and that perhaps an additional distance or angle measurement is needed to that sta-

**TABLE 14.7 Excerpts from Standards and Specifications for Geodetic Control Networks (FGCC 1984)**

**Traverse.** Traverse is a measurement system comprised of joined distance and theodolite observations supported by occasional astronomic observations. Traverse is used to densify horizontal control.

#### NETWORK GEOMETRY

ORDER CLASS	FIRST	SECOND I	SECOND II	THIRD I	THIRD II
Station spacing not less than (km) . . . . .	10	4	2	0.5	0.5
Maximum deviation of main traverse from straight line . . . . .	20°	20°	25°	30°	40°
Minimum number of bench mark ties . . . . .	2	2	2	2	2
Bench mark tie spacing not more than (segments) . . . . .	6	8	10	15	20
Astronomic azimuth spacing not more than (segments) . . . . .	6	12	20	25	40
Minimum number of network control points . . . . .	4	3	2	2	2

The new survey is required to tie to a minimum number of network control points spaced well apart. These network points must have datum values equivalent to or better than the intended order (and class) of the new survey. Whenever the distance between two new unconnected survey points is less than 20 percent of the distance between those points traced along existing or new connections, then a direct connection must be made between those two survey points. In addition, the survey should tie into any sufficiently accurate network control points within the station spacing distance of the survey. These ties must include EDM or taped distances. Nonredundant geodetic connections to the network stations are not considered sufficient ties. Nonredundantly determined stations are not allowed. Reciprocal vertical angles or geodetic leveling are observed along all traverse lines.

**Instrumentation.** Only properly maintained theodolites are adequate for observing directions and azimuths for traverse. Only precisely marked targets, mounted stably on tripods or supported towers, should be employed. The target should have a clearly defined center, resolvable at the minimum control spacing. Optical plummets or collimators are required to ensure that the theodolites and targets are centered over the marks. Microwave-type electronic distance measurement equipment is not sufficiently accurate for measuring first-order traverses.

**TABLE 14.7** (Continued)

ORDER CLASS	FIRST	SECOND I	SECOND II	THIRD I	THIRD II
Theodolite, least count . . . . .	0.2"	1.0"	1.0"	1.0"	1.0"

**Calibration Procedures.** Each year and whenever the difference between direct and reverse readings of the theodolite depart from 180° by more than 30", the instrument should be adjusted for collimation error. Readjustment of the cross hairs and the level bubble should be done whenever their misadjustments affect the instrument reading by the amount of the least count.

All electronic distance measuring devices and retroreflectors should be serviced regularly and checked frequently over lines of known distances. The National Geodetic Survey has established specific calibration base lines for this purpose. EDM instruments should be calibrated annually, and frequency checks made semiannually.

**Field Procedures.** Theodolite observations for first-order and second-order, class I surveys may be made only at night. Electronic distance measurements need a record at both ends of the line of wet and dry bulb temperatures to ±1°C and barometric pressure to ±5 mm of mercury. The theodolite, EDM, and targets should be centered to within 1 mm over the survey mark or eccentric point.

ORDER CLASS	FIRST	SECOND I	SECOND II	THIRD I	THIRD II
<b>Directions</b>					
Number of positions . . . . .	16	8 or 12†	6 or 8*	4	2
Standard deviation of mean not to exceed . . . . .	0.4"	0.5"	0.8"	1.2"	2.0"
Rejection limit from the mean . . . . .	4"	5"	5"	5"	5"
<b>Reciprocal Vertical Angles (along distance sight path)</b>					
Number of independent observations direct/reverse . . . . .	3	3	2	2	2
Maximum spread . . . . .	10"	10"	10"	10"	20"
Maximum time interval between reciprocal angles (hr) . . . . .	1	1	1	1	1
<b>Astronomic Azimuths</b>					
Observations per night . . . . .	16	16	12	8	4
Number of nights . . . . .	2	2	1	1	1
Standard deviation of mean not to exceed . . . . .	0.45"	0.45"	0.6"	1.0"	1.7"
Rejection limit from the mean . . . . .	5"	5"	5"	6"	6"
<b>Electro-Optical Distances</b>					
Minimum number of measurements . . . . .	1	1	1	1	1
Minimum number of concentric observations/measurement . . . . .	1	1	1	1	1
Minimum number of offset observations/measurement . . . . .	1	1	—	—	—
Maximum difference from mean of observations (mm) . . . . .	60	60	—	—	—
Minimum number of readings/observation (or equivalent) . . . . .	10	10	10	10	10
Maximum difference from mean of readings (mm) . . . . .	§	§	§	§	§

ORDER CLASS	FIRST	SECOND I	SECOND II	THIRD I	THIRD II
<b>Infrared Distances</b>					
Minimum number of measurements . . . . .	1	1	1	1	1
Minimum number of concentric observations/measurement . . . . .	1	1	1	1	1
Minimum number of offset observations/measurement . . . . .	1	1	1‡	—	—
Maximum difference from mean of observations (mm) . . . . .	10	10	10‡	—	—
Minimum number of readings/observation . . . . .	10	10	10	10	10
Maximum difference from mean of readings (mm) . . . . .	§	§	§	§	§
<b>Microwave Distances</b>					
Minimum number of measurements . . . . .	—	1	1	1	1
Minimum number of concentric observations/measurement . . . . .	—	2**	1**	1**	1**
Maximum difference from mean of observations (mm) . . . . .	—	150	150	200	200
Minimum number of readings/observation . . . . .	—	20	20	10	10
Maximum difference from mean of readings (mm) . . . . .	—	§	§	§	§

† 8 if 0.2", 12 if 1.0" resolution.  
 \* 6 if 0.2", 8 if 1.0" resolution.  
 § as specified by manufacturer.  
 ‡ only if decimal reading near 0 or high 9's.  
 \*\* carried out at both ends of the line.

Measurements of astronomic latitude and longitude are not required in the United States, except perhaps for first-order work, because sufficient information for determining deflections of the vertical exists. Detailed procedures can be found in Hoskinson and Duerksen (1952).

**OFFICE PROCEDURES**

ORDER CLASS	FIRST	SECOND I	SECOND II	THIRD I	THIRD II
<b>Azimuth closure at azimuth check point (seconds of arc) . . . . .</b>					
	1.7 √N	3.0 √N	4.5 √N	10.0 √N	12.0 √N
<b>Position closure . . . . .</b>					
	0.04 √K	0.08 √K	0.20 √K	0.40 √K	0.80 √K
after azimuth . . . . . or or or or or					
adjustment† . . . . . 1:100,000 1:50,000 1:20,000 1:10,000 1:5,000					

(N is number of segments, K is route distance in km)  
 † The expression containing the square root is designed for longer lines where higher proportional accuracy is required. Use the formula that gives the smallest permissible closure. The closure (e.g., 1:100,000) is obtained by computing the difference between the computed and fixed values, and dividing this difference by K. Note: Do not confuse closure with distance accuracy of the survey.

**Geodetic Leveling.** Geodetic leveling is a measurement system comprised of elevation differences observed between nearby rods. Leveling is used to extend vertical control.

**TABLE 14.7** (Continued)

**NETWORK GEOMETRY**

ORDER CLASS	FIRST	FIRST	SECOND	SECOND	THIRD
	I	II	I	II	
Bench mark spacing not more than (km).....	3	3	3	3	3
Average bench mark spacing not more than (km).....	1.6	1.6	1.6	3.0	3.0
Line length between network control points not more than (km).....	300	100	50	50 (double-run) 25 (single-run)	25 10

New surveys are required to tie to existing network bench marks at the beginning and end of the leveling line. These network bench marks must have an order (and class) equivalent to or better than the intended order (and class) of the new survey. First-order surveys are required to perform check connections to a minimum of six bench marks, three at each end. All other surveys require a minimum of four check connections, two at each end. "Check connection" means that the observed elevation difference agrees with the adjusted elevation difference within the tolerance limit of the new survey. Checking the elevation difference between two bench marks located on the same structure, or so close together that both may have been affected by the same localized disturbance, is not considered a proper check. In addition, the survey is required to connect to any network control points within 3 km of its path. However, if the survey is run parallel to existing control, then the following table specifies the maximum spacing of extra connections between the survey and the control. At least one extra connection should always be made.

DISTANCE, SURVEY TO NETWORK	MAXIMUM SPACING OF EXTRA CONNECTIONS (KM)
0.5 km or less .....	5
0.5 km to 2.0 km .....	10
2.0 km to 3.0 km .....	20

**INSTRUMENTATION**

ORDER CLASS	FIRST	SECOND	SECOND	THIRD	THIRD
	I	II	I	II	
<b>Leveling Instrument</b>					
Minimum repeatability of line of sight .....					
	0.00"	0.00"	0.00"	0.00"	0.00"
Leveling rod construction .....					
	IDS	IDS	or ISS	ISS	Wood or Metal
<b>Instrument and rod resolution (combined)</b>					
Least count (mm).....					
	0.1	0.1	0.5-1.0*	1.0	1.0

(IDS—Invar, double scale)  
 (ISS—Invar, single scale)  
 † if optional micrometer is used.  
 \* 1.0 mm if 3-wire method, 0.5 mm if optical micrometer.

Only a compensator or tilting leveling instrument with an optical micrometer should be used for first-order leveling. Leveling rods should be one piece. Wooden or metal rods may be employed only for third-order work. A turning point consisting of a steel turning pin with a driving cap should be utilized. If a steel pin cannot be driven, then a turning plate ("turtle") weighing at least 7 kg should be substituted. In situations allowing neither turning pins nor turning plates (sandy or marshy soils), a long wooden stake with a double-headed nail should be driven to a firm depth.

**CALIBRATION PROCEDURES**

ORDER CLASS	FIRST	FIRST	SECOND	SECOND	THIRD
	I	II	I	II	
<b>Leveling instrument</b>					
Maximum collimation error, single line of sight (mm/m) .....					
	0.05	0.05	0.05	0.05	0.10
Maximum collimation error, reversible compensator type instruments, mean of two lines of sight (mm/m) .....					
	0.02	0.02	0.02	0.02	0.04
Time interval between collimation error determinations not longer than (days)					
	7	7	7	7	7
	1	1	1	1	7
Maximum angular difference between two lines of sight, reversible compensator .....					
	40"	40"	40"	40"	60"

**TABLE 14.7** (Continued)

Order Class	First I	First II	Second I	Second II	Third
<b>Leveling rod</b>					
Minimum scale calibration standard . . . .	N	N	N	M	M
Time interval between scale calibrations (yr) . . . . .	1	1	—	—	—
Leveling rod bubble verticality maintained to within . . . . .	10 <sup>+</sup>	10 <sup>+</sup>	10 <sup>+</sup>	10 <sup>+</sup>	10 <sup>+</sup>

(N—National standard)  
(M—Manufacturer's standard)

Compensator-type instruments should be checked for proper operation at least every 2 weeks of use. Rod calibration should be repeated whenever the rod is dropped or damaged in any way. Rod

levels should be checked for proper alignment once a week. The manufacturer's calibration standard should, as a minimum, describe scale behavior with respect to temperature.

**FIELD PROCEDURES**

ORDER CLASS	FIRST I	FIRST II	SECOND I	SECOND II	THIRD II
Minimal observation method. . . . .			micrometer or 3-wire	3-wire	center wire
Section running. . . . .	SRDS or DR or SP	SRDS or DR or SP	SRDS or DR† or SP	SRDS or DR*	SRDS or DR§
Difference of forward and backward sight lengths never to exceed per setup (m) . . . . .	2	5	5	10	10
per sectin (m) . . . . .	4	10	10	10	10
Maximum sight length (m) . . . . .	50	60	60	70	90
Minimum ground clearance of line of sight (m) . . . . .	0.5	0.5	0.5	0.5	0.5
Even number of setups when not using leveling rods with detailed calibration . . . . .	yes	yes	yes	yes	—
Determine temperature gradient for the vertical range of the line of sight at each setup. . . . .	yes	yes	yes	—	—
Maximum section misclosure (mm) . . . . .	$3\sqrt{D}$	$4\sqrt{D}$	$6\sqrt{D}$	$8\sqrt{D}$	$12\sqrt{D}$
Maximum loop misclosure (mm) . . . . .	$4\sqrt{E}$	$5\sqrt{E}$	$6\sqrt{E}$	$8\sqrt{E}$	$12\sqrt{E}$
<b>Single-run methods</b>					
Reverse direction of single runs every half day . . . . .	yes	yes	yes	—	—
<b>Nonreversible compensator leveling instruments</b>					
Off-level/relevel instrument between observing the high and low rod scales. . . . .	yes	yes	yes	—	—
<b>3-wire method</b>					
Reading check (difference between top and bottom intervals) for one setup	—	—	2	2	3
Read rod 1 first in alternate setup method. . . . .	—	—	yes	yes	yes



TABLE 14.7 (Continued)

ORDER CLASS	FIRST I	FIRST II	SECOND I	SECOND II	THIRD II
<b>Double scale rods</b>					
Low-high scale elevation difference for one setup not to exceed (mm)					
With reversible compensator. . . . .	0.40	1.00	1.00	2.00	2.00
Other instrument types					
Half-centimeter rods. . . . .	0.25	0.30	0.60	0.70	1.30
Full-centimeter rods. . . . .	0.30	0.30	0.60	0.70	1.30

(SRDS—Single-Run, Double Simultaneous procedure)

(DR—Double-Run)

(SP—SPur, less than 25 km, double-run)

D—shortest length of section (one-way) in km

E—perimeter of loop in km

† Must double-run when using 3-wire method.

\* May single-run if line length between network control points is less than 25 km.

§ May single-run if line length between network control points is less than 10 km.

tion. A large but circular ellipse indicates that the northings and eastings are balanced for that point, but more accurate measurements are needed, or additional measurements are needed, from other stations. If many ellipses are elongated and all point in the same direction, the network is unbalanced along that direction. This might be the result of an azimuth deficiency that allows one side of the network to rotate slightly. An additional control point or observed azimuth will help alleviate this condition.

The program also develops *relative error ellipses* to express the relative position of two points. Two points with large station ellipses may actually have a small relative ellipse between them, meaning that their absolute positions in the survey are weak but that they are correlated with each other and move together. The length of the relative ellipse projected onto the line between the two points is a measure of the precision of the distance between the two points. The length of the relative ellipse perpendicular to the line is a measure of the precision of the bearing between the two stations. The plus or minus relative errors of the bearing and distance for each course are based on the values contained in its associated relative error ellipse. The distance precision is computed by dividing the relative distance error by the length of the line.

#### ACKNOWLEDGMENT

The author wishes to thank Mr. Alan R. Drago of Trimble Navigation, Ltd., Herndon, Virginia, and Mr. David R. Doyle, Mr. David B. Zilkoski, and Mr. Edward J. McKay with the National Geodetic Survey, Silver Spring, Maryland, for their assistance in the preparation of this chapter.

#### REFERENCES

American Land Title Association and the American Congress on Surveying and Mapping. 1962 *Minimum Standard Detail Requirements for Land Title Surveys and Classifications of ALTA/ACSM Land Title Surveys*, ALTA and ACSM, Washington, D.C., updated 1986, 1988, 1992.

Buckner, R. B. 1983. *Surveying Measurements and Their Analysis*, Landmark Enterprises, Rancho Cordova, Calif.

Federal Geodetic Control Committee. 1974/1976. *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys*, National Geodetic Survey, Silver Spring, Md., February 1974, reprinted January 1976.

Federal Geodetic Control Committee. 1975/1978. *Specifications to Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys*, National Geodetic Survey, Silver Spring, Md., July 1975, reprinted May 1978.

Federal Geodetic Control Committee. 1988/1989. *Geometric Geodetic Accuracy Standards and Specifications for Using GPS Relative Positioning Techniques*, Version 5.0, National Geodetic Survey, Silver Spring, Md. 1974/1976.

Federal Geodetic Control Subcommittee. 1984/1990. *Standards and Specifications for Geodetic Control Networks*, National Geodetic Survey, Silver Spring, Md., September 1984, reprinted October 1990.

#### ADDITIONAL READING

Bouchard, Harry, and Francis H. Moffet, *Surveying*, 5th ed., International Textbook Company, Scranton, Pa., 1965.

Brinker, Russell C., and Roy Minnick, *The Surveying Handbook*, Van Nostrand Reinhold, New York, 1987.

Brinker, Russell C., and Paul R. Wolf, *Elementary Surveying*, Harper & Row, New York, 1984.

Davis, Raymond E., and Joe W. Kelly, *Elementary Plane Surveying*, McGraw-Hill, New York, 1969.

Davis, Raymond E., Francis S. Foote, James M. Anderson, and Edward M. Mikhail, *Surveying Theory and Practice*, 6th ed., McGraw-Hill, New York, 1981.

Mikhail, Edward M., *Observation and Least Squares*, Dun-Donnelly, New York, 1976.

Mikhail, Edward M., and Gordon Gracie, *Analysis and Adjustment of Survey Measurements*, Van Nostrand Reinhold, New York, 1981.

Reilly, James P. *Improving the Accuracy of Field Measurements*, P.O.B., Detroit, 1987.

Reilly, James P. *Improving Your Field Procedures*, P.O.B., Detroit, 1990.

Wolf, Paul R., *Adjustment Computations: Practical Least Squares for Surveyors*, 2d ed., Landmark Enterprises, Rancho Cordova, Calif., 1980.



## CHAPTER 15

# TOPOGRAPHIC SURVEYS

*Carlyle W. Blomme, R.L.S. / Mary M. Root, L.S. /*

*Eric V. Day, P.L.S. / Updated/Revised: Curt Sumner, L.S.*

### INTRODUCTION

While the boundary survey is the skeleton for the physical characteristics of property involved in a land development project, the topographic survey map represents the remainder of its anatomy chart. In mapping, topography is the delineation of natural and artificial features that define a given place. Although the topographic features of a property may have some significance to project feasibility and the rezoning process, their primary importance is initially realized in the preliminary design phase. The horizontal and vertical position of natural and man-made features will play a critical role in determining how a site will be engineered to suit the intended use of the property. Consideration must also be given to how these features relate to those lying on adjoining properties and within street rights of way or easements. Determining how utility services can be provided to the proposed development will be among the first preliminary design tasks to be undertaken.

Topographic information for areas well outside of the project site boundaries is often needed to make preliminary decisions regarding design considerations such as storm drainage patterns. When this is the case, the use of smaller scale topographic maps, such as those available from the United States Geological Survey (USGS) with a scale of  $1'' = 2000'$  or from local government agencies with a scale of  $1'' = 200'$ , is appropriate. As the design process begins to focus more on intricate details, larger scale maps are needed to insure that site engineering is as balanced as possible with regard to earth moving and the use of utilities. Actual map scales will be determined by the design team. The size and relative detail of a site will often determine the surveying methods employed. For large areas, aerial

photography and photogrammetry are an economical means of providing detailed maps. These surveys have become increasingly more accurate with improved technology. Field methods employing electronic survey systems may be used on other sites to create quality maps in a timely fashion. For small sites, other methods—such as alidade and planetable or baseline surveys—may prove more cost effective.

Experienced personnel, competent in the use of current equipment and who are familiar with modern techniques, are an essential ingredient in concluding surveys. Completion of field and office work includes attention to detail and accuracy. The meshing of field and office tasks is critical for the orderly work progression. Required tolerances for survey work are also important because incorrectly located data or data located from control points that do not meet required tolerance will result in improper design.

Only boundary location is more crucial to land development than the topographic survey. Because the topographic map is an integral part of the design process, omission of valuable information in a topographic survey can lead to costly problems.

### PLANNING TOPOGRAPHIC SURVEYS

Planning for topographic surveys includes consideration of the site's physical features, required scale and accuracy, contour interval appropriate for the type of project, cost, and type of instruments available. All of these factors affect the survey methods and equipment used in the topographic survey.

Among the most important aspects of planning a topographic survey is the communication between the designer and those who will collect the field data on which the map

will be based. A proper level of understanding of the design concept by those who collect the data will result in a more comprehensive survey and reduce the likelihood that numerous data gathering exercises will be required. The responsibility for insuring that this level of understanding is adequate lies with both parties. Neither should make assumptions about whether the understanding exists, nor be timid about asking questions to clarify issues.

When a comfortable level of understanding has been reached, a decision can be made concerning which method is to be employed to conduct the data gathering survey. Surveys to gather topographic data, like all other surveys, must begin with an appropriate control system. Therefore, the first step in planning for the topographic survey of a land development project is the completion of well-planned horizontal and vertical control networks. These networks must be geometrically strong and must be well monumented.

The number and location of the control points depends on the nature of the project and on the surveying method used. As a matter of convenience during the design and construction phases of the project, control points should be interspersed throughout the site. It may be cost effective to use traverse points located in the boundary determination as horizontal control stations for the topographic survey. Establishing control points as part of the boundary traverse loop and cross-ties through the middle of the site provide a network convenient for use throughout the design and construction phases of the project. To do this, adequate forethought on traverse location is needed at the time when the traverse is being set for the boundary determination. Provisions for the protection of survey monuments by contractors are necessary and cost effective.

The map scale required for the most demanding use controls mapping requirements. This is because map reduction for planning is generally satisfactory but enlargements do not usually produce the detail necessary for actual project design. Determination of the most important information requirements before performing the initial survey prevents the need for additional field trips by survey crews. A determination of the contour interval and the area of coverage are controlling factors when planning the fieldwork.

### Gathering the Data

Data must be gathered and compiled in such a way that it can be used for preliminary and final design. Historically, surveys have been conducted utilizing a plane table and alidade or a transit/theodolite and tape radial survey for site locations, or a baseline survey for linear projects such as roads and utility lines. None of these methods produces data that can be easily transferred into the digital environment used in today's design process. This does not imply that maps developed using these instruments are invalid. It simply means that their usefulness may be limited for topographic mapping of large sites and long-line surveys, or those with a significant number of features. As technology has advanced, the electronic distance-measuring device

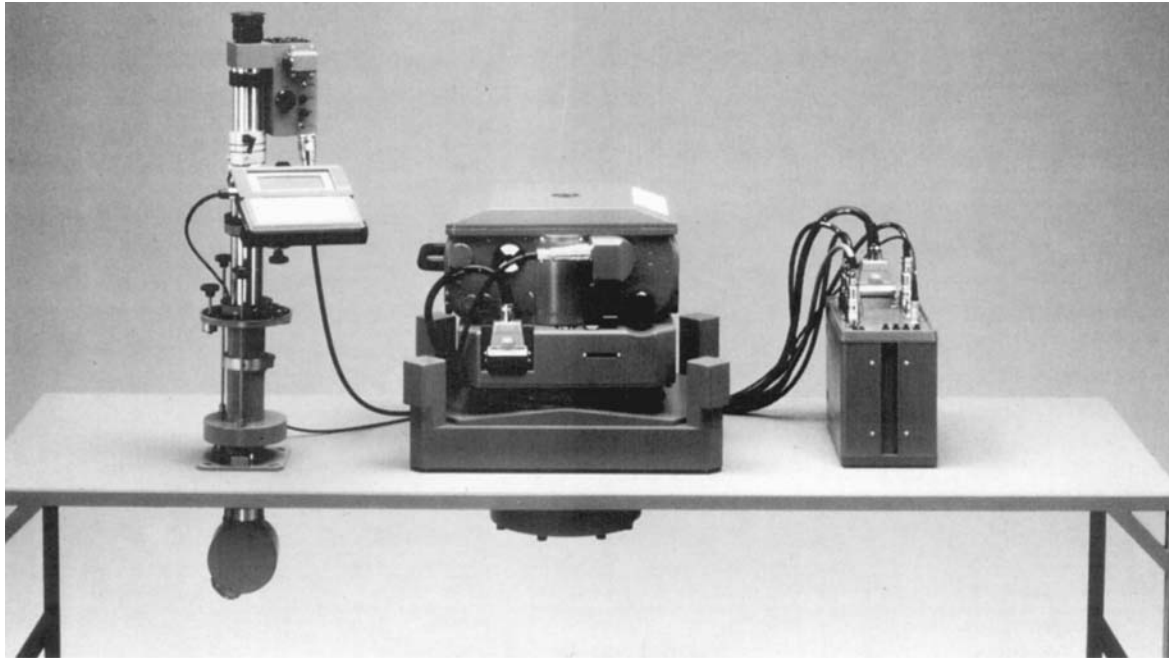
(EDM) and electronic data collector have replaced these instruments for field-run surveys. Developments that are more recent have combined these two into what is commonly referred to as a "total station." Photogrammetry, which uses aerial photographs combined with ground control points to generate topographic maps, has become the preferred method for surveying a majority of sites for land development design purposes. Small sites, particularly those with extensive features, are still commonly surveyed using an electronic data collection device. The maps created using photogrammetry can be either large or small scale. In some cases, the aerial photography is conducted in such a way as to allow both large and small scale maps to be created, thus providing a broader view of the project area while serving the preliminary and final design processes. The photogrammetric survey method is not appropriate for determining boundary lines. Both the electronically gathered field survey data and photogrammetric survey data can be compiled into digital topographic maps to serve as the base layer for design files.

The method selected for gathering data will influence the specific details of how it is to be accomplished. Regardless of the method chosen, it is critical that data be gathered so that compilation is done in a format acceptable to the designer. This is particularly true when the data is to be collected through a field-run survey. A part of the communication mentioned earlier between designer and surveyor should focus on the type of equipment and processing system the surveyor will use, and the type of design software the designer will use. Specific attention should be given to what layering system or data library is required. Data accurately collected and processed is not very useful unless it is presented in a format the designer can use. The reason this aspect of the process is so important for field collection of data is that the coding system for data collection should be established prior to the survey in order to simplify adjustments that may otherwise be difficult during compilation. In the case of photogrammetry or similar methods of gathering data, the issues of proper coding and formatting are addressed during compilation also, but there is no field survey component.

### PHOTOGRAMMETRY AND TOPOGRAPHIC MAPPING

Photogrammetry is the science and art of preparing accurate maps or obtaining reliable measurements from photographs. Photogrammetric mapping is the most common method used at present for preparing medium- and small-scale maps for use in land development work. In addition, large-scale maps can also be prepared photogrammetrically depending on the amount of ground cover on the site and the level of detail required. Two types of photogrammetry—terrestrial and aerial—are recognized. Aerial photogrammetry is by far the most common for topographic surveys for use in land development projects. (See Figure 15.1.)

Two different types of data can be obtained photogrammetrically. Metric data concerns information related to distance, area and elevation. This is of primary interest to the



**FIGURE 15.1** RMK TOP Aerial Camera System (Courtesy of Carl Zeiss, Inc. Thornwood, NY)

surveyor, land planner and developer. Interpretative data also comprises a major part of photogrammetry. Today this field of study has expanded to include interpretation of information related to the earth and the environment. The tools used for this field go beyond photographs and include multispectral scanners, side-looking airborne radar, thermal scanners, and other devices resulting from modern technology. The use of these tools for land development design has not yet been fully determined.

Photogrammetry is increasingly becoming the method most often used in topographic mapping. Its advantages include speed of coverage, cost, completeness of coverage, and the details included in such coverage. The disadvantages are that field surveys are still required to precisely locate features such as building corners, curb and gutter elevations, and details for underground utilities, as well as the fact that areas with tree cover cannot be effectively mapped when leaves or evergreen canopies are present. The principal instruments involved in photogrammetric mapping are the aerial camera and the photogrammetric plotter. The aerial camera is used to provide the photographs on which mapping is based. The photogrammetric plotter is used to convert the perspective projection on the photograph to an orthogonal projection on a map. Technologies have been developed through which converting the photography into mapping can be accomplished with a computer program. This method is known as “soft copy” mapping.

**General Procedures.** Mapping needed for land development often requires a series of maps showing different data and drawn at different scales. The basic data needed for topographic mapping are generally obtained by contracting with a photogrammetric mapping firm. Normally, the en-

gineering firm specifies scale and coverage required for the mapping, as well as the required final product (hard copy maps or computer files). Ground control can be contracted, or, in consultation with the photogrammetrist, can be provided by the firm responsible for land development planning. Both horizontal and vertical control networks will be required. Because the control surveys are expensive, every effort should be made to integrate the photogrammetric control surveys with the boundary and construction surveys anticipated for the project.

Once the photography and ground control has been obtained, the photogrammetric mapping firm will use a stereoplottor instrument to produce the finished maps or computer files. The maps (or files) prepared by the stereoplottor operator will include corrections for lens distortion, earth curvature, and other mapping errors.

While photogrammetrically prepared maps are basic to most land development planning, it is often necessary to perform simple measurements of distance, elevation, and position from aerial photographs. This is particularly true when making preliminary studies of a site for potential development. The basic plane and coordinate geometry associated with these measurements is discussed below.

### Vertical Photo Calculations

Figure 15.2 illustrates the relationships that exist between points on a vertical photograph and those points on the ground. This relationship is one of similar right triangles. The distance from  $o'$  to the focal point ( $f$ ) in Figure 15.2 is the camera focal length. This is the same length as the distance from the focal point to point  $o$ . This length ( $o-f$  and  $o'-f$ ) is part of the camera specifications used in photogrammetry and therefore is a known quantity. Assuming

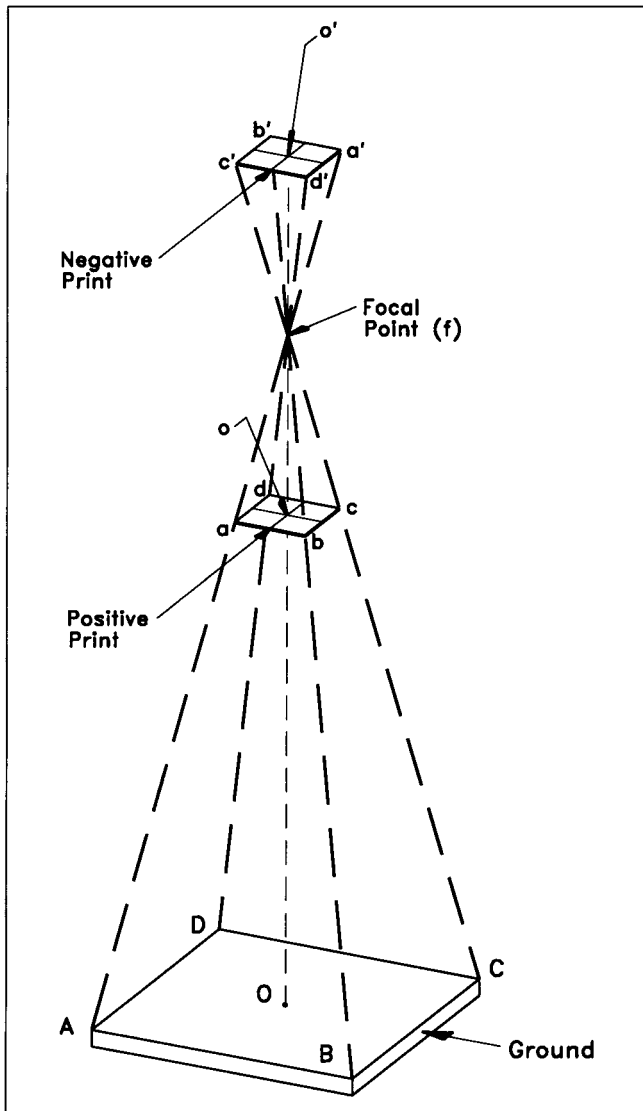


FIGURE 15.2 Vertical aerial photo geometry

the line  $o'-o$  lies in a truly vertical plane, then this line forms a perpendicular to the plane of the negative, the plane of the positive print and the plane of the ground. This relationship allows for computations using similar right triangles.

When using aerial photographs, the scale of the photo is foremost in importance. Since terrain varies in elevation from point to point, photo scale is a changing value and is therefore a relative term. Expression of the scale at any point on a vertical photograph depends on the elevation of that point. In Figure 15.3a, line  $A'B'$  represents the reference datum. The relief is nearly level as represented by line  $AB$ . A vertical photograph taken with a camera at  $L$  would show the length of the actual ground line  $AB$  as line  $ab$ . By similar triangles, the ratio  $ao/AO$  equals the ratio  $ob/OB$ . The scale of the photograph is uniform across the photograph. However, in a more realistic situation (Figure 15.3b), the variation in ground elevations is depicted by curve  $AO$  and  $OB$ . A vertical photograph taken from  $L$  would show

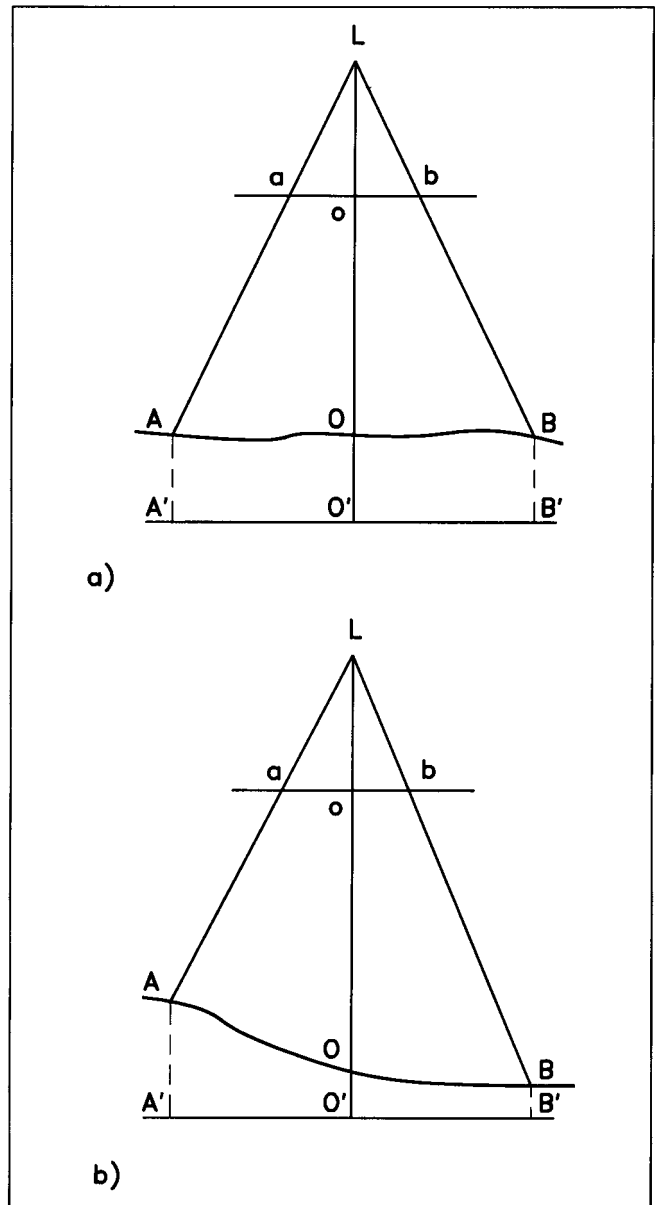


FIGURE 15.3 Effect of relief on scale

points  $A$ ,  $O$ , and  $B$  as  $a$ ,  $o$ , and  $b$ . Now the ratios  $ao/AO$  and  $ob/OB$  are not equal, and the scale of the photograph varies.

The variation in scale just noted is one reason why maps prepared from aerial photos require ground control measurements. These measurements are used to provide a constant scale between any points on the map.

The scale used most often in the early phases of a land development project is the average scale for the entire photograph. There are several ways to find an approximate scale value. When the actual ground distances between points in the photograph are known, then a ratio of photo length to ground length produces the photo scale.

Suppose for example, a football field shows clearly in a photograph, then the photo scale is obtainable. The distance from end to end of a football field is 120 yards. Sup-

pose the field measures 2.3 centimeters (cm) on the photograph. First, convert all units of measure into common units. In this case, the unit of conversion is feet. The 120-yard field is therefore 360 ft and the converted distance on the photo is 0.075 ft. The representative fraction is

$$\frac{\text{Photo Distance}}{\text{Ground Distance}} = \frac{1}{X} \quad (15.1)$$

$$\frac{0.075}{360} = \frac{1}{X}$$

therefore:

$$1:4800 \text{ or } 1 \text{ inch} = 400.0 \text{ ft}$$

Ratios between corresponding points on maps, of known scale, and those on aerial photos yield approximate photo scales. The average photo scale is one based on the average elevation of the covered area in relation to the datum plane. Expression of this scale is a ratio of the camera focal length ( $f$ ) to the flying height, minus the average elevation of the coverage area.

$$\text{Scale}_{\text{avg}} = \frac{\text{Focal Length}_{\text{camera}}(f)}{\text{Flying Height}_{\text{above datum}} - \text{Terrain Elevation}_{\text{avg}}} \quad (15.2)$$

The average scale determined from this relationship is obviously only as reliable as the input data. Scales derived in a similar manner for points on the photographs use the same relationship, except there is a substitution for the average terrain elevation with the particular point elevation.

Thus the scale at any point is

$$\text{Scale}_{\text{point}} = \frac{\text{Focal Length}_{\text{camera}}}{\text{Flying Height}_{\text{above datum}} - \text{Elevation}_{\text{point}}} \quad (15.3)$$

After determination of the photo scale, coordinates fixing point location provide the basis of computations for other metrical values such as distances and elevations.

The center of an aerial photograph, determined from the fiducial marks, is the principal point. This point is the origin for the (local) photo coordinates. Lines joining opposing fiducial marks intersect at the principal point of the photo. Designations of coordinates of points then follow the pattern illustrated in Figure 15.4.

Fiducial marks appear on vertical photographs but not always in the position shown in Figure 15.4. Some photographs have fiducial marks located at the four corners, and some cameras produce eight of these marks. Such marks, therefore, do not always have the shape or pattern shown in Figure 15.4.

Before beginning a discussion of photogrammetric calculations for photo coordinates, some words of caution are in order. When systematic errors of various types exist, a consideration of them becomes necessary to attain high-quality results from aerial photographs. Such errors as shrinking or stretching of the photograph material, errors

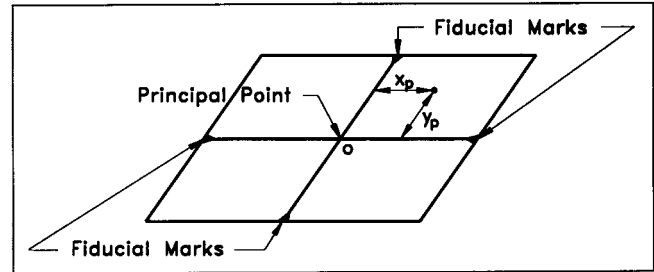


FIGURE 15.4 Defined points on vertical photographs

in camera lens manufacturing and errors caused by a failure of the fiducial axes to produce the proper principal point lead to systematic flaws in the results. Distortions caused by the curvature of the earth and atmospheric refraction cause similar problems. Corrections made for these and other errors are proportional to the desired accuracy of the output.

### Photo Coordinates

For most computations involving aerial photographs, some ground control precedes the computations. However, this does not mean that the control surveys precede the taking of photographs. Control is both horizontal and vertical. Photo control differs from other control surveys because spacing in relation to the photographs is critical. The number of control stations and their location depends on the accuracy requirements for the survey.

As with other control surveys, required accuracy primarily dictates the equipment and surveying procedures used to carry out the work. In cases involving land development, this control will later serve as the control for additional survey work. The criterion for accuracy is therefore the same as that demanded for the most exacting work based on the control. First- and second-order control stations, set by National Geodetic Survey, state geodetic agencies, and others, serve as primary control. Additional control for most development projects requires second order surveys.

Horizontal positions based on geodetic latitude and longitude or northings and eastings of state coordinate systems serve best for land development control because they eliminate the problem of errors due to curvature of the earth and allow broad area coverage while retaining satisfactory survey precision. Vertical control should be on the same datum to which utility ties will be made.

Once having established horizontal and vertical control, one proceeds with finding other values necessary to computations directly from the photographs. One of those values is flying height ( $H$ ) as measured from the reference datum. Finding the value for flying height is obtained using two points with known (non-identical) elevations. The following example illustrates this procedure.

Suppose the camera used in the photography has a focal length of 152 mm. Assume that a control line measured between two points ( $A$  and  $B$ ), as shown on a photo taken



with the camera mentioned, is 1478 ft in length and that their elevations, determined during the vertical control survey, are 1677 ft and 1704 ft, respectively.

Starting at the principal point of the photo as shown in Figure 15.4, carefully measure the photographic coordinates of the ground points (A and B). With these photo coordinates, find the photo distance between the two points. Use Eq. 15.4 for finding the approximate flying height.

$$\frac{a \text{ to } b_{\text{Photo Distance}}}{A \text{ to } B_{\text{Ground Distance}}} = \frac{f_{\text{focal length}}}{H_{\text{Flying Height}} - h_{\text{avg. elev. of ground points A and B}}} \quad (15.4)$$

Complete the remainder of the problem using trial and error procedures, thus obtaining the actual flying height. The formulas involved follow

$$X_A = \frac{H - h_A}{f} x_a \quad (15.5)$$

$$X_B = \frac{H - h_B}{f} x_b \quad (15.6)$$

$$Y_A = \frac{H - h_A}{f} y_a \quad (15.7)$$

$$Y_B = \frac{H - h_b}{f} y_b \quad (15.8)$$

Next, determine distance:

$$A \text{ to } B = \sqrt{(X_A - X_B)^2 + (Y_A - Y_B)^2} \quad (15.9)$$

where:

$H$  = flying height (in the same units as ground elevation  $h$ )

$h$  = ground elevation at indicated point (A or B)

$f$  = Camera focal length (in millimeters)

$X$  or  $Y$  = ground coordinates at indicated ground points (A or B) and

$x$  or  $y$  = photo coordinates (in millimeters) at indicated photo points ( $a$  or  $b$ ).

Make a comparison with this computed value for the line  $AB$  and the measured length of the same line from the control survey. It is unlikely that they will agree, and therefore a revised value for  $H$  is necessary. Calculate a new  $H$  value using the following equation:

$$\frac{\text{Correct Value for A to B}}{\text{Computed Value for A to B}} = \frac{\text{New Value for H}}{\text{Last Approximated Value for H}} \quad (15.10)$$

Computations follow for the data given above. The photo coordinates for point A are  $x_a = -12$  mm and  $y_a = -17$  mm. Coordinates at point B are  $x_b = 7.3$  mm and

$y_b = 75$  mm. From these coordinates the photo distance representing the control line  $AB$  is 94 mm. Using Eq. (15.4), determine the approximate flying height as 4080 ft.

$$X_A = -189.7 \quad Y_A = -268.8 \quad (15.11)$$

$$X_B = +114.1 \quad Y_B = +1172.4$$

Use this value and Eqs. (15.5) to (15.8) for finding trial coordinates at points A and B. The following values result from the first trial.

Once you have determined trial values for the ground coordinates of the given points, use Eq. (15.9) for calculating the distance between the trial points. In this example, the computed value between trial points is 1061 ft. The actual distance between these points from the ground control survey is 1478 ft. The flying height is therefore in error, and the necessary adjustment by Eq. (15.10) follows.

$$\frac{1478}{1473} = \frac{\text{New Value for H}}{4080} \quad (15.12)$$

$$\text{New Value for H} = 4094$$

Having a new trial flying height, calculate new trial coordinates from Eqs. (15.5) through (15.8). Using these new values, find the matching distance between the coordinates using Equation (15.9), as illustrated above. Again, compare this distance to the actual ground distance. Repeat the cycle until the computed distance between the coordinates equals the actual ground distance. The desired degree of accuracy determines the number of repetitions needed. In the example, flying height is 4088.5 ft. At this height, the computed result of line  $AB$  is 1478.06 ft.

The scale at the datum plane is determined from the following equation.

$$\text{Scale}_{\text{datum}} = \frac{\text{Camera Focal Length}}{\text{Flying Height}} \quad (15.13)$$

$$\begin{aligned} \text{Scale}_{\text{datum}} &= \frac{152 \text{ mm}}{4088.5 \text{ ft}} \\ &= \frac{1}{8198} \text{ or } 1 \text{ in.} = 683 \text{ ft} \end{aligned}$$

The scale at average ground level on the same photograph is

$$\text{Scale}_{\text{average}} = \frac{\text{Camera Focal Length}}{\text{Flying Height} - \text{Elevation}_{\text{average}}} \quad (15.14)$$

$$\begin{aligned} \text{Scale}_{\text{average}} &= \frac{152 \text{ mm}}{4088.5 - 1690.5} \\ &= \frac{1}{4808.6} \text{ or } 1 \text{ in.} = 400.7 \text{ ft} \end{aligned}$$

For best results the procedure just described should be

repeated with a second set of points. The second set should be located at right angles to the first.

### Determination of Object Heights from Aerial Photographs

Object height from measured relief displacement of an object is but another use of photographs. Relief displacement is the apparent movement that occurs along lines that radiate from the principal point of a vertical photograph. The movement is a result of elevation above or below the datum plane of the photo and the central perspective projection provided by the photograph. This displacement is then proportional to the radial distance from the principal point of the photo to the top of the object for which height is desired as the height of that object is proportional to the flying height, as expressed by Eq. 15.15.

$$\begin{aligned} \text{Object Height} & \quad (15.15) \\ &= \frac{\text{Displacement} \times \text{Flying Height}}{\text{Radial Distance to the Top of Object}} \\ &= \frac{(r - r_1) \times H}{r} \end{aligned}$$

where:

$r_1$  = The distance from the principal point to the bottom of the object

$r$  = The distance from the principal point to the top of the object

$H$  = The Flying Height

Assume that the radial distances to the top and bottom of a building from the principal are 42.0 mm and 40.9 mm, respectively. The flying height above the ground as calculated in a previous example is 2398 ft. The following example illustrates the method of determining the height of the building.

$$\begin{aligned} \text{Object Height} &= \frac{(42 - 40.9) \times 2398}{42} \quad (15.16) \\ &= 62.8 \text{ ft} \end{aligned}$$

### Finding Distance Between Points of Known Elevation

With the flying height and elevations of two points known, it is possible to find the horizontal distance between two points on a photograph. Equations 15.5 through 15.9 comprise the basis for determining the desired distance. Subscripts change from  $A$  and  $B$  to  $C$  and  $D$  as appropriate for the points used.

Assume that two points,  $C$  and  $D$ , lie on the photograph used earlier to find the flying height. The focal length of the camera was 152 mm. The respective elevations of the two points are 1649 and 1692 ft. The photo coordinates of the two points as measured from the principal point are  $x_c = 5$  mm,  $y_c = 15$  mm,  $x_d = 60$  mm and  $y_d = 28$  mm.

The flying height determined earlier was 4088.5 ft. Using these values, substitute them in the above mentioned equations. The corresponding ground coordinates are:

$$X_c = 80.2 \quad Y_c = 240.7 \quad (15.17)$$

$$X_d = 946.0 \quad Y_d = 441.5$$

Inserting these values into Eq. (15.9) gives the length of line  $CD$ .

$$\begin{aligned} CD &= \sqrt{(80.2 - 946.0)^2 + (240.7 - 441.5)^2} \quad (15.18) \\ &= 889 \text{ ft} \end{aligned}$$

The problems presented to this point involve work on single photographs. For the resolution of elevations of ground points, use two lapping photographs each containing the points under observation. Mathematical linkage of these photographs is necessary.

### Determination of Air Base in Aerial Photogrammetry

Air base, the term given to this linkage, is the ground distance between exposure stations of adjoining photographs. It is the distance between exposures that causes the difference in parallax angles. This allows the user to view ground relief stereoscopically and thus discern depth in photographs. Measuring the difference in parallax enables determination of elevation. These calculations center on the relationship of the ground distance between photo exposures to that of the flying height above the average terrain elevation.

Calculations for air base involve finding the absolute parallax of a point. To accomplish this, measure the photographic coordinates of the point on two adjacent photos. Once having accomplished this, a stereocomparagraph or contour finder serves to find elevations at other points more rapidly.

Following is an example on how to find the air base. Use Eq. (15.19) to find the absolute value for parallax.

$$p = x - x_1 \quad (15.19)$$

where:

$p$  = Absolute parallax

$x$  = The photograph coordinate of the point of known elevation as measured on the right photograph

$x_1$  = The photograph coordinate of the point of known elevation as measured on the left photograph

Equations (15.20) to (15.23) compose the parallax formulas leading to air base determination. The solution is one by trial and error, much like that for flying height. In these equations, substitute a trial value for the air base  $B$  for use with the scaled coordinate values from the pair of photos in use. The absolute parallax  $p$  also comes from these last mentioned values.

$$X_{A_{\text{Ground Coordinate Elev. Known}}} = \frac{\text{Air Base}}{\rho_a} X_{a_{\text{Photo Coordinate}}} \quad (15.20)$$

$$Y_{A_{\text{Ground Coordinate Elev. Known}}} = \frac{\text{Air Base}}{\rho_a} Y_{a_{\text{Photo Coordinate}}} \quad (15.21)$$

$$X_{B_{\text{Ground Coordinate Elev. Unknown}}} = \frac{\text{Air Base}}{\rho_b} X_{b_{\text{Photo Coordinates}}} \quad (15.22)$$

$$Y_{B_{\text{Ground Coordinate Elev. Unknown}}} = \frac{\text{Air Base}}{\rho_b} Y_{b_{\text{Photo Coordinate}}} \quad (15.23)$$

With trial values of the ground coordinates now available, check the measured distances between the points used against the computed trial distance using Eq. (15.24). When these values are equal, the air base is correct.

$$A \text{ to } B = \sqrt{(X_A - X_B)^2 + (Y_A - Y_B)^2} \quad (15.24)$$

where the Xs and Ys are determined from Eqs. (15.20) to (15.23).

After completing the trial and error solution for the air base, resolve the flying height by substitution in Eq. (15.25). Equation (15.25) also provides the parallax of future points of unknown elevations.

$$\begin{aligned} H_{\text{Flying Height}} - h_{\text{Known Ground Elev.}} \\ = \frac{\text{Air Base}}{\rho_{\text{Absolute parallax}}} \text{Camera Focal Length} \end{aligned} \quad (15.25)$$

For an example, assume the length of a control line from points A to B is 2137 ft. Assume the elevation at point A is 907 ft. Two photographs available each show points A and B. Measurements made of the two points on both photos yield the results shown in Table 15.1.

Using Eq. (15.19), solve for  $p_a$  and  $p_b$ .

$$\rho_a = 17.6 \text{ mm} - (-52.8 \text{ mm}) = 70.4 \text{ mm} \quad (15.26)$$

$$\rho_b = 118.6 \text{ mm} - (47.9 \text{ mm}) = 70.7 \text{ mm}$$

Next, find ground coordinates using Eqs. (15.20) to (15.23). Assuming a trial air base value of 2000 ft, the results are as follows:

$$X_A = \frac{2000 \text{ ft}}{70.4 \text{ mm}} \cdot 17.6 \text{ mm} = 500.0 \text{ ft} \quad (15.27)$$

$$Y_A = \frac{2000 \text{ ft}}{70.4 \text{ mm}} \cdot -9.3 \text{ mm} = -264.2 \text{ ft}$$

$$X_B = \frac{2000 \text{ ft}}{70.7 \text{ mm}} \cdot 118.6 \text{ mm} = 3355.0 \text{ ft}$$

$$Y_B = \frac{2000 \text{ ft}}{70.7 \text{ mm}} \cdot -109.8 \text{ mm} = 3106.1 \text{ ft}$$

Substituting these values into Eq. (15.24) gives the computed length of line AB as 4417 ft. Since the actual

measured ground length is 2137 ft, the trial air base is in error. Solve the following proportion or a new trial air base.

$$\frac{\text{Actual Ground Length}_{A \text{ to } B}}{\text{Computed Length}_{A \text{ to } B}} : \frac{\text{Corrected Air Base}}{\text{Trial Air Base}} \quad (15.28)$$

Solving the proportion above yields a new value for the air base of 1061 ft. Use this value and again solve Eqs. (15.20) to (15.23). The calculated length of line AB equals the measured length, and therefore the air base is now correct and equals 1061 ft.

Proceed with the solution of Eq. (15.25) to determine flying height. For the given example, the results of the computation follow.

$$\begin{aligned} H_{\text{Flying Height}} &= \frac{\text{Air Base}}{\rho_{\text{Absolute parallax at point of known elev.}}} \\ &\quad \times \text{Camera Focal Length} + h_{(\text{Known point elev.})} \end{aligned} \quad (15.29)$$

$$H_{\text{Flying Height}} = \frac{1061}{70.4 \text{ mm}} 152 \text{ mm} + 907 \text{ ft} = 3197.8 \text{ ft}$$

The completion of the calculations for the length of the air base and the flying height provides necessary information for determining elevations of other ground points in the same photos by solving Eq. (15.25) for  $h$ .

To illustrate, begin by assuming a need for an elevation at some ground point. With the air base, flying height and camera focal length now known, measure the photo coordinates of the new point. The best method of determining these measurements is with a stereocomparator for results of higher order, but a scale will suffice for less accurate elevations.

Those measured coordinates of the point C are as follows: Left photo = 59.5 mm, Right photo = -10.6 mm. Substituting these coordinates in Eq. (15.19), the absolute parallax ( $p_c$ ) is 70.1 mm. The elevation of point C is therefore:

$$\begin{aligned} h_{\text{elev.}} &= H_{\text{Flying Height}} \\ &= \left[ \frac{\text{Air Base}}{P_c} \text{Camera Focal Length} \right] \\ &= 3198 \text{ ft} - \left[ \frac{1061 \text{ ft}}{70.1 \text{ mm}} 152 \text{ mm} \right] \\ &= 897.4 \text{ ft} \end{aligned} \quad (15.30)$$

There was no consideration given to tilt in photographs in any of the example problems presented. This basic introduction in the use and computations involved in photo interpretation allows for marginal use with minimum investment in equipment.

Photogrammetry for land development projects involves gathering of extensive topographic data. Such a collection process often involves many specialists including engineers, surveyors, planners and photogrammetrists. Processing the amount of information necessary for such projects involves

**TABLE 15.1 Left Plot Right Plot**

BEARING	1/2 DEFLECTION ANGLE	SIN	RADIUS	CHORD
N 64° 00' E	29° 30'	0.4924	10 units	9.85 units
S 57° 00' E	23° 30'	0.3987	10 units	7.97 units
S 10° 00' E	37° 00'	0.6018	10 units	12.04 units
S 64° 00' W	26° 00'	0.4384	10 units	8.77 units
N 64° 00' W				

use of stereoscopic plotters. There are several types of plotters available but the analytical plotters provide accurate coordinates to a computer for transfer to Land Information Systems or use in Computer-Aided Design Systems for mapping purposes.

## FIELD METHODS FOR TOPOGRAPHIC MAPPING

### General Procedures

Field methods for topographic mapping require that the surveyor accurately locate the positions of both the topographic and planimetric features on the site. Field methods are necessary in land development where detailed location of surface and subsurface utilities is required, where a limited area is involved, or where slope and drainage are critical. In most cases, this requires the location of a large amount of data, which must be presented on a large-scale map.

The positional bases for topographic mapping are the horizontal and vertical control networks. These may be established prior to topographic mapping. Alternatively, control network extension may proceed simultaneously with the topographic survey. Ideally, horizontal control stations should be located so that the topographer will have sufficient room to work, and so that he or she will be able to map the entire area necessary. Points to be mapped are typically located by radial observation from the horizontal control network. The elevations of points sighted are computed trigonometrically or by use of differential leveling.

Instrumentation formerly employed in the topographic surveying field is rapidly being replaced by what is generically known as the electronic total station surveying system equipped with an electronic data collector (see Figure 15.5). This equipment has the greatest potential for producing high accuracy results with a minimum expenditure of time in any area where detailed topography is required. The re-

cently developed robotic total station instruments allow one person to gather data for a topographic map rather than the typical two- or three-person field crew; however, it is sometimes still effective to have crew members performing activities such as drawing sketches and measuring manhole



**FIGURE 15.5** Total station (on foreground tripod), data collector (attached to leg of tripod), and prism (on background tripod). (Courtesy of Ray Root)

depths while the robotic station operator is obtaining other data. Of course, the former means of measuring detailed topography existed for ages before electronics came into use, and these are still used today. For instance, the plane table with alidade has been a mainstay of topographic surveying for hundreds of years, while the theodolite with tape and/or stadia rod has been widely employed throughout the past century.

All of these topographic measuring systems have advantages and disadvantages. In comparison, we note that while the electronic system requires a high initial acquisition and training investment, the potential for enhanced production will quickly prove the instrumentation to be superior in most surveying situations. The electronic system can provide highly accurate data, which can easily be loaded into a computer. The link between field and office is no longer miles of distance and volumes of notes but is reduced to the simplicity of an interface cable. The computer dramatically expands the usability and accessibility of the topographer's data. Multiscaled plans for all sorts of land development may be generated from the data without physical manipulation of hardcopy manuscripts.

In contrast to the electronic system, the alidade provides a perishable, single-scale drawing, of limited accuracy, which must be handled repeatedly by draftsmen and designers in order to produce finished plans. However, if a single plan with contour tracing is sufficient for the need, the alidade may be best.

The theodolite and tape and/or stadia rod are the immediate predecessors of the present total electronic survey system. All of the data collected by this means may ultimately be used with advantage in a computer environment; however, the means of recording the information and entering it into the computer is manual and, therefore, inefficient. Use of this data requires much entry time and is more susceptible to human error.

**The Total Station and the Data Collector Electronic Mapping Using the Total Station.** To begin, the surveyor first occupies an existing horizontal control station. He or she then orients the instrument to another station in the network. Orientation is typically made by sighting on a prism mounted on a plumbing pole held at the backsight station. Alternatively, pointing on a tripod-mounted prism can be used. A typical prism mounted on a plumbing pole is shown in Figure 15.5. The height of the plumbing pole (and the tripod mounted prism if used) must be recorded in the data collector, together with other information required for the job.

The data collector usually has a series of user prompts to begin collection at each station. Typically, the prompts begin by asking for a file name, a user's name, date, temperature, and barometric pressure, which is then followed with prompts for the set-up: instrument point number (or traverse station number), height of instrument, backsight point number, and prism height. When the instrument op-

erator then backsights the prism, the data collector records the raw data, consisting of the horizontal angle, vertical angle, and distance. The distance and the difference in elevations between the two known stations is verified with the known data.

With the internal data file initiated, the instrument oriented, and the data collector initialized, the topographic data collection begins. Each type of feature usually has a numerical or alphabetic code. Codes may be augmented with a brief description. The data collector prompts the operator for point number, description and code, prism height, and a choice of pre-set measurement sequences. The last of these entries activate the total station instrument, and the raw data is recorded. This information is displayed on the data collector for the operator to approve, reject, or edit. If approved, the collector advances to the next point number, ready for the next shot. If rejected or edited, the bad shot is retained and labeled as such, and the next point number is utilized for the correct information. This is useful for cross-checking the data. Sample codes and abbreviations for data collector use are shown in Figure 15.6.

For complex sites or features, a detailed sketch must be prepared to accompany the collector print-outs (see Figure 15.7). The sketch should include outlines of buildings and structures, dimensions, measurements, types of materials, sizes of pipe, descriptions, data collection point numbers, and any other pertinent facts.

An efficient way to collect the data is to shoot each feature continuously, as much as circumstance permits. This enables the data collector operator to repeat-key some of the shots, and offers the software consecutive shots for creating line sequences. For example, along a road the prism handler may locate the edge of pavement along one side as far as practical directed away from the instrument. The centerline can be located with continuous points in the direction towards the instrument and then return up the other edge of pavement. This method for location is sometimes helpful to the person who processes the data in the office because it keeps a consecutive series of point numbers for a particular feature. It is not mandatory, however, to collect data in this manner. Most software used to process raw field data will recognize points with the same code and combine them to draw the feature represented by the code. The shots should be placed at horizontal changes in direction or vertical changes in elevation, and/or at the distance interval commensurate with the map scale; e.g., a map scale at  $1" = 50'$  requires at least one shot every 50 ft.

Since topographic surveyors must translate a three-dimensional terrain to a two-dimensional map, the actual success of a topographic survey depends upon the location of essential points, which define a particular terrain. Every ridge, gradebreak, toe or top of slope is studied carefully for the exact placement of the location point that tells the story. Every manmade feature is similarly interpreted and delineated by a series of adeptly placed shots. Because a computer generates the contour lines, care must be taken

<p>01 - Horizontal Control TRV PXP - Panel Point</p>	<p>11 - Centerlines CL. Centerline Road</p>	<p>33 - Electric ELE</p> <hr/> <p>35 - Building BLD</p>
<p>05 - Property/Boundary IPF EIP Existing Iron Pipe EIR Existing Rod/Rebar/Pin GDR EPT Existing Pinch Top EIO Other (Describe) MKT Marked Tree N North S South E East W West MON Existing Conc. Mon. NIP New Iron Pipe PK. P.k. Nail RW Right of Way Mon. STN Stone</p>	<p>17 - Pavement EP. Edge of Pavement EPL EP Left EPR EP Right</p> <hr/> <p>18 - Curb TC. Top of Curb TCL Top of Curb Left TCR Top Curb Right BC. Back of Curb (Top) FC. Face of Curb (Flow)</p> <hr/> <p>19 - Concrete CON CCP Corner Concrete Pad EC. Edge of Concrete SW. Edge of Sidewalk</p>	<p>37 - Utilities Misc UTL OU. Overhead Utility PD. Telephone Pedestal UU. Underground Utility CTV Cable TV MCP Utility Marker Post UMN Utility Manhole VLT Utility Vault</p> <hr/> <p>42 - Miscellaneous MIS MB. Mail Box RS. Road Sign GL. Ground Light</p>
<p>07 - Grade Elevations SPT GR. Ground Spot Shot GRP Pavement Spot Shot GRC Concrete Spot Shot GRG Gravel Spot Shot</p>	<p>20 - Shoulder SHL</p> <hr/> <p>21 - Parking Stripes/Paint Lines PRK</p> <hr/> <p>22 - Utility Poles UPL GP. Guy Pole GW. Guy Wire LP. Light Pole PP. Power Pole SP. Service Pole TP. Telephone Pole</p>	<p>44 - Wetlands WET</p> <hr/> <p>58 - Break BRK TB. Top of Bank BB. Bottom of Bank</p> <hr/> <p>60 - Walls WAL CW. Centerline Wall EW. Edge of Wall</p>
<p>08 - Stream/Pond/River STR Stream DL. Ditch Line WL. Edge of Water</p>	<p>23 - Water WAT HYD Fire Hydrant MW. Monitoring Well PIV Post Indicator Valve WL. Waterline WE. Well WM. Water Meter WV. Water Valve</p>	<p>61 - Test Pit TP.</p> <hr/> <p>62 - Unimproved UNI PAT Trail or Path EG. Edge of Gravel DT. Edge of Dirt</p>
<p>09 - Vegetation VEG OB Ornamental Bush OT Ornamental Tree HDG Hedge WDL Woodline</p> <p>TR Tree A (Cherry), Size (In.) B (Beech), Size (In-&gt; C (Cedar), etc... D (Cottonwood) E (Elm) F (Ash) N (Tupelo) G (Locust) P (Pine) H (Hickory) Q (Oak) J (Burch) R (Dogwood) K (Sasafrass) S (Sycamore) L (Sweet Gum) T (Poplar) M (Maple) Z (Misc.)</p>	<p>26 - Storm STM BXC Box Culvert CMP Corrug. Metal Pipe CPP Corrug. Plastic Pipe DIP Ductile Iron Pipe DIG Drop Inlet-Curb Grt. DIS Drop Inlet-Curb Std. RCP Reinf. Conc. Pipe SDM Storm Drain Manhole TCP Terra Cotta Pipe YIG Tard Inlet-Grate YIS Yard Inlet-Std. I (Invert), Size (In.) T (Top), Size (In.)</p> <hr/> <p>29 - Sanitary SAN CO. Cleanout SSM Sanitary Sewer Manhole</p>	<p>63 - Railroad FR. Frog PS. Point of Switch RR. Railroad IR Inside of Rail CL Centerline of Rail</p> <hr/> <p>Desc. Format for Point L L - P P P D S . . . . . --Sub Desc. ----Draw Code -----2 or 3 Char Point Descriptor -- , - , Das Character -- 2 Digit Layer Code</p> <p>Use a "." (Period) as a Place Holder When There Is Only 2 Chars in the Point Descriptor and in Place of the Draw Code When There Is None.</p>
<p>10 - Fence FEN Fence GDR Guard Rail GPP Guard Pipe Post</p>	<p>32 - GAS GAS GL. Gas Line GM. Gas Meter GV. Gas Valve</p>	<p>Draw Codes (7th Position) L - Line C - Curve Z - End Draw Sequence Y - Close into Initial Point</p>

FIGURE 15.6 Codes and abbreviations used for data collectors

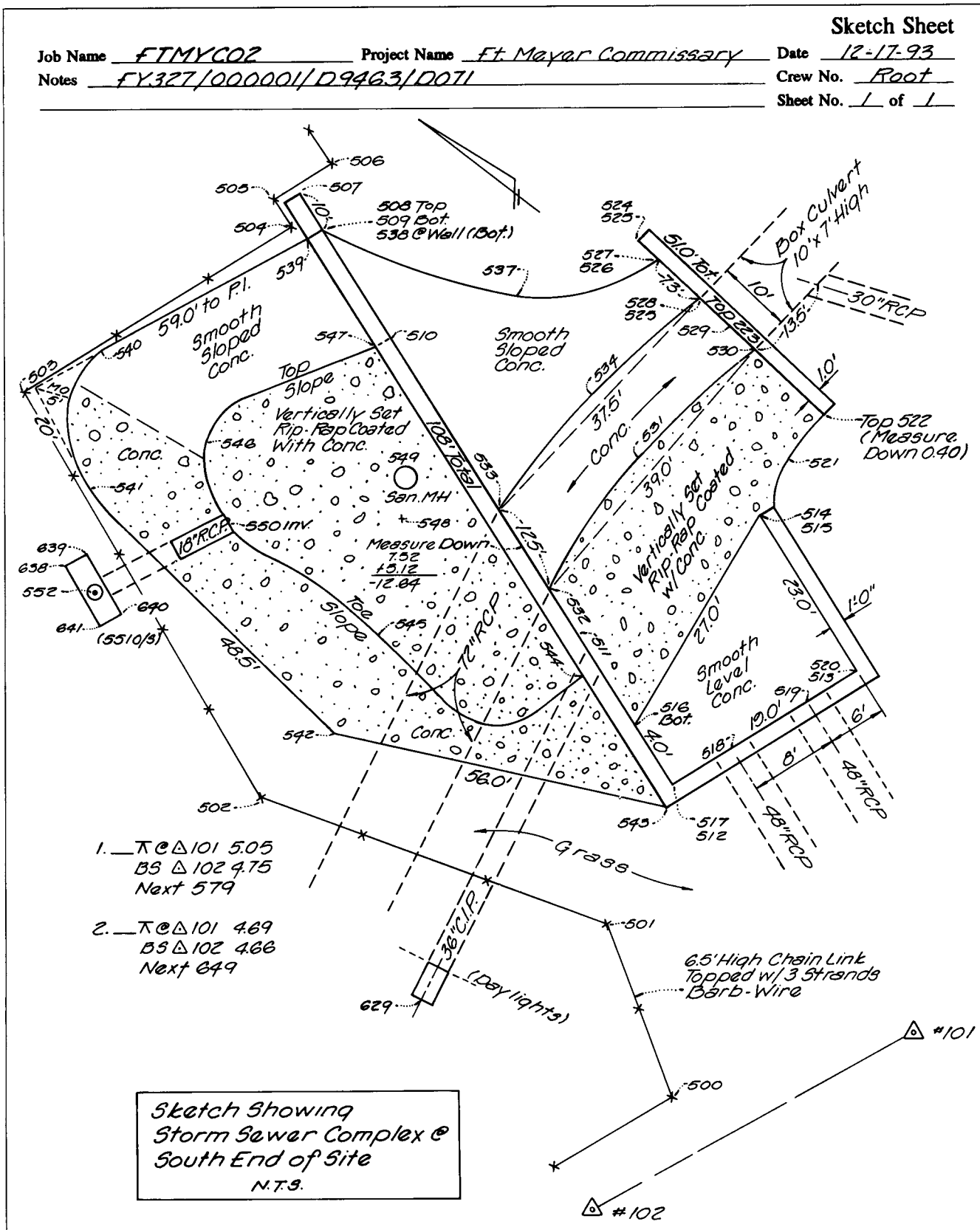


FIGURE 15.7 Sample field sketch to accompany data collection raw file

to space the shots in such a way that they do not appear to cross each other, particularly in curves (see Figure 15.8). In addition, the prism handler must select obstruction-free sight lines to the instrument, change the height of the prism as necessary, and maintain constant communications. Obviously, having an experienced topographer on as the prism handler expedites matters. This person can also confidently “call” shots; that is, relate new information to the previously-made shot. Many data collectors possess a feature that enables one to reference a shot left, right, back, ahead, up, and down in relation to the last instrument-shot point. This is particularly useful in thickets, woods, obstructed places, and steep terrain where sightlines may have to be selected and/or cut out with brushhook or machete. Under such circumstances, an experienced topographer distinguishes between the essential, the necessary, and the optional shots, and makes the shots accordingly.

Topographic mapping using electronic data collection has become the standard for field-run surveys. Software continues to be developed that improves the field-to-office conversion of data into mapping. Purchasing the proper equipment and developing a system for field and office procedures that enhance ones ability to make those conversions faster and more efficiently is critical to the ultimate success of a land development project.

The use of Global Positioning System (GPS) equipment to collect topographic data is becoming more prevalent. Advances in the satellite constellation, real-time computation of point positions, and the reduction of equipment costs have led to a wider use of GPS equipment for a variety of

surveying activities. As the required investment in equipment decreases and new technologies enhance the surveyor’s ability to use the equipment, the use of GPS for data collection related to topographic mapping may become a standard practice.

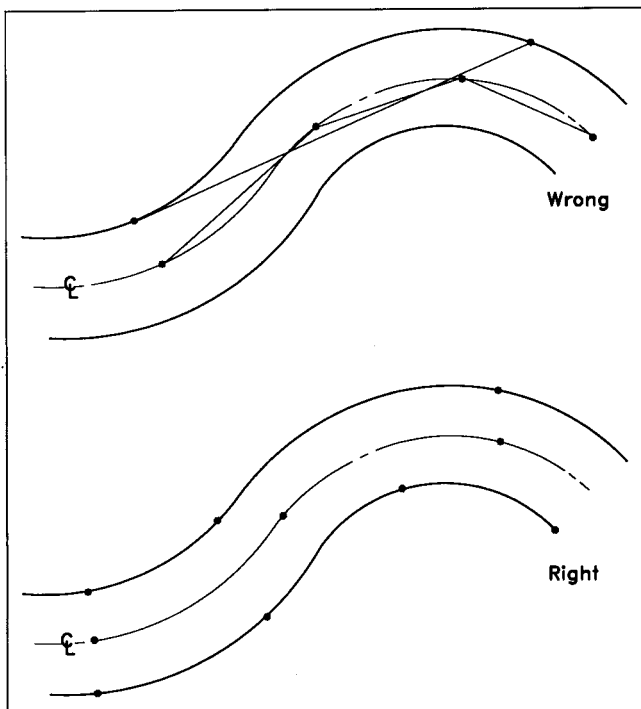
### Coordinating Field and Office Procedures for Electronic Survey Systems

Because of recent advances in computer science and survey software (e.g., CADD), it is easy to be misled by the premise that the machine, combined with knowledgeable programmers and operators, can provide a solution for every problem. The office practitioner, however, is well aware that no software comes close to handling the everyday situations in the real world. The honest computer operator will also acknowledge that the quality of the end product rests predominately within the skills and knowledge of the field topographer, and not with the computer. An experienced topographic surveyor performs two functions. First, the surveyor will have mastered the art of field data collection, good surveying practice, and the proper employment and care of his equipment. Second, the surveyor feels equally at home in the office, making daily use of the mathematical sciences. The surveyor will be conversant with the operation of various computing systems and software capable of producing a topographic map. This individual will have access to the client, planner and/or engineer who will be using the survey. Having made inquiries of all end users, and having been supplied with field sketches and electronically collected field data augmented by visual field inspection, the surveyor generates a three dimensional graphical/mathematical model of the project, henceforth referred to as the topographic survey map.

Realistically, however, a somewhat different situation exists in a typical multi-service land development firm. Here, production is balanced against optimal achievement, yielding a useful, cost efficient product. In this setting, the field topographer’s job is often completed by an office technician. A good deal of care is needed in communications, if the work is to be completed on time and within budget.

Upon selection of electronic data collection as a means of creating the topographic survey map, office procedures begin with a meeting between the field party chief and the surveyor of the computing department. A lack of communication between these individuals will usually result in error, frustration, and inefficiency. The large majority of topographic surveys are done in concert with other field tasks such as boundary survey or stakeout for clearing, soil borings, or improvements. In most instances, knowledge of foregoing survey operations will reside within the files of the computing department.

The party chief, in collaboration with the computing department personnel, must establish a data format. Each point located in the field must be uniquely identified as to horizontal location, vertical elevation, and physical character. Depending on data collector and computer software



**FIGURE 15.8** Data collection shot placement along curves



flexibility and compatibility, some or all of these values may be catalogued within the data collector for delivery to the computer. Point numbers, subject-specific or layer-specific numeric codes, and physical descriptions and notations are often used. In most instances, a standard set of codes and descriptive abbreviations alleviate problems with interpreting field data.

Before entering the field, the computing department may download existing traverse control and other pertinent coordinate data into the data collector. Whether such coordinate data is available or used within the data collector, the computer must be able to assign a unique identity to each three-dimensional collection point. Most coordinate geometry software uses point number identification. In this case, a block of point numbers must be reserved. Failure to do this will eventually result in the destruction of overlapping data, extensive editing of point numbers in the field data book, and unpleasant confrontation between office and field personnel.

Before going to the field, the party chief should review the project with the computing department surveyor in order to set survey priorities. What datum will be used? What accuracy and precision are acceptable under the scope of services? What will the horizontal scale and contour interval be? Will extra observations be needed to demonstrate minute vertical detail (that is, top of curb, inlet grates, pond risers and pipe inverts, manhole rims and inverts, slab elevations, height of buildings)? Is the project devoid of detail or is it covered with buildings and roads? If there are numerous structures, have they been previously located or will a location survey be part of the task? How will the data be collected in order to develop contours along the face of buildings, walls, curblines, critical section ditches and streams? What will the topographic data be used for? Can the data be collected in a random pattern, minimum grid interval, or stationed baseline/offset cross section? All of these considerations can have immense impact on the suitability of the finished topographic map. The technician who simply computes what is given without asking such questions beforehand will likely receive much grief at the hands of his or her supervisor.

The field data usually comes to the office stored within the data collector. The angles, distances and rod readings may have already been reduced to coordinate data by software within the data collector, or reduction may be the job of the office technician. All data will then be loaded, via interface cable, into the computer. This data will usually be composed of the data collector file, a readable field book file and any newly created coordinate data. The party chief will report any uncorrected entries to the computer operator. The reduction of the field data into coordinate data should be done when the party chief is present. Only he or she can report and resolve problems in the data collector file resulting from missing, corrupt or incorrectly entered data. On occasion, data does become corrupt and unusable. Illegal data entry, bad file exit procedure, or bad data editing

procedures are most often the causes. On occasion, worn cables, bent interface connectors and ports, and shorted entry buttons may be to blame.

Perhaps the single most important information coming out of the field is a graphical sketch and other handwritten field notes. These hand-drawn hardcopies will show physical improvement detail, corresponding point numbers used in data collection, benchmarks, vertical control loops, dimensions of physical features (buildings, pipes, walls, pavement widths), and lines and features to be honored by the computer for interpolating contours. In the absence of the party chief, the field sketch is the technician's only dependable instruction for putting together a puzzle, which may involve thousands of three-dimensional data points.

Once all field data is reduced, all points defining breaks in the lay of the land must be connected within the computer screen file. If an adequate coding system has been devised, the connection of these points will automatically occur in the computer processing of the raw data so that the person in the office will not need to do so. These lines will be referred to as *break* lines. All contours will be interpolated along these lines. A contour of a given value will naturally break in a certain direction as it runs between the interpolated points of equal elevation. Examples of break lines include top or bottom of bank, top or bottom of curb, building/wall faces, edge of pavement, centerline of stream, ditch or road or any continuous linear or curvilinear break in the elevation of the land. Every line segment will have two corresponding coordinates defining the location and elevation of the ends of the lines. Using interpolation, the computer may easily locate where any given contour elevation falls.

Having set up these known natural and man-made control lines, the technician will initiate a computer network of similar break lines, connecting all other miscellaneous points within the file that are intended for topographic compilation. The technician will protect the integrity of the input data by making use of elevation, numeric, or proximity filters to eliminate the use of extraneous data points within the coordinate file. No network line will cross another. Network lines will be constructed in such a manner that they form a continuous pattern of triangular sections whose vertices are composed of field points. The maximum allowable length of the network lines (selected by the technician) dictates the accuracy of the topography. The density of field observations, however, limits the length of the network lines. The computer cannot create a network in areas where data collection is insufficient to accommodate a given line length. No contours are obtainable in such an area. The technician must then decide if more field data is necessary or if a longer network line is acceptable.

When the entire network is complete and all errors have been dealt with, the technician has a good network file or Triangulated Irregular Network (TIN) file from which contours of any interval may be derived. After the technician selects an interval and scale suitable for the end user, the

computer creates by interpolating the location of all points within the network for each contour. Contours are then drawn (straight line) within the triangular network sections by graphical depiction on the screen. The technician may elect to have the computer automatically smooth out the faceted, unnatural or deceptive breaks in the contour lines. Further refinement may be obtained by hand tracing the desired contour alignment on the screen with a mouse or other instrument. The topographic survey map is then ready for plotting. The TIN file should be kept inviolate; it can be used again and again to generate additional topographical maps of varying contour intervals, as well as centerline profiles, grading scenarios, and earth takeoffs.

### Alidade-Planetable Techniques

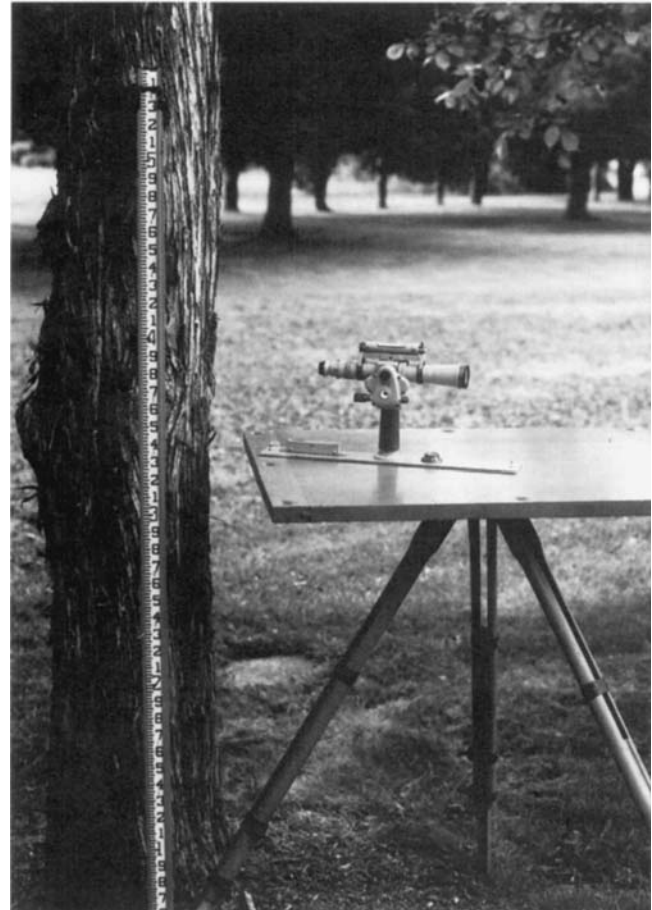
Alidade-planetable topography has been a means of making maps since the sixteenth century. Early surveyors realized the advantages of using a direct graphical means to create and check a map in the field. The planetable is a drafting board specially adapted for use atop a surveying tripod. Attached to the planetable is a scale map of the area onto which spot elevations are written based on rod readings through the alidade.

The alidade is an instrument that consists of a telescope mounted onto a straightedge. A vertical circle and leveling bubbles are also integral components of the alidade (see Figure 15.9). The straightedge—that is, blade—is oriented in the same direction as the line of sight of the alidade. Horizontal and vertical distances are measured using stadia by sighting a graduated rod.

The table is oriented and leveled over a known point. The straightedge blade of the alidade is pivoted around the point of the occupied station such that any location sighted in the field becomes a ray along the blade of the alidade between the pivot point and the sighted point. The point is readily plotted from the pivot by scaling the distance along the blade obtained from the rod reading. The point is represented by a dot, which becomes the decimal point for the elevation.

Drawing media for the map usually consists of a durable and stable material that is not susceptible to shrinkage or expansion due to atmospheric conditions. Mylar is the typical choice. To save time in the field, the control stations can be pre-plotted on the mylar hard-copy. Using grid lines or tick marks, which will indicate the northing and easting coordinates in use, the traverse stations are plotted using their individual coordinate values. The lines between stations are drawn and the number and elevation of each is written. Often, this information will be inked on the underside of the semi-transparent mylar in order to spare it from the rigors of field conditions.

The planetable has changed little in over 300 years of existence, but its modern modifications make it faster and easier to set-up, level and orient. The tripod may have adjustable or one-piece (“stiff”) legs but the top generally has a device known as a Johnson head. This ball and socket

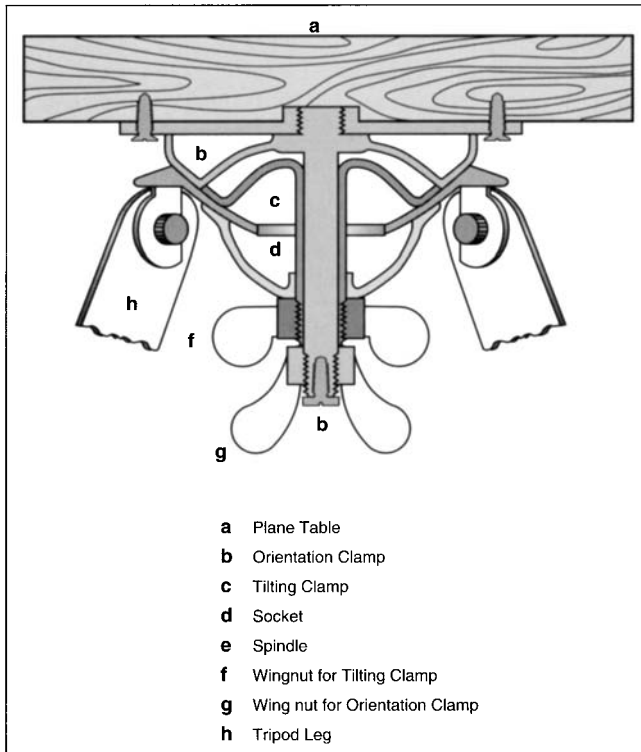


**FIGURE 15.9** Alidade, planetable and rod (photo)

device, patented by W.D. Johnson of the U.S. Geological Survey, permits the user to tilt and rotate the drawing board in any direction. As illustrated in the cross-section drawing (Figure 15.10) of the Johnson head assembly, wingnut “A” controls the tilt, and wingnut “B” controls the rotation of the board.

The hard copy—that is, the map—is affixed to the board, either with clips or masking tape. A straight pin or needle is stuck into the board at the traverse station to be occupied. For the set-up, the topographer orients that table roughly to correspond to the backsight and foresight stations and tentatively sets the tripod legs into the ground. A plumb line is suspended beneath the location of the straight pin, and the tripod legs and board are adjusted until the planetable is plumbed firmly over the station to within 0.1 ft of the corresponding point on the board. When this is accomplished, the alidade is placed on top of the table and its circular bubble used to level the board. The next task is to orient the board both horizontally and vertically.

Horizontal and vertical positioning will be controlled by means of three points: the one occupied, a foresight and a backsight. A sight consists of orienting the board, locking in horizontally, reading the stadia distance, and obtaining a vertical reading. A sight to a previous set-up point may



- a Plane Table
- b Orientation Clamp
- c Tilting Clamp
- d Socket
- e Spindle
- f Wingnut for Tilting Clamp
- g Wing nut for Orientation Clamp
- h Tripod Leg

**FIGURE 15.10** Johnson head assembly (From Davis, R. et al. 1981. *Surveying Theory Practice*. 6th ed. New York: McGraw-Hill. Reprinted with permission from McGraw-Hill)

suffice for either the backsight or foresight. If plane table traverse is required to obtain two required sights, closure should be assured before the resumption of topography. Vertical information is obtained by two methods: differential leveling and trigonometric leveling, both achieved by sighting a surveyors rod graduated in feet, tenths of feet, and hundredths of feet.

Distances are obtained using stadia. In the telescope of the alidade there are cross-hairs and stadia wires (see Figure 36.6). Cross-hairs are horizontal and vertical index lines in the form of a cross through the center of the eyepiece reticle. The stadia wires are proportionally placed at one-half an interval from the cross-hairs. (Some instruments also provide a one-quarter interval stadia wire.) In modern alidades the full interval's proportion is 1:100. Thus, one foot of rod (or one meter) intercepted between the full stadia wires means that the rod is 100 ft (or meters) distant from the instrument. An experienced observer will focus on the distant rod, use the drive screw to place the lower stadia wire on an even foot, and then read the upper stadia wire. When obstructions prevent the full stadia interval from being read, the half-stadia interval is read and then doubled to obtain the full distance (quarter-stadia interval multiplied by four).

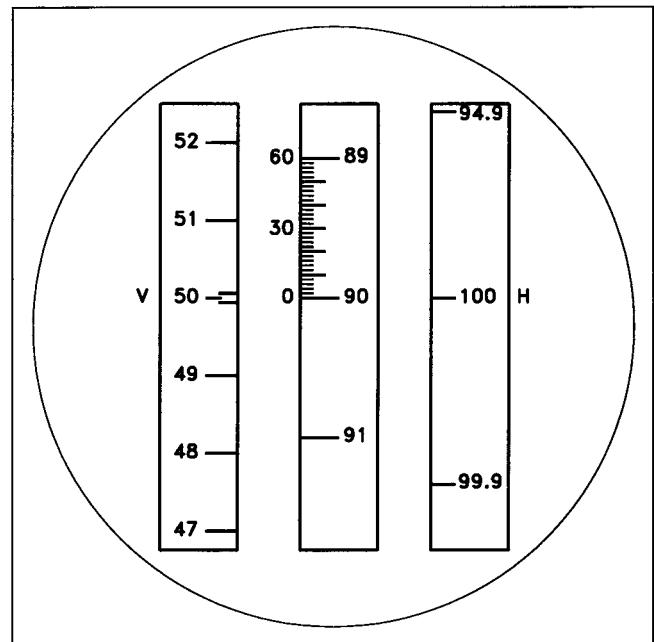
Differential leveling may be used where the cross-hair of the alidade, set on a level horizon, intercepts the rod. This

is known as a level reading, and it is reduced directly from the alidade's height of instrument. If the horizon line of the alidade is above or below the rod, trigonometric leveling is applied. To ease calculations, many alidades are equipped with a Beaman arc.

A Beaman arc is "a specially graduated arc attached to the vertical circle of an alidade . . . to simplify computing elevation difference for inclined stadia sights. The arc, designed by W.M. Beaman of the U.S. Geological Survey, is graduated such that each division on the arc is equal to  $100(\frac{1}{2} \sin 2A)$ , where  $A$  is the vertical angle. In use, the index is set exactly on a graduation and the difference is obtained by multiplying the stadia intercept by the number of Beaman arc divisions."<sup>1</sup>

Figure 15.11 shows the verniers within the telescope of the alidade. The V scale on the left shows the Beaman arc divisions, expressed in single steps. The center scale shows the vertical angle expressed in degrees and minutes. The H scale on the right shows the distance correction per 100 ft, expressed in feet and tenths. These scales are trigonometrically inter-related. In the example shown, the line of sight is level, so the Beaman arc step reads 50, the vertical angle is at 90, and the horizontal correction is at 100 ft.

Notekeeping (Figure 15.12) is a vital component of planetable topography. The notes are a record of the set-up locations, the backsight and foresight stations, the horizon-



**FIGURE 15.11** A look through the microscope at K&E vernier, self-reducing alidade

<sup>1</sup> American Congress on Surveying and Mapping, and American Society of Civil Engineers. *Definitions of Surveying and Associated Terms*. Revised 1978. Reprint. Bethesda, Maryland: American Congress on Surveying and Mapping, 1989.

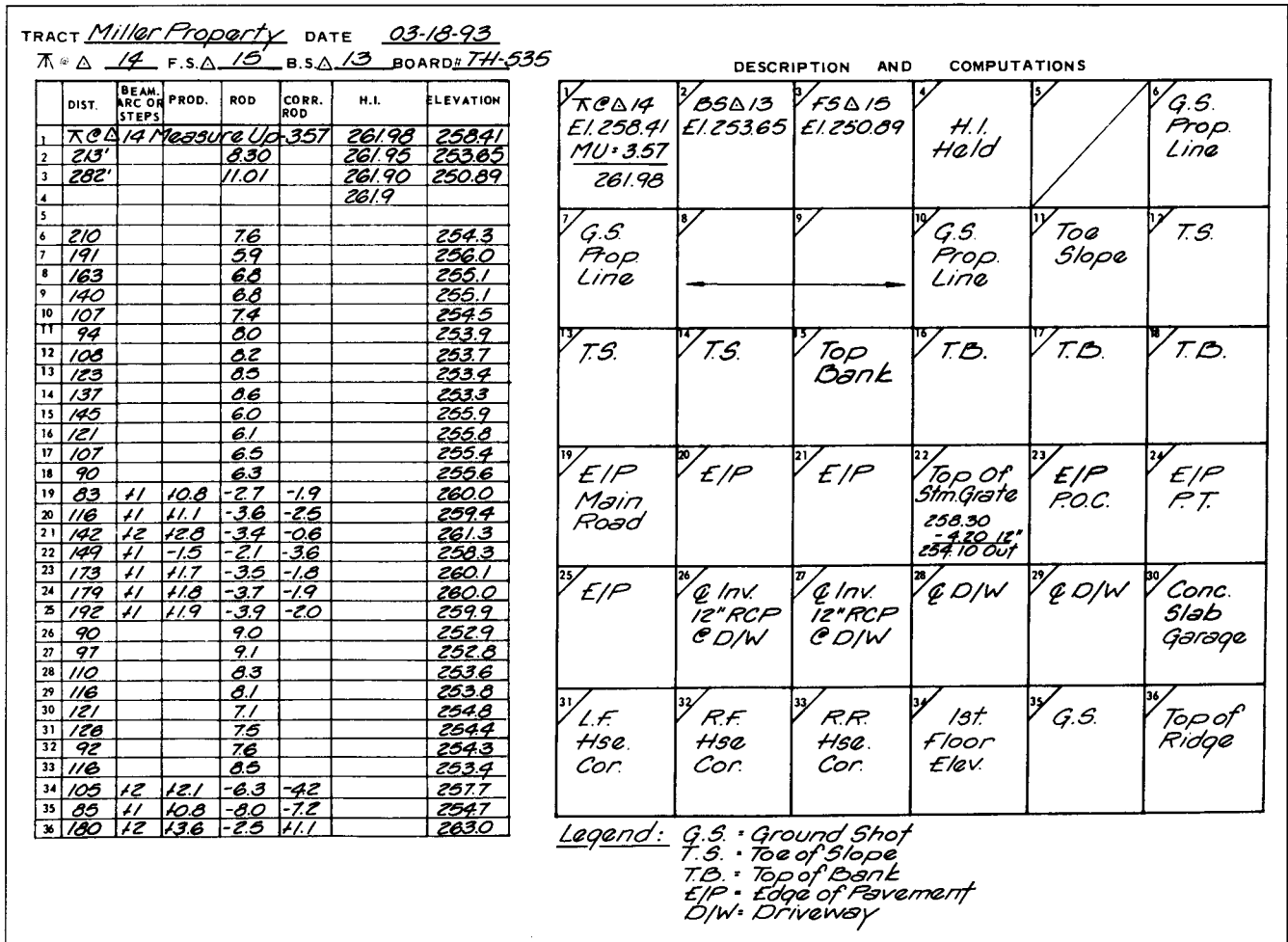


FIGURE 15.12 Alidade plane-table notes

tal and vertical readings, the mathematical computations and the shot descriptions. The notes are used immediately, literally as they are being created. Later they will be verified in the office, and corrections made to the hard copy. The information on the set-up, sights, distance, and description make it easy to find and rectify any bad shots.

**Field Checks with the Alidade-Planetable**

A distinct advantage of planetable topography is the many checks and verifications inherent to the entire map-making process. At the beginning of the set-up, three known horizontal and vertical points are consulted. Previously plotted locations are verified at set-up matchlines. The topographer is frequently able to catch mathematical errors made by the notekeeper. The office checker apprehends any that slip through. The rodperson also checks the drawing in progress, to be sure the topographer has interpreted well the sequences and dimensions involved. Omissions are avoided, as all plotting is performed in the field.

The following standards of accuracy should be applied to alidade-planetable work:

1. No contour error shall exceed  $\pm 1/4$  contour interval
2. No error in individual ground shot, picked at random, shall exceed  $\pm 0.3$  ft
3. Distance between ground shots shall not exceed 1" on scale
4. Contour interpolation shall be done with extreme care in every direction (radially)

The contouring of the map is best performed by the topographer on-site, but many survey companies prefer an office person to perform this task to save field-time. However, the topographer should at least contour the tricky areas while the notekeeper and rodperson are gainfully employed obtaining inverts, cutting line, or measuring structures.

Figures 15.13 and 15.14 are sample checklists that may save time and prevent any details from being overlooked in the field and office.

Subdivision Name \_\_\_\_\_ (Section, Lot No.) \_\_\_\_\_

Contract/Work Order No. \_\_\_\_\_ Date \_\_\_\_\_

\_\_\_\_\_  
(Signature)

A. Detail Necessary:

1. _____	Centerline and edge of pavement	18. _____	Concrete slips
2. _____	Drainage ditches	19. _____	Wells or springs
3. _____	Driveway entrances	20. _____	Swamps or standing water
4. _____	Driveway culverts	21. _____	Graves
5. _____	Intersecting road pavement	22. _____	Fences or walls
6. _____	Existing curb and gutter	23. _____	Streams
7. _____	Power poles and identifying numbers	24. _____	Wooded areas delineated
8. _____	Existing storm and sanitary sewers, tops and inverts	25. _____	Rock outcroppings
9. _____	Sewer pipe sizes and graphic direction between structures	26. _____	Obvious cut or fill grading on property
10. _____	Water valves and hydrants	27. _____	Profiles for a distance of 300' past boundary of tract, on centerline of swales, streams, ditches and roads
11. _____	Existing buildings	28. _____	"Monarch" trees located and described
12. _____	Ruins	29. _____	50' overlap of boundary on perimeter of property
13. _____	Rights-of-way in use	B. Notekeeping:	
14. _____	Rights-of-way abandoned	1. _____	Height of instrument checks
15. _____	Railroads	2. _____	Reduction of all rod shots
16. _____	Inshore piers	3. _____	Description of all shots
17. _____	Piles	4. _____	Notes indexed to manuscript number

FIGURE 15.13 Planetable topographic check list

Subdivision Name \_\_\_\_\_ (Section, Lot No.) \_\_\_\_\_

Contract/Work Order No. \_\_\_\_\_ Date \_\_\_\_\_

\_\_\_\_\_  
(Signature)

1. _____	Topography of property completed	22. _____	Grid ticks
2. _____	Topography extended sufficiently beyond property	23. _____	Area of property
3. _____	All contours drawn	24. _____	Bench mark datum -- USGS, County Sanitary Sewer, etc.
4. _____	Names of all streams	25. _____	Surveyor's signature (Use "Approved;" not "Certified Correct")
5. _____	Names of all streets	26. _____	Scale
6. _____	Pavement width and type	27. _____	Any utility lines
7. _____	Limits of any wooded areas	a. _____	Storm drains (inverts and tops)
8. _____	Show all culverts, manholes, catch basins, etc., and pipe sizes and inverts of all structures	b. _____	Sanitary sewers (inverts and tops)
9. _____	Show all sanitary sewer manholes and inverts and pipe sizes	c. _____	Waterlines
10. _____	Direction of flow on all storm and sanitary sewers	d. _____	Gas lines
11. _____	Delineate any graveyards or burial grounds	e. _____	Electric lines (overhead and underground)
12. _____	Show all poles and pole numbers	g. _____	Pole numbers
13. _____	Show all telephone and powerlines (overhead and underground)	h. _____	Railroad tracks
14. _____	Show all improvements	i. _____	Curb and gutter
15. _____	Show all buildings and type (e.g., 1-1/2 story brick)	j. _____	Any abutting streets
16. _____	Show metes and bounds of property	k. _____	Rock
17. _____	Indicate type of boundary corners	l. _____	Fence -- type
18. _____	Show all surrounding property owners, subdivision names and lot numbers, etc.	m. _____	Old logging road
19. _____	In title, show contour interval	n. _____	Top and bottom banks of streams
20. _____	File number	o. _____	Springs
21. _____	North arrow: indicate type	p. _____	Trees
		28. _____	Contact all utility companies to make certain of existence and correct location of all underground utilities

FIGURE 15.14 Checklist for drafting topographic surveys

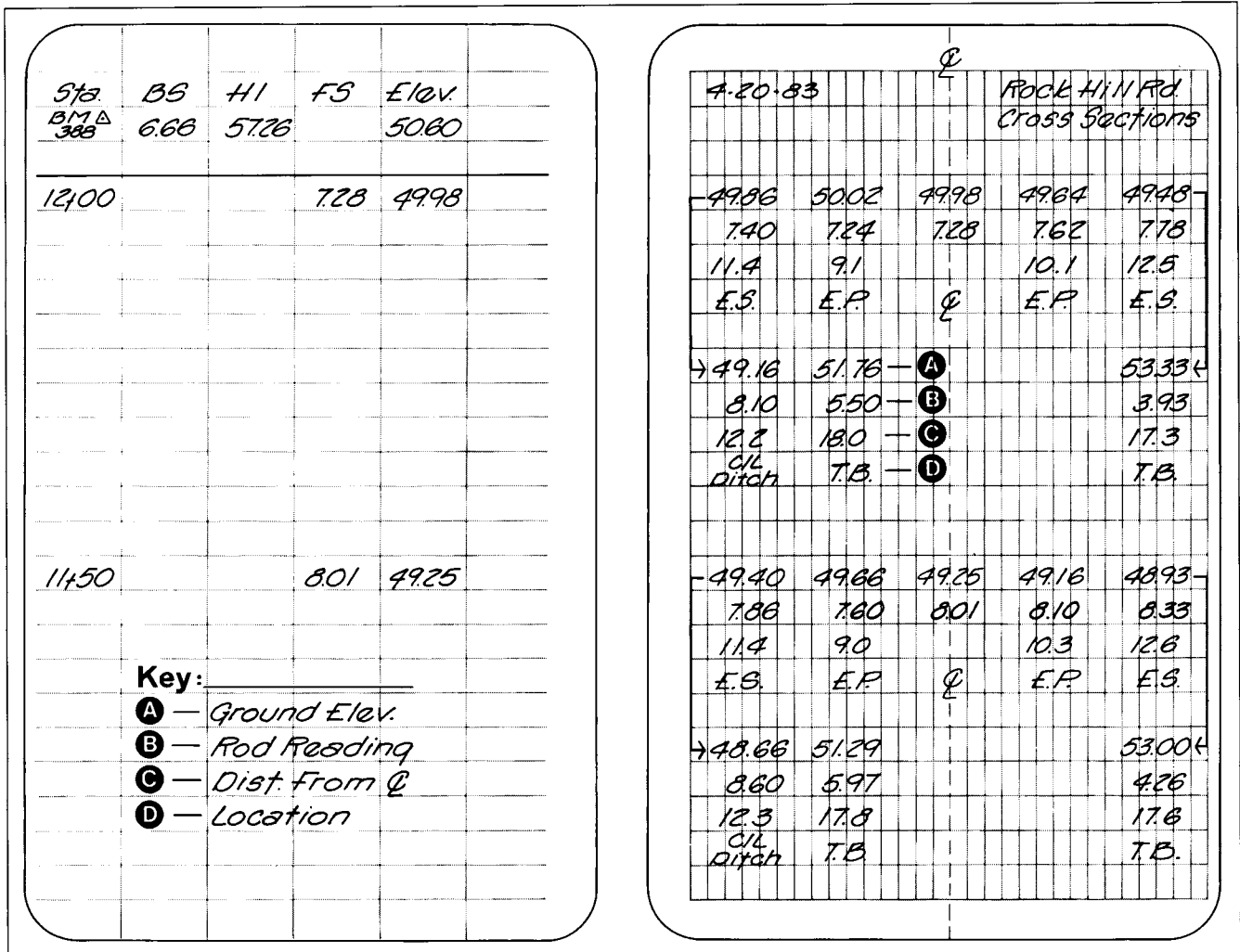


FIGURE 15.15 Baseline field notes

**BASELINE TOPOGRAPHY**

A baseline topographic survey relates horizontal and vertical data to a known reference line, that is, a baseline. This method is particularly suited for long-line surveys, such as roads, utility corridors, and streams. The baseline is stationed along its length. For each feature, the station and the distance left or right of the baseline is noted and an elevation obtained.

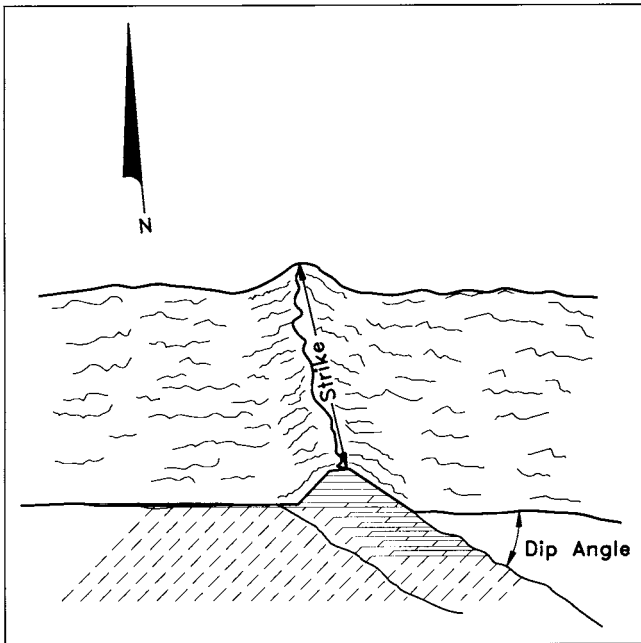
The baseline can be the centerline of a road, an offset line, a property line, or a traverse line, whichever is the most practical. The line is stationed in increments, such as 25 ft, with a hub or nail set at each one. Elevations are then established at each station.

Baseline field notes are unique in that they begin at the bottom and run up the page. As shown in Figure 15.15, the notes are set up with the profile level run on the left page and the cross-section notes on the right page. To accommodate the cross sections, plenty of space is left between the profile stations. On the cross-section page, a line is struck from top to bottom to represent the baseline. Each location is written to the right or to left of the line, de-

pending upon its location in the field. The data for each shot consists of a station along the baseline, a rod shot, a distance off the baseline, and an abbreviated description. The distances with the descriptions are obtained first, and the shots are marked with keel or flagging. When all the horizontal measurements are complete, the leveling run commences. With elevations established at each baseline station, they can also be used for benchmarks. On pavement, concrete, storm or sanitary structures, the rod is read to the nearest hundredth of a foot. On ground shots, the rod is read to the nearest tenth of a foot. After the levels are run, the rod shots are subtracted from the height of instrument (H.I.) and the elevations are written above the rod shot. In the office, the math is checked, and the profile and cross sections are plotted to the appropriate scale.

**DETERMINATION OF STRIKE AND DIP OF OUTCROPS**

Strike and dip represent the orientation of weakness planes in geological seams. Figure 15.16 shows the relationship of strike to dip. Dip is perpendicular to the strike. Strike is the compass bearing of a plane, while dip is a measure of



**FIGURE 15.16** Strike and dip

the inclination of the plane. The dip angle is the angle of inclination of the plane measured from the horizontal.

Finding the strike and dip depends on given information and the type of solution method chosen. The example included for illustration in this chapter uses trigonometric procedures. Other options for determining results include graphic methods that involve descriptive geometry and solutions that employ a nomograph.

**EXAMPLE:** The data from three core holes is listed in Table 15.2. Calculate the true strike and dip of the marker bed.

The computations involve solving the following three simple equations.

$$\tan \alpha_{x \text{ to } y} = \frac{\text{Elev.}_{\text{RCH}} - \text{Elev.}_{\text{MCH}}}{\text{Dist.}_{\text{RCH to MCH}}} \quad (15.31)$$

where:

- $\alpha$  = The apparent dip
- RCH = The reference corehole
- MCH = Marker bed corehole

$$\tan \gamma = \cot \alpha_1 \times \tan \alpha_2 \times \text{cosec } \mu - \cot \mu \quad (15.32)$$

where:

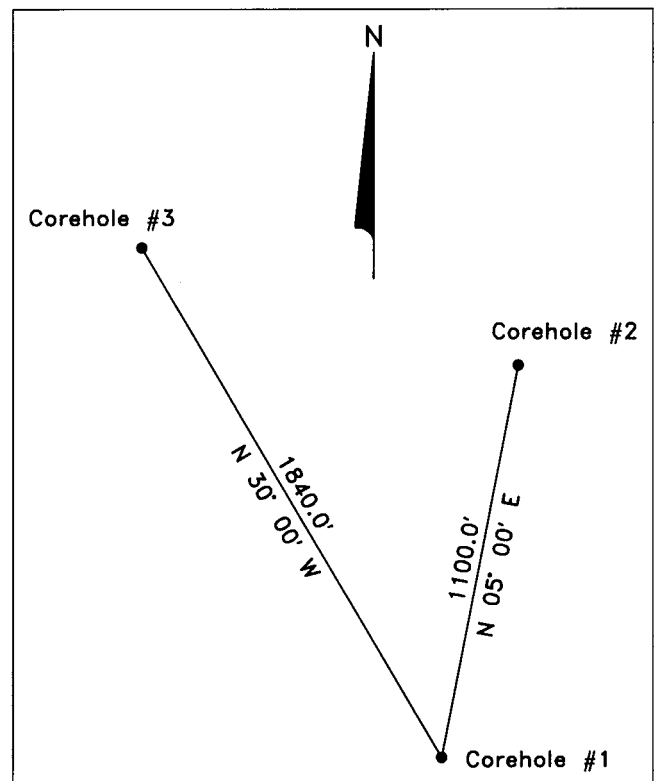
- $\gamma$  = The angle measured  $A_{1 \text{ to } 2}$  to the true dip direction
- A = Azimuth (or bearing) between coreholes
- $\alpha_1$  = Apparent dip between the reference corehole and a second corehole
- $\alpha_2$  = Apparent dip between the reference corehole and a third corehole
- $\mu$  = Angle between azimuths for apparent dips  $A_{1 \text{ to } 2}$  and  $A_{1 \text{ to } 3}$

$$\tan \delta = \tan \alpha_1 \times \sec \gamma \quad (15.33)$$

where:

- $\delta$  = The true dip
- $\alpha_1$  = The apparent dip between the reference corehole and a second corehole
- $\gamma$  = The angle measured  $A_{1 \text{ to } 2}$  to the true dip direction

1. Assume a reference point. For the example let that be corehole #1 (see plot of Figure 15.17).



**FIGURE 15.17** Sketch of corehole data

**TABLE 15.2** Data for Coreholes Determine Strike and Dip

COREHOLE No.	ELEVATION OF SEAM BED	DISTANCE
1	850 ft	1-2, 1100 ft
2	725 ft	1-3, 1840 ft
3	600 ft	

.....

Bearing from corehole #1 to #2: N 05° 00' E  
 Bearing from corehole #2 to #3: N 30° 00' W



2. Calculate the apparent dips using Eq. (15.31).

$$\tan \alpha_{x \text{ to } y} = \frac{\text{Elev.}_{\text{RCH}} - \text{Elev.}_{\text{MCH}}}{\text{Dist.}_{\text{RCH-MCH}}} \quad (15.34)$$

$$\tan \alpha_{1 \text{ to } 2} = \frac{125}{1100.0} = 0.11364$$

$$\alpha_{1 \text{ to } 2} = 6.483^\circ$$

$$\tan \alpha_{1 \text{ to } 3} = \frac{250}{1840.0} = 0.13587$$

$$\alpha_{1 \text{ to } 3} = 7.737^\circ$$

With two apparent dips determined and the horizontal angle between them also known, Eq. (15.32) supplies the direction of the true dip. This angle, determined in Eq. (15.32), is measured from the azimuth  $A_{1 \text{ to } 2}$  thus establishing the line of the true dip direction.

$$\begin{aligned} \tan \gamma &= \cot \alpha_1 \times \tan \alpha_2 \times \operatorname{cosec} \mu \times \cot \mu \\ &= \cot 6.483^\circ \tan 7.737^\circ \times \operatorname{cosec} 35^\circ - \cot 35^\circ \\ &= 8.80000 \times 0.13587 \times 1.74345 - 1.42815 \\ &= 0.65641 \end{aligned}$$

$$\therefore \tan^{-1} \gamma = 33.2812^\circ = 33^\circ 1' 652.4'' \quad (15.35)$$

Use Eq. (15.33) to find the true dip.

$$\begin{aligned} \tan \delta &= \tan \alpha_1 \times \sec \gamma \quad (15.36) \\ &= 0.113636 \times \frac{1}{\cos 33^\circ 1' 652.4''} \\ &= 0.113636 \times 1.196190 = 0.135931 \end{aligned}$$

$$\therefore \tan^{-1} \delta = 7.74$$

Figure 15.17 is a plot of the example and Figure 15.18 represents a graphic solution. Plot the two apparent dips from an origin with their lengths equal to the cotangent of their respective values. Next, join the termination point of the respective apparent dips. The connecting line represents the true strike. A perpendicular line from the true strike line through the origin represents the cotangent of the true dip. Values determined by scale yield close approximations of the mathematical results.

## SUMMARY

Topographic maps are an essential part of the planning process. The construction phases of land development projects rely on topographic maps for grades and location of important topographical features. If such maps are not of a high quality they can cause costly mistakes and time delays. Time spent planning the fieldwork and mapping for these surveys reduces the likelihood of errors and omissions in the final product.

The time for making determinations concerning type or combination of types of survey methods appropriate for col-

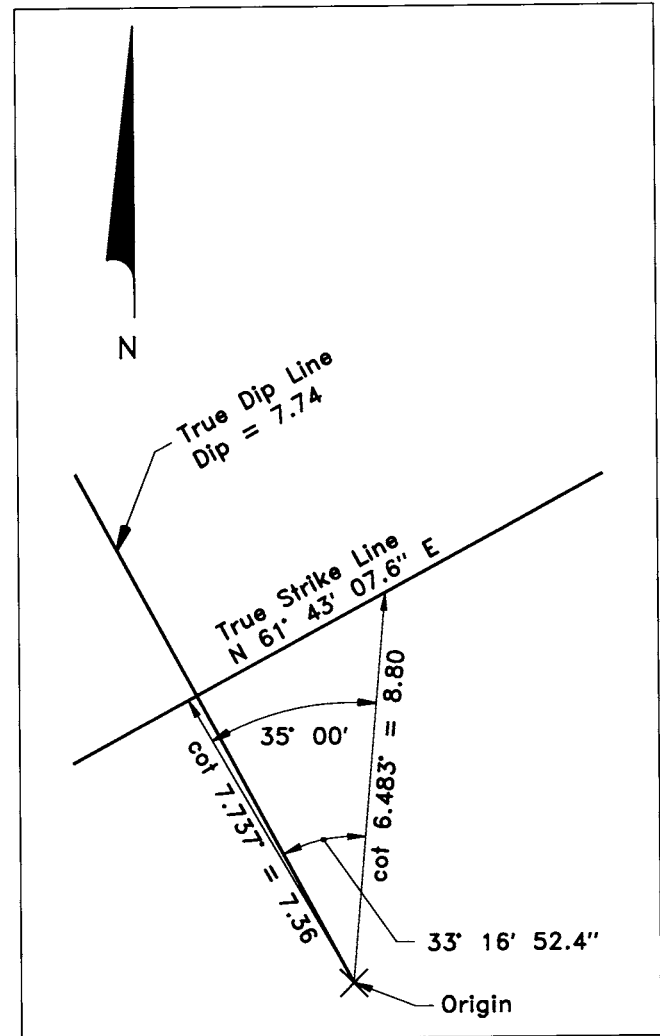


FIGURE 15.18 Graphic solution to determine strike and dip

lection of the necessary field data in the most effective and efficient manner is during the planning process. This is also the time for establishing a contour interval that will satisfy accuracy requirements.

The process moves forward from the planning stage to the fieldwork. Field crews establish a survey control network for both horizontal and vertical surveys that follow. These surveys require coordination with boundary surveys and in some instances they are combined. Existing features are located including buildings, roads, utilities, waterways, vegetation and other features that might impact on the development. Elevations required for developing contours are determined. This data is reduced to a form necessary for mapping.

Decisions on the size of drawings, scale ratios, labels and necessary notes precede the mapping process. Plot points in their proper location as defined by the horizontal control. Label elevations of respective points and begin the interpolation process appropriate to the data collected. Draw the contour lines on the determined interval. Impose information on other features related to the topography. Finally, on

completion of the drawing, make a check of the information shown prior to release of the map.

The method for gathering the data for a topographic map is simply a choice based on the circumstances associated with the property to be mapped. Whether one chooses an electronic survey system, photogrammetry, or one of the systems more commonly used in previous years, the concepts for determining what features will be mapped and what other considerations are to be given remain consistent.

#### REFERENCES

Davis, Raymond E., Francis S. Foote, James M. Anderson, and Edward M. Mikhail. *Surveying Theory and Practice*. 6th ed., McGraw-Hill Book Co., New York, 1986.

*Geodetic and Topographic Surveying*. Dept. of the Army Technical Manual. Government Printing Office, Washington, 1970.

Low, Julian W., *Plane Table Mapping*. Harper & Brothers, New York, 1952.

*Map Reading*, Dept. of the Army Field Manual. Government Printing Office, Washington, D.C., 1969.

Moffitt, Francis H., and Harry Bouchard, *Surveying*. 9th ed., Harper Collins Publishers Inc., New York, 1992.

Sloane, Roscoe C., and John M. Montz, *Elements of Topographic Drawing*, 2nd ed., New York: McGraw-Hill, 1943.

*The Surveying Handbook*, edited by Russell C. Brinker and Roy Minnick. Van Nostrand Reinhold, New York, 1987.

Wolf, P.R., *Elements of Photogrammetry*, 2nd ed., New York: McGraw-Hill, 1983.



# PART IV.B REFINEMENT OF PREVIOUS ASSUMPTIONS

## CHAPTER 16

# PRELIMINARY ENGINEERING

*David A. Jordan, P.E.*

### INTRODUCTION

Preliminary engineering is an integral component of the third step in the land development design process: schematic design. As discussed earlier in this handbook, schematic design is preceded by feasibility and site analysis (step 1) and conceptual design (step 2). The last stage of the design process is final design. The focus of this chapter is on preliminary engineering associated with the development of the schematic design.

Preliminary engineering is a refinement of the previous two steps in the design process. The purpose of this refinement is to create a deliverable, often in the form of a graphic, such as a site plan or rezoning development plan, but may include checklists and/or reports suitable for review by the developer/builder and local governing agencies. Generally, preliminary engineering is performed by civil engineers. However, the developer/builder, planner, architect, reviewers at the local governing agencies, as well as citizens may be contributors to this process.

This stage of the design process includes an engineering review of the schematic design/preliminary site layout. During the earlier design stages, the development program was determined, pertinent information gathered, the site analyzed with all opportunities and constraints presented, and a preliminary site layout prepared. Now it is time to fine-tune the design by refining the previous assumptions.

Although preliminary engineering reflects a level of design detail necessary for review and approval by local governing agencies, often a preliminary engineering deliverable is not always required by local governing agencies. However, the developer/builder himself typically requests this level of detail prior to incurring the cost associated with

developing the final site plan. The deliverable documents, produced during preliminary engineering, represent a final check of the development program before the more detailed final engineering proceeds. This provides, not only the developer/builder and/or local governing review agencies, but also citizens, with a greater sense of comfort as to the feasibility and construction costs associated with the proposed development program.

At this stage, the developer/builder may be the owner of the property or may be the contract purchaser with contingencies built into the contract, such as getting a rezoning or other application approved. Another contingency may be achieving a specific lot yield or obtaining the desired gross floor area of a building.

It is important to understand that the development program is always subject to change, particularly if it is presented to citizens and/or public review agencies. Hence, the preliminary engineering phase of the design process is oftentimes an evolutionary process that can last for months or even years as the public process progresses. As an example, it is not uncommon for changes to the site grading and layout to occur as a result of citizen comments requesting additional open space and tree preservation areas.

### COMPONENTS OF A PRELIMINARY ENGINEERING STUDY

Although the level of detail required when performing a preliminary engineering study may vary, depending on the developer/builder's needs or the submission requirements of the local governing jurisdiction, certain items are always required.

A comprehensive base map is an essential part of the preliminary engineering analysis. Ideally, this item includes field-run topographic survey information, the site boundary,

and the existing structures and features of the site. The base map should also include the site opportunities, such as buildable areas and natural site amenities, and site constraints, such as wetlands, floodplains, and environmental corridors. All of these items should be surveyed or at least tied down to the boundary survey. It is important that any demolition requirements be addressed as well with the base map.

The focus areas for a comprehensive preliminary engineering study are identical to those that will be performed during final engineering; the difference is in the level of detail. Whereas final engineering documents are more polished and can be used for construction, the preliminary engineering documents are refined enough to ensure that problems do not arise if preparing the final documents for construction. The components of a preliminary engineering study are as follows:

### **Suburban Street Design (and Site Layout)**

The horizontal configuration of roadways from the schematic design should be finalized and checked for geometric accuracy. Widths of proposed pavements should be shown based on expected traffic volumes. Any curbs, gutters, and sidewalks should also be depicted. Profiles of roadways should be prepared to verify vertical geometric accuracy, sight distance, and cut/fill requirements.

The preliminary layout should be tied down to the site boundary, and lots and/or land divisions should be computed and checked for geometric accuracy. Building setbacks should be established and building envelopes represented.

All on-site and off-site improvements should be shown, including easements necessary for utilities and ingress/egress (access). (See Figure 16.1.)

### **Storm Drainage Design**

The horizontal placement and sizing of the storm sewer pipe and structures should be done after the streets and general site layout have been designed. Typically, profiles of the storm sewer system are part of the final design. However, select profiling should occur during preliminary engineering to locate vertical conflicts with other utilities. Additionally, flow capacities of proposed and existing systems should be checked. Since the adequacy of the drainage outfalls is critical, as-built information of existing storm systems and field run cross-sections of existing drainage channels should be obtained.

### **Design of Stormwater Management Facilities**

Sizing the stormwater detention facilities is typically part of the preliminary engineering stage of the design process. In order to size the facilities, it is important to determine the specific water quality or pollution-removal requirements as well as the allowable stormwater release rates for the proposed development. The water quality requirements vary greatly from region to region within the country and even

vary significantly at the regional level. Although the configurations and grading of such facilities are subject to change during the final engineering, it is important to determine if enough room has been set aside to achieve the required quality and release rate.

Additionally, the spillway capacity of the stormwater management/water quality facilities should be determined. Oftentimes the spillway capacity, which is related to the hazard classification of the facility, has to be increased if the facility is found to be a hazard to downstream dwellings.

Subsequently, a dam break analysis should be performed and hazard classification should be determined. If off-site easements are required downstream for a breach, these should be obtained as soon as possible.

### **Floodplain Studies**

Existing floodplains that were approximated during the site analysis stage of design should be modeled using field-run topography. Additionally, these floodplains should be brought into the base map so that their expanse can be more accurately identified.

### **Grading and Earthwork**

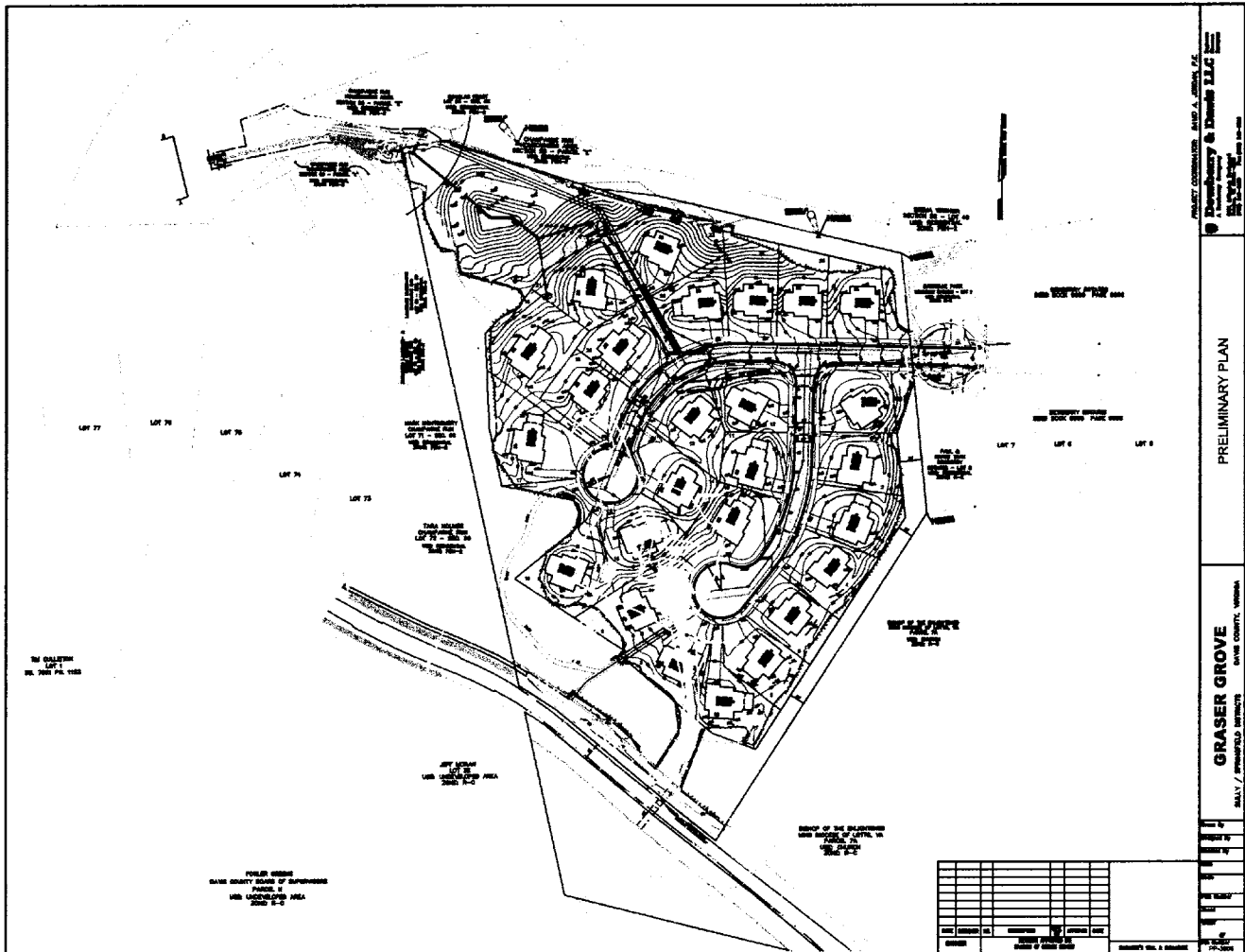
Grading is one of the most important steps of the preliminary engineering process. It determines the extent of the clearing limits of the project and determines if any requirements for off-site construction are needed. Additionally, earthwork analyses can be performed using the preliminary grading plan to check whether the proposed project balances or will require the import of fill or export of cut.

The civil engineer should review soils mapping and/or available geotechnical reports during preliminary engineering. The investigation should include problem soils and rock formations.

### **Wastewater Collection/Wastewater Treatment**

The horizontal placement of the sanitary sewer should be determined. Additionally, as-built information of existing sanitary sewer pipes should be obtained to verify capacities and vertical elevations for future tie-ins. Profiles of the sanitary sewer are normally part of final design. However, select profiling should occur during preliminary engineering to locate vertical conflicts with other utilities and determine whether there is adequate cover over pipes, such as at stream crossings and in roadways.

Downstream treatment plants should be contacted to confirm that they have capacity for the proposed flows from the development. If public sanitary sewer is not available to the project, alternative methods for wastewater treatment will need to be explored. As an example, if septic systems are being used for sewage treatment, they should be preliminarily sized and configured. In order to do this, percolation tests in the area of these proposed fields will have to be performed to determine the rate at which the soil can absorb the effluent.



**FIGURE 16.1** Example of a preliminary site plan.

### Water Distribution/Water Treatment

The horizontal configuration of the water supply system should be determined. Water pressures in the immediate vicinity of the proposed development should be checked, as well as the fire flow requirements.

If the public water supply system is not accessible, then borings for well water must be performed or other methods of water supply to the project will need to be examined.

### Erosion and Sediment Control

A general erosion and sediment control program should be determined at this stage of the design program to ensure compliance with local code requirements. Some jurisdictions will review and approve rough grading/erosion and sediment control plans so the developer/builder can get on-site and start construction while preliminary and/or final engineering efforts are still under way.

### Cost Estimating

Preliminary cost estimates of the proposed improvements, including the fees associated with the development pro-

gram, can be determined at this stage of the project now that the preliminary sizing of the above-mentioned items is complete. Such costs can be approximated using nationally published reference manuals or by using the local governing agency pricing guidelines. Oftentimes the developer/builder has a database of cost guidelines it wishes to utilize or the designer uses previously designed and built projects for unit price costs.

### Miscellaneous Items

Checklists and reports are often prepared during the site analysis and site selection stage of the design process. It is good practice to update and/or prepare new checklists and reports as part of the preliminary engineering process. An example of a preliminary engineering checklist is provided in Figure 16.2.

During the preliminary engineering stage of the design process, it may become apparent that waivers and/or modifications to local design standards and zoning ordinances may be required. Obtaining waivers or modifications may be critical in meeting the objectives of the proposed devel-

### Checklist For Preliminary Engineering Study

Date: \_\_\_\_\_  
Prepared by: \_\_\_\_\_

**A. Basic Project Information**

1.0 Project Name: \_\_\_\_\_

2.0 Project Location: \_\_\_\_\_

3.0 Client: \_\_\_\_\_

4.0 Project Manager: \_\_\_\_\_

5.0 Lead Engineer: \_\_\_\_\_

6.0 Planner / Other Team Members: \_\_\_\_\_

	Yes	No	N/A	Comments
7.0 Has Project Manager visited site? If so, give date(s): _____	_____	_____	_____	
8.0 Has Lead Engineer visited site? If so, give date(s): _____	_____	_____	_____	
9.0 Has Planner and/or Other Team Member(s) visited site? If so, give date(s): _____	_____	_____	_____	

**B. Street Design / Site Layout**

1.0 Has the horizontal configuration of the roadways been checked for geometric accuracy? \_\_\_\_\_

2.0 Are the pavements widths correct? \_\_\_\_\_

3.0 Has sight distance been verified both vertically and horizontally? \_\_\_\_\_

4.0 Are sidewalks and/or trails shown in accordance with the requirements? \_\_\_\_\_

5.0 Have the lots and/or land divisions been computed and checked for geometric accuracy? \_\_\_\_\_

6.0 Are all setbacks and building restriction lines shown? \_\_\_\_\_

7.0 Are all existing on-site and off-site easements shown? \_\_\_\_\_

8.0 Are proposed easements shown (approximate widths and locations)? \_\_\_\_\_

9.0 Are off-site easements required for the project? \_\_\_\_\_

**C. Storm Drainage**

1.0 Have the horizontal placement and sizing of storm sewer and storm structures been performed? \_\_\_\_\_

2.0 Has select profiling been performed to determine possible vertical conflicts between utilities? \_\_\_\_\_

3.0 Have all existing storm drainage outfalls been checked for adequacy? \_\_\_\_\_

**FIGURE 16.2** Example of a preliminary checklist.

<b>D. Stormwater Management</b>			
1.0	Has storm water detention facility(s) been sized to achieve the required quality and release rate?	_____	_____
2.0	Has the spillway capacity been determined?	_____	_____
3.0	Has a preliminary dam break analysis been performed?	_____	_____
<b>E. Floodplain</b>			
1.0	Is there a floodplain present?	_____	_____
2.0	If so, has it been modeled using field-run topography?	_____	_____
<b>F. Grading / Earthwork</b>			
1.0	Has a grading study been performed?	_____	_____
2.0	Has an earthwork analysis been performed?	_____	_____
3.0	Have the soils been evaluated?	_____	_____
<b>G. Wastewater Collection / Wastewater Treatment</b>			
1.0	Is public sanitary sewer available to the proposed project?	_____	_____
2.0	If so, is there capacity in downstream sewer and treatment facilities for the proposed improvements?	_____	_____
3.0	Have the horizontal placement and sizing of sewer been determined?	_____	_____
4.0	Has select profiling been performed to determine possible vertical conflicts between utilities?	_____	_____
5.0	If public sanitary sewer is not available, what method will be used for wastewater treatment?	_____	_____
<b>H. Water Distribution / Water Treatment</b>			
1.0	Is public water service available to the proposed project?	_____	_____
2.0	If so, is there adequate pressure in the existing system and are the fire flow requirements met?	_____	_____
3.0	If public water is not available, what method will be used for water supply?	_____	_____
<b>I. Erosion and Sediment Control</b>			
1.0	Has a general erosion and sediment control program been determined for this project?	_____	_____
<b>J. Cost Estimates</b>			
1.0	Have preliminary costs been determined for the proposed improvements? If so, list them.	_____	_____
2.0	Have fees associated with the development program been determined? If so, list them.	_____	_____

**FIGURE 16.2** (Continued) Example of a preliminary checklist.



K. Miscellaneous			
1.0	Are any waivers and/or modifications to local design standards and zoning ordinances required?	_____	_____
2.0	Are there any wetlands and/or other environmental constraints associated with this site?	_____	_____
3.0	Is gas service available to this project?	_____	_____
4.0	Is electric service available to this project?	_____	_____
5.0	Is telephone service available to this project?	_____	_____
6.0	Is cable television service available to this project?	_____	_____

**FIGURE 16.2** (Continued) Example of a preliminary checklist.

opment program, to facilitate the eventual construction efforts, or to speed up the design review process. Unique or nonstandard construction details should be preliminarily prepared and analyzed during this stage of the design process.

Gas, electric, cable, and telephone line availability should be checked if not already investigated during the feasibility stage of the development program.

### VALUE ENGINEERING PRIOR TO FINAL DESIGN

An important part of preliminary engineering is determining the most economical approach to design and construction without significantly altering the design program. The process of economizing the design program is termed *value engineering* and should be accomplished before final engineering.

The main purpose of this effort is to identify cost-saving opportunities to the developer/builder. It may or may not include all members of the project team, such as the architect, planner, public agency reviewer, attorney, construction manager, and developer/builder.

It may become apparent, for instance, after the preliminary grading study and earthwork calculations are performed, that the site elevation should be raised or lowered to balance the earth materials. Reconfiguring lots may reduce street lengths and utility requirements. Perhaps regrading or adding retaining walls might preserve more open space and trees.

Preliminary construction cost estimates and review fee estimates may or may not affect the design program or the design and construction schedules.

Value engineering should be accomplished prior to final design because changes that occur after final engineering has commenced are costly and create schedule problems. Delays to plan approval and to obtaining permits for construction are almost certain if major design changes are required during final engineering to save costs.

### ADDITIONAL READING

- Brown, Thomas L., *Site Engineering for Developers and Builders*, National Association of Home Builders, Washington, D.C., 1988.
- Colley, B.C., *Practical Manual of Land Development*, McGraw-Hill, New York, 1993.
- De Chiara, Joseph, and Lee E. Koppelman, *Site Planning Standards*, McGraw-Hill, New York, 1978.
- Galaty, Fillmore W., Wellington J. Allaway, and Robert C. Kyle, *Modern Real Estate Practice*, Real Estate Education Company, Chicago, 1988.
- National Association of Home Builders (NAHB), *Land Development*, NAHB, Washington, D.C., 1987.
- National Association of Home Builders (NAHB), *Land Development Manual*, NAHB, Washington, D.C., 1974.
- Northern Virginia Builders Association, *The Basics of Land Acquisition*, Northern Fairfax, Va, 1985.
- Northern Virginia Building Industry Association and Virginia Bankers Association, *Guidelines for Environmental Site Assessment*, Fairfax, Va., 1990.
- Urban Land Institute (ULI), *Residential Development Handbook*, ULI, Washington, D.C., 1978.

## CHAPTER 17

# ENVIRONMENTAL AND NATURAL RESOURCES

*Kim Larkin*

### OPEN SPACE AND VEGETATION PRESERVATION

“Tree protection in land development should be a shared community responsibility. Developers need to approach their projects with some environmental sensitivity. Citizens need to understand the needs of developers. And local governments must be flexible, yet recognize and support the need for tree protection and replacement. If all three sectors cooperate, our cities can grow while remaining beautiful and environmentally healthy.” (Macie and Moll, 1989).

#### Introduction

An increase in the awareness of the benefits that open space and vegetative buffers provide, both individually and in aggregates, has caused the issue of vegetation preservation and augmentation to become an integral part of the land development process. As our built environment expands, it is imperative that land development activities not compromise the long-term quality of our environment. To this end, the responsibility of land stewardship is expanding beyond the traditional disciplines of agronomy, environmental and soil science, forestry and landscape architecture to include various engineering and legal professionals, corporations, community and urban planners, public officials, and private citizens. In short, any individual who participates in the land development process must understand and appreciate the important functions and values provided by vegetative buffers in climate, air, and water quality preservation. Forested area preservation, as well as other open space preservation is a vital part of our economic future.

Open space preservation in many cases is not limited to the preservation of individual trees alone; rather, the term “open space preservation” generally implies preservation of

existing trees and other associated vegetation, including the understory trees, shrubs, vines, and herbaceous plants.

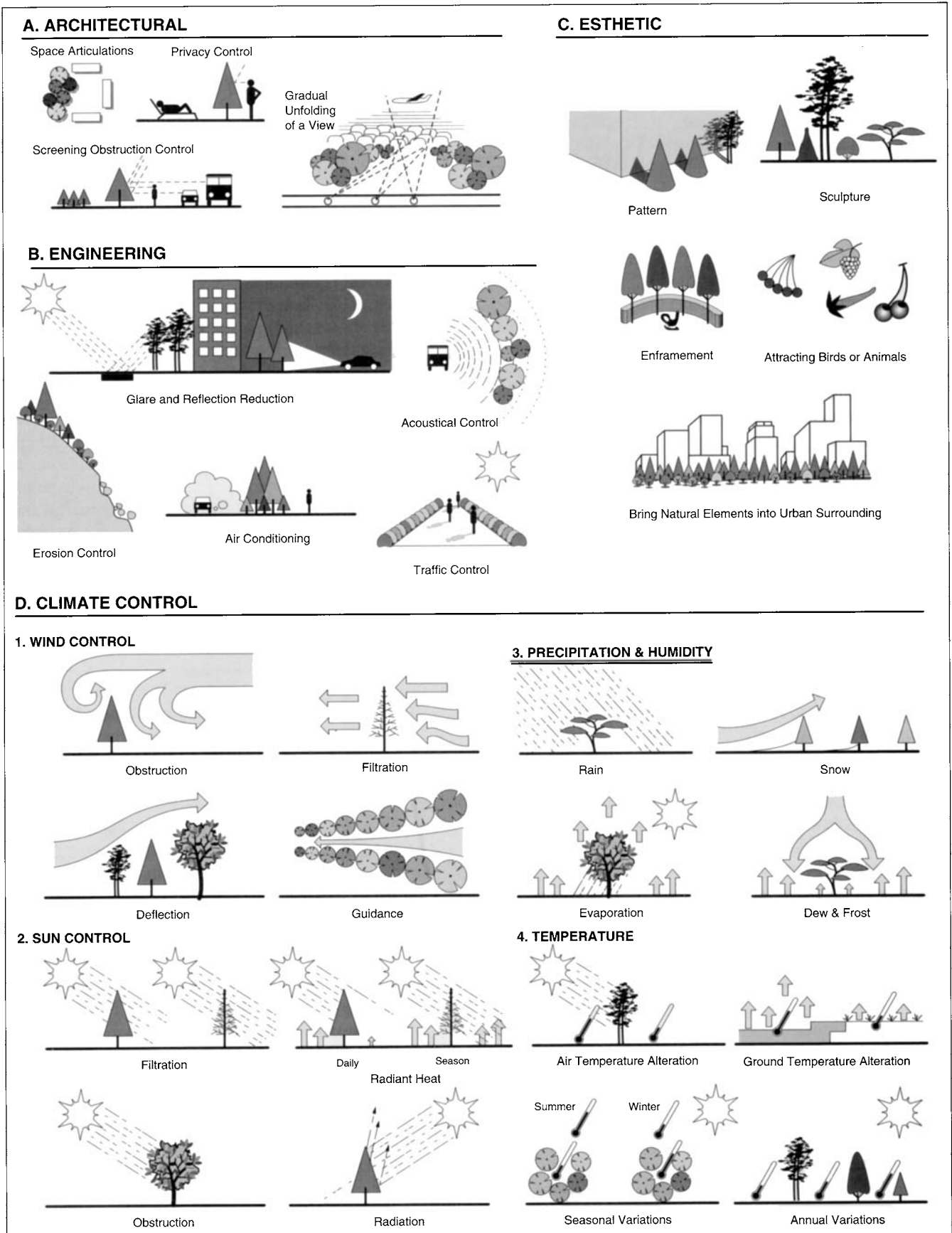
#### The Values and Benefits of Trees and Open Space

Understanding the significance of tree preservation requires a basic understanding of the values and benefits provided by trees. Figure 17.1 illustrates the functional uses of trees and their associated vegetation, from which the following values and benefits are derived. The values and benefits related to architectural design elements include:

- Aesthetics
- Space definition and articulation
- Screening undesirable views
- Complementing or softening architecture
- Creating a sense of unity among inharmonious buildings
- Providing textural and pattern variety
- Buffering incompatible land uses
- Attracting wildlife

Trees affect the microclimate of an area by moderating the effects of sun, wind, temperature, and precipitation. Such climatological values and benefits include:

- Intercepting, filtering, or blocking unwanted solar radiation
- Blocking undesirable wind by obstruction
- Directing wind flow by deflection



**FIGURE 17.1** Functional uses of plant material which have measurable value and benefits. (From International Society of Arboriculture: *Guide to Plant Appraisal*, 8th ed., 1992)

- Reducing wind velocities by filtration
- Moderating temperature changes (although this is more directly a function of solar radiation interception)

The value and benefits of trees and open space relative to site development and engineering are equally as important. These include:

- Decreasing stormwater runoff directly through interception of rainfall and water uptake through the root system, as well as filtering pollutants contained in runoff from the adjacent watershed
- Stabilization of soils
- Reducing the glare and reflection characteristically generated by the combination of buildings and/or roadways and natural and/or artificial light
- Acting as noise attenuators
- Interacting with the particulate matter and gasses known to cause air pollution to significantly reduce the concentrations of these pollutants
- Adding extra oxygen to the atmosphere
- Recreational opportunities (gardening, ballfields, hiking trails)
- Education
- Non-polluting transportation (bike trails)
- Tourism
- Flood protection

All of the architectural, aesthetic, climatological, and engineering uses give value to trees that can be described in both economic and legal terms (Miller, 1988). Because of these measurable assets, trees and open space contribute to and enhance property values. The resulting sum of values equates to an improvement in the quality of life for everyone.

### Preservation/Protection Ordinances

Open spaces are considered valuable environmental resources, and the incremental loss of these resources has prompted many jurisdictions to enact legislation to preserve and protect forested areas and farmlands, as well as individual trees. In addition, increasingly poor water quality and urbanization in areas that were once very rural have led communities to protect more than just the natural resources, but entire watersheds and viewsheds as well. This legislation is generally enforced through local zoning or landscape ordinances. Generally, these ordinances are not intended to place unreasonable constraints on land development. Many of these ordinances have been created to act as guides for land development so that the preservation of a valuable resource or viewshed is balanced with site de-

velopment. The idea underlying preservation ordinances is that if site development is balanced with appropriate preservation, then the end result is largely beneficial to all those participating. Land development and preservation need not be mutually exclusive. In fact, open space preservation has been demonstrated to prevent flood damage, attract investment, revitalize cities, boost tourism, along with preserving the environment (Lerner & Poole, 1999).

Since communities differ in their natural environment, political structures, cultural traditions and legal framework, preservation ordinances can and do vary between jurisdictions. However, all have the similar goal of protecting significant natural resources or historic views. All of the regulations or ordinances are, and need to be, specific to local conditions, circumstances, needs, and desires (Robinette, 1992).

Regulations within preservation ordinances may also vary widely between jurisdictions. Some localities are quite specific as to the percentage of existing vegetation that must be retained and have designed methodologies for determining the extent of existing vegetation to be targeted for retention. Other localities simply offer general guidelines requiring that all attempts possible be made to retain existing vegetation. Most ordinances will establish the definitions, procedures, penalties, and appeals process specific to the individual locality for preservation and protection. In some instances, preservation ordinances are developed to protect entire ecosystems, such as those areas that are known habitats of rare, threatened or endangered species, or watersheds that drain into a public water supply.

Since variation between ordinances can be great, and all ordinances are specifically reflective of the local policies, circumstances and needs of a particular area, it is more critical here to understand the general basic premise for an ordinance, rather than the details specific to many localities. Be aware that specific preservation ordinances exist in a majority of the towns, cities, and urbanized areas nationwide and that they may either be separate or included as part of larger ordinances. It is the responsibility of the site designer (or design team) to be cognizant of the requirements particular to the jurisdiction in which a specific project is located.

### Open Space and Tree Preservation and the Site Development Process

It is critical to understand that in order for open space and tree preservation to be successful, it must be integrated into the early planning stages of land development. It is nearly an impossible task to address these types of preservation later in the site design stages and achieve the optimum balance between the built and natural environments. Retrofitted open space and tree preservation has a greater chance of failure and can actually create unwanted liabilities. Properly planned and implemented open space and tree preservation can enhance the aesthetic, natural and economic environments for all.

Preservation is best accomplished during the early planning or feasibility phase of site development, as part of the site inventory and analysis. To ensure the success of any preservation program, the tasks associated with the formulation of the preservation plans should be completed by a professional, knowledgeable in the topics of biology, physiology, taxonomy, and ecology, as well as the relationship between those ecosystem functions and land development. Many methodologies exist for determining the quality of trees or other natural resources, but all have a common goal of identifying the high quality habitats existing on a particular site. The following discussion addresses the basics applicable to most methods of vegetation analysis and preservation. Again, familiarity with regulations pertinent to the locality within which the site is located is the responsibility of the site designer or design team.

Prior to any site visit, a review and analysis of the existing site conditions will provide an overview of the limits of the existing vegetation, the site topography, soils locations and descriptions, riparian areas such as streams, ponds and/or wetlands, open fields, existing buildings, and other existing and/or natural features. All of these living and nonliving factors influence the species and quality of vegetation indigenous to the site. A review of recent aerial photography of the site affords an overview of the site and a characterization of existing vegetative patterns.

Existing site information or data collected during a site visit can range from general to detailed, depending upon the level of information desired or required regarding the vegetation. At a minimum, the data collected during a site visit should include the following:

- Identify individual, significant trees, groups of similar trees
- The species present
- The quality of forest structure
- Relative age and condition
- Note any riparian areas

Other information can include:

- The locations of scenic vistas, important wildlife habitats, or obvious past or present management activity (e.g., logging, prescribed burning, etc.)
- Transplantable tree stock
- Obvious insects or diseases present
- Any other natural or cultural features that may contribute to the significance of the vegetative cover and ambiance of the project

The appropriate analysis of this collected data identifies and prioritizes the quality of the existing vegetation and its placement in the landscape. The prioritization of the existing vegetation in turn influences the placement of roads,

buildings, parking lots, utilities, and other development program features integrated into the conceptual design of the site. The objective is to place site improvements where there will be a minimal impact on the desirable vegetation targeted for preservation.

As the site design progresses from the conceptual design phase to the schematic design phase, the preservation or conservation plan should be developed concurrently to detail clearing limits, preservation areas or protection zones, and protection techniques to implement the desired preservation successfully.

It should be noted during project planning that the COE regulatory division, as well as many state governments, has placed a recent emphasis on the preservation of vegetative buffers located adjacent to riparian areas (streams, rivers, lakes, etc.), particularly if the buffer area is forested. Riparian forest buffers have been found to have significant benefits to water quality (Klapproth & Johnson, 2000). Planning for the preservation of these riparian buffers during the conceptual design phase may assist with future permitting.

Ideally, all trees along clearing limits and proximate to construction should be located by survey methods, plotted on a map, and identified by species and diameter size. These individual trees along the clearing limits should be evaluated for their respective health and structural condition, expected longevity, critical root zone areas, potential tolerance to construction activity, the suitability of the species for the new use, and the level of ongoing maintenance (Harris, 1992). This evaluation process will assist in identifying which trees are the best or most appropriate candidates for preservation. Typically, groups of trees are more successfully preserved than individual trees, and younger trees are more tolerant of changes in their environment than older trees.

### Forest Preservation, Location, and Function

For the purpose of this discussion, it is important to understand the biological and structural differences between trees growing under different conditions. Trees located in the forest interior have few lower limbs, the root systems must compete with many neighbors for water and nutrients, their crowns spread considerably in competition for sunlight, and their growth is slowly upward toward the light. When site grading removes some of the surrounding trees, previously interior trees are exposed to desiccation from wind and sun, their hydrologic regime is altered (usually becoming drier), and they become subjected to increased insect and disease infestations. These trees will tend to have a spindly appearance and will overtime sprout numerous short branches. Windthrow can also be a critical problem for these trees because of their top-heavy structure and relatively shallow root systems.

Trees that grow naturally at the edge of a forest function to protect the forest interior. Therefore, their growth pattern tends to be outward, away from competing neighbors, both above and below ground. Most of the branches on edge

trees grow outward toward the abundant light with few branches occurring on the shaded side, resulting in a lopsided structure. When site clearing removes trees from the shaded side of edge trees, the trees are subject to increased stresses from the new exposure, increased occurrence of insects and disease, and windthrow. When construction activity alters the existing grade on the open side, these trees suffer considerable root damage, since their roots extend far beyond the canopy dripline, and are also subject to windthrow.

Isolated trees have adapted to life in an exposed situation. They do not compete directly with other trees for sunlight, water, and nutrients. Their growth patterns are outward and upward in all directions. Generally, isolated trees have well-rounded crowns and branch in all directions along most of the length of the trunk. Roots of isolated trees generally spread further from the trunk than do roots of trees growing in competition with each other.

### Protection of Trees During Construction

In tree preservation, the root zone area is the critical and limiting factor of success. Plants depend upon roots for water and mineral uptake, storage of food reserves, and the synthesis of needed organic compounds and anchorage. Generally, tree roots are located in the upper 36 in. of the soil, with the majority of the roots in the upper 18 in. However, two factors that can significantly alter that generalization are the soil texture and existing forest litter. The finer the soil texture, the shallower the absorbing tree roots; similarly, the thicker the leaf litter, the shallower the absorbing roots. In many cases, the feeder roots of trees growing in finer texture soils and/or mature forest situations will occur in the upper six to twelve inches of soil (Harris, 1992).

Any land development activity that affects the roots of trees will directly affect the above-ground portions, either proportionately or disproportionately. Recent research has found that tree roots can extend far beyond a tree's dripline, typically branching out from the trunk a distance of 0.5 to 1.5 times the tree's height (McDaniel, 1997). Healthy roots grow where the soil aeration, moisture, soil temperature, nutrients, and soil structure are favorable.

Unless the tree canopy is physically or chemically damaged in some way, or the trunk or branches are physically or mechanically damaged, most construction-related injury involves root damage. One common form of construction-related tree root damage (Figure 17.2) occurs because of extensive disturbance of the soil peripheral around preservation areas while removing the trees adjacent to the preservation areas (e.g., pulling stumps instead of grinding them). Other damage includes:

- Root severance in the topsoil stripping process or in lowering grades
- Excavating for footings or trenching for utilities

- Raising the grade, which can suffocate roots
- Compacting the soil, which crushes and suffocates roots
- Changing the hydrologic regime (wetter or drier) by altering existing drainage patterns
- Storing, dumping or disposing of construction materials that are toxic to plants in or near preservation areas, like concrete washout areas

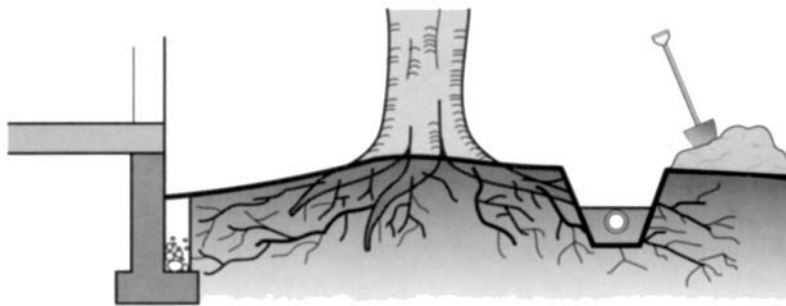
For these reasons, tree protection fencing or other protection devices should generally be located outside the dripline of trees that are to be preserved and should be implemented prior to or at the beginning of any clearing and grading activities.

The tree preservation zones, delineated during the schematic design phase for the site, should be further refined during the conceptual design (preliminary site layout) phase. Often, compromises between the site development program and tree preservation zones occur as the proposed site grading is refined. Trees deemed to be severely impacted by the proposed construction should be identified, primarily according to the impacts on the critical root zones, with recommendations for their future disposition. Extensive coordination and communication between the design team and the professional preparing the tree preservation plan is absolutely essential, especially at this stage, because successful integration of the development program and the existing natural environment is dependent upon clear and concise communication and coordination.

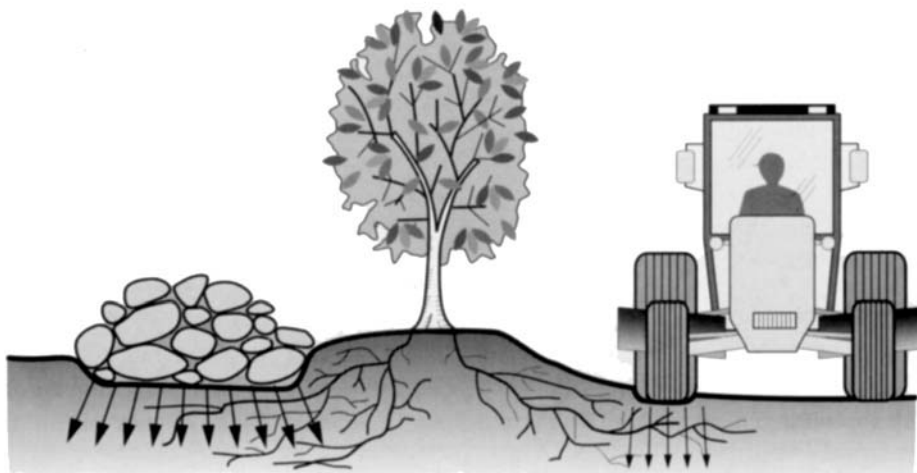
**During Construction and Beyond.** Open space preservation, as with tree and forest preservation, must be incorporated into the early planning process of site development. It can include the preservation of old field ecosystems, riparian areas of all types, specific geological formations, springs, etc. The preservation of these areas varies widely. Old fields are primarily made up of a mix of perennial flowers, grasses, and what some would classify as weeds. Many wildlife species rely on these old fields for foraging and reproduction. In fact, many song bird species rely strictly on old fields as their primary habitat. Old field areas are accustomed to some manipulation, such as annual bush hogging, grazing, or fire. Therefore, provided that these areas are not graded, and the seedbank is protected, old fields tend to be more resilient than forested areas during construction activities. However, the long-term development plan for such an area should recognize that some type of manipulation will likely be necessary in the long term to maintain these areas as open fields. The preservation of unique geological features (e.g. caves and natural springs) may require the review and recommendations of a geologist or hydrogeologist to ensure their preservation after a proposed change in drainage or compaction.

### Open Space and Tree Preservation Plan

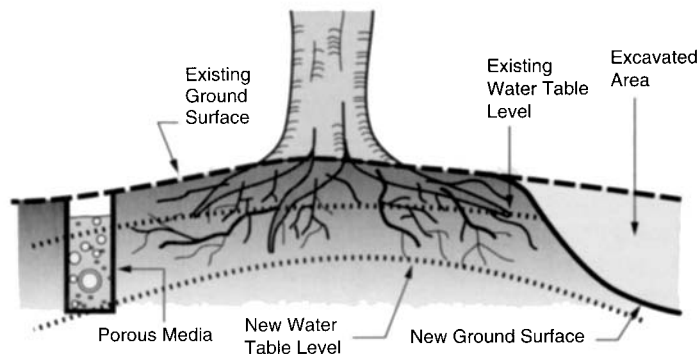
As the site development progresses from the preliminary to the final site layout phase, written detailed recommenda-



**Root Damage from Construction of Utility Trenches and Building Foundations**



**Root Damage from Prolonged Concentrated Loads Such as Loads from Construction Materials and Vehicles**



**Lowering of Water Table Level by Grade Changes and Seepage through Porous Media**

**FIGURE 17.2** Construction-related damage to tree roots.

tions regarding site specific preservation areas, tree preservation techniques, and remedial care of tree preservation areas are incorporated into the refined preservation plan. The plans and specifications should include a written description and/or graphic depiction of the recommended protection fencing and its location relative to preservation areas, as well as preservation techniques employed to accommodate grade changes (tree wells, retaining walls, spring boxes, drainage, or aeration systems). In addition, the horticultural care of preserved trees, maintenance of open fields, and preservation or treatment of spring boxes should be incorporated into the final plans for all phases of construction, including utility installation, pre-construction treatment, and post-construction maintenance.

Generally, the graphic preservation plan and written specifications are included in the final site plans and become part of the construction permitting process. At this point, it is probable that the coordination of the preservation plan will be extended to include the appropriate local reviewing agencies. Larger jurisdictions may have a separate forestry or natural resources staff who coordinates these plans, whereas smaller jurisdictions may rely on the forestry and natural resources staff at the state level. Regardless of their size, many jurisdictions now include preservation plans as part of the legal construction documents; therefore, many jurisdictions assess penalties for preservation plan violations just as they would any other construction violation.

Additionally, many jurisdictions require the inclusion of a long-term protective legal agreement, usually in the form of a recorded conservation agreement, to protect the longevity of preservation areas. These conservation easements place restrictions or conditions on uses within the preservation areas, many of which are permanent and are recorded as land records.

Finally, successful preservation requires active, respectful communication between all participants for the duration of the project. This means not only between the professional members of the site development team but with anyone who will work on the site. For instance, an uninformed contractor, involved in the early stages of site construction can do unintentional but irreparable damage to prospective preservation areas. However, if the role and contributing value of each participant is effectively communicated, then the project succeeds and all benefit. Tree preservation is no longer incidental to land development, to be treated as an amenity; it has become both economically prudent and environmentally essential.

*"A town is saved not more by the righteous men in it than by the woods and swamps that surround it." Henry David Thoreau.*

## WETLANDS

Many development projects impact stream channels or wetlands that constitute waters of the U.S. and, as such, are regulated by the U.S. Army Corps of Engineers (Corps) under the Clean Water Act. Originally known as the Federal

Water Pollution Control Act Amendments of 1972 (PL 92-500), it was amended in 1977 and renamed the Clean Water Act (CWA), although you will not find a law specifically named the Clean Water Act. If you want to look it up, it is U.S. Code Title 33 (Navigation and Navigable Waters) Chapter 26 (Water Pollution Prevention and Control), Section 1251. Chapter 36 of the Title 33 deals with the Corps of Engineers authority to regulate development under Chapter 26 Section 1251. As written, the Clean Water Act's Section 404 statute regulates the discharge of dredged or fill material into navigable waters of the United States. As a result of a law suit settlement in 1975 (*Natural Resources Defense Council v. Calloway*, 392 F. Supp. 685 D.D.C 1975), jurisdictional authority was extended to non-navigable waters and wetlands.

This authority has been challenged several times as to the actual jurisdictional reach intended by the CWA, and it continues to be challenged. Most recently the Corps authority to regulate isolated wetlands has been of great debate. Isolated wetlands are those wetlands that have no visible evidence of a surface water connection to navigable waters.

A recent ruling (Jan. 2001) by the U.S. Supreme Court (*Solid Waste Agency of Northern Cook County v. U. S. Army Corps of Engineers*) found that the isolated waters in question did not meet the definition of "navigable waters" under the CWA and, further, that the use of the "Migratory Bird Rule" linking the regulatory authority to the Intrastate Commerce was not supported by the CWA. This has allowed the filling of isolated wetlands, many of which are havens for rare and endangered species, as well as filters for groundwater resources.

Historically, federal wetlands regulations allowed draining and channelization of wetlands. However, as the result of settlement of a lawsuit in early 1992 (North Carolina Wildlife Federation, et al. v. Tulloch, Civil No. C90-713-CIV-5-BO (E.D.N.C. 1992) the Corps and the Environmental Protection Agency (EPA) agreed to extend their regulatory authority to include mechanized land clearing, ditching and channelization in wetlands as ditching activities resulted in a discharge of fill material from the excavation activity "incidental fallback." This extended regulatory reach was challenged in 1998 and defeated, allowing wholesale excavation and draining of wetlands provided the dredge material was placed in an upland area, not side cast into wetlands or waters. According to the EPA Office of Water, between the 1998 ruling and April 2001, upwards of 20,000 acres of wetlands and 150 miles of streams were targeted for destruction as a result of this loophole. Consequently, the EPA and the Corps published a ruling that modifies the definition of "discharge of dredge material." This definition, found in 33 CFR part 323.2, now states: "the Corps and EPA regard the use of mechanized earth-moving equipment to conduct landclearing, ditching, channelization, instream mining or other earth-moving activity in waters of the United States as resulting in a dis-



charge of dredged material unless project-specific evidence shows that the activity results in only incidental fallback.” In other words, all who propose to conduct excavation work in waters of the U.S. must notify the federal regulatory authority and prove that the ditching results in only “incidental fallback.”

For now, the Corps of Engineers has backed down on regulating isolated waters and wetlands and has asked for proof of impact for excavation activities. However, other suits are pending. To combat wetland destruction, many states have closed this loophole by passing regulations on a statewide level.

### Wetland Functions

After a long history of wetland destruction, wetlands are increasingly recognized as providing desirable functions to society. Today, wetlands are known to act as chemical sinks or transformers that can improve water quality. They act as “kidneys of the landscape” (Mitsch and Gosselink, 1986) by removing excess nutrients in rivers and streams. At the same time, wetlands reduce flooding by desynchronizing flood flows in drainage basins. This is done by storing storm water and releasing it slowly into streams, thus reducing peak flows. When located along streams and bays, wetlands can buffer erosive forces and hold sediments, preventing loss of shoreline. Wetlands can recharge or discharge aquifers, depending upon their position on the landscape, and therefore may either replenish the aquifer or contribute to base flow in streams.

### Water Quality

As water purifiers, wetlands act to reduce nutrients, chemical wastes, and turbidity. Nutrients such as phosphorus and nitrogen are removed from water by wetland plants and are utilized for growth. Utilizing the nutrients not only prevents algal blooms, but the increased biological oxygen demand created as the algal masses decompose can kill fish and other aquatic life as oxygen levels drop. A wetland's ability to absorb nutrients varies between individual wetlands and wetland types. Furthermore, a wetland can be overloaded with nutrients and thus be unable to assimilate all of the nutrient load.

Secondary and tertiary treatment of sewage wastewater has been demonstrated on numerous projects allowing effluent to flow through wetlands. Organic compounds are decomposed and nitrogen is converted to its gaseous form and removed (Mitsch and Gosselink, 1986). Heavy metals adsorbed to sediments fall out as sediments accumulate. As sediments fall out of the water column because of reduced flow velocity, turbidity is reduced allowing sunlight to penetrate to support growth of submerged aquatic vegetation (SAV).

### Fish and Wildlife Habitat

Wetlands are among the most productive ecosystems in the world in terms of photosynthetically producing energy from

the sun and recycling nutrients. They provide food and habitat for an array of wildlife and are critical in the nesting, migration, and wintering of waterfowl. While not accounting for nearly as high a percentage of our total land area, wetlands are critical to the life history of over one-third of our nation's threatened and endangered plant and animal species. Coastal wetlands provide spawning habitat, as well as nursery and feeding areas, for much of our commercially important fish and shellfish.

As an example, consider the role of SAV in an estuary such as the Chesapeake Bay. Historically, vast numbers of waterfowl (primarily redhead and canvasback ducks) wintered in the freshwater areas of the Bay where wild celery beds were abundant. Declines in SAV in the Bay and its rivers have led to a decline in the number of canvasbacks and redheads overwintering in the area. In addition, commercially important fish species such as striped bass, shad, and menhaden are dependent upon the Bay's SAV for spawning, feeding, and nursery habitat. Though likely not the sole reason for their diminished numbers, as SAV declined in the Bay, the commercial and sport fishery for striped bass was reduced to the point where all fishing for the species was banned for several years in the 1980s.

### Effects on Flooding

Wetlands within a watershed can reduce flood flows along streams through the storage of floodwaters they provide. Floodwaters are retained by the wetland and released gradually after the storm. The relative ability of a wetland to alter flood flows depends on several variables, including its size relative to the size of the watershed, its relationship to other wetlands in the watershed, and the amount of urbanization in the watershed. In many respects, wetlands serve as a natural form of stormwater management. Coastal wetlands provide similar benefits in terms of reducing the potential flood damages associated with tropical storms, hurricanes, northeasters, and the like. The wetland vegetation dissipates wave energy, thus reducing potential wave heights, while coastal wetlands serve as a buffer to protect shoreline areas from wave-induced erosion.

### Recreation

In addition to commercially important fish and shellfish that depend on wetlands for all or part of their life cycle, many of the species pursued by sport fishermen are spawned or reared in our coastal wetlands. Hunting, especially for waterfowl, is ever popular and generates substantial benefits to local economies. Waterfowl hunters spend over 300 million dollars annually on guns, ammunition, lodging, guides, boats, decoys, and other equipment in pursuit of their quarry (Feierabend and Zelazny, 1987). In addition to consumptive uses like hunting, fishing, and trapping, many individuals spend a great deal of time (and money) on non-consumptive uses such as bird watching, hiking, and canoeing in wetlands. The EPA estimates that

over 50 million people annually visit wetlands for bird watching (USEPA, 1988).

### Wetland Losses

More than 50% of the wetlands in the continental U.S. have been destroyed in the last 200 years, according to the U.S. Fish and Wildlife Service (Tiner, 1984). That is, of the approximately 215 million original acres, only about 99 million acres remained by the mid-1970s. Between the mid-1950s and mid-1970s, a net loss of approximately 9 million acres of wetland occurred. Drainage of wetlands for agriculture was responsible for 85% of losses, with development causing approximately 13% of the losses. Most losses due to agriculture occurred in the Lower Mississippi Valley and in the Prairie Pothole Region of the upper midwest. Bottomland hardwood forest and marshes, respectively, were affected in these regions.

The loss of wetlands from urban and suburban development was greatest in coastal states. Over 90% of the wetlands in California have been drained for agriculture or filled for development since initial settlement. Over two-thirds of wetland losses in Delaware between the mid-1950s and mid-1970s (Tiner, 1984) were attributable to development.

Most recently, the greatest wetland loss in the U.S. has occurred in the coastal marshes of Louisiana. Approximately 40 sq mi of freshwater and brackish water are converted to open water and more brackish wetland each year. Causes of this destruction are both man-made and natural. Land subsidence, rising sea levels, and manipulation of the Mississippi River and its watershed are to blame (Tiner, 1984).

### Wetlands Defined

Wetlands, as defined by the U.S. Army Corps of Engineers, means “those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.” (33 CFR Part 328.3b)

Wetlands are a varied resource with many names, usually associated with the prevalent vegetation. When wetlands share the same type of vegetation, they can take the same name. For example, forested wetlands can be called swamps, and those dominated by grasses are marshes. No two wetlands have identical plant species diversity or the same hydrologic and soil conditions. Generally, what all wetlands have in common is soil saturation or inundation for extended periods (i.e., wetland hydrology) and vegetation adapted to these soil conditions (i.e., hydrophytic vegetation).

The most commonly encountered classification system of wetlands was developed by the U.S. Fish & Wildlife Service (FWS) (*Classification of Wetlands and Deepwater Habitats*

*of the United States*, Cowardin, et al. 1979). The structure of this classification is hierarchical and includes five systems further defined by subsystems, classes, subclasses, dominance types, and modifiers. The palustrine system includes all non-tidal wetlands dominated by trees, shrubs, persistent emergent, emergent mosses or lichens, and all tidal wetlands where salinity is less than 0.5%. Classes under the palustrine system describe the most common vegetative categories, including emergent, scrub-shrub, and forested wetlands. These vegetative classes are further defined by subclasses that describe the dominant type of vegetation, examples of which are listed in Table 17.1.

Wetlands can also be assigned modifiers that describe the water regime or hydrologic conditions of the area, water chemistry, soils, and other special conditions. (See Table 17.2).

As can be seen, the use of such a system allows for a fairly detailed description of a particular wetland area. In addition, the use of such familiar terms as marsh, bog, or swamp may mean different things in different regions of the country. The use of the FWS system not only describes a particular wetland more completely but also reduces confusion between different regional interpretations.

Wetlands are located somewhere along a “natural wetness continuum” that exists on the landscape. Locating the boundary between wetlands and uplands (non-wetland areas) on this continuum is the objective of wetland delineation manuals. The definition of what constitutes a jurisdictional wetland has proven controversial; many versions of a single delineation manual have been generated with the full complement of federal agencies contributing. However, for one reason or another, the currently accepted version of the manual is the *U.S. Army Corps of Engineers Wetlands Delineation Manual* (Environmental Laboratory 1987).

### Methodology Criteria

The methodology for determining the presence of a wetland is based upon three elements that define wetlands: soils, vegetation, and hydrology. For an area to be identified as a jurisdictional wetland, the area must contain these three wetland criteria: the presence of hydric soils, hydrophytic vegetation, and wetland hydrology, or evidence thereof as in disturbed areas. Field indicators of the three wetland criteria are provided in the manuals, and when the field indicators of each of the criteria are observed, the area is determined to be a wetland.

Hydric soil is defined as “a soil that is saturated, flooded, or ponded long enough during the growing season to develop anaerobic conditions in the upper part (USDA-SCS, 1991).” These soils are typically recognized in the field by observing soil color or the presence of a thick organic layer. Hydrophytic vegetation is defined as plant life growing in water or on a substrate that is at least periodically deficient in oxygen as a result of excessive water content (Environmental Laboratory, 1987).

**TABLE 17.1 Classification Hierarchy of the Palustrine System Showing Class, Subclass, and an Example of Dominant Vegetation**

PALUSTRINE SYSTEM		
CLASS	SUBCLASS	EXAMPLE
Emergent	Persistent	Cattail, Cordgrass
	Nonpersistent	Pickerelweed
Scrub-Shrub	Broad-leaved Deciduous	Alder, Buttonbush
	Needle-leaved Deciduous	Tamarack (sapling)
	Broad-leaved Evergreen	Inkberry, Bog Laurel
	Needle-leaved Evergreen	Pond Pine (sapling)
Forested	Broad-leaved Deciduous	Red Maple, Tupelo Gum
	Needle-leaved Deciduous	Bald Cypress
	Broad-leaved Evergreen	Sweet Bay, Mangroves
	Needle-leaved Evergreen	Northern White Cedar

From Cowardin, L.M.V. Carter, F.C. Golet, and E.T. Larue. 1979. *Classifications of Wetlands and Deepwater Habitats of the United States*. Washington, D.C.: U.S. Fish and Wildlife Service.

**TABLE 17.2 Modifiers to be Applied at Class Level and Lower in the Classification Hierarchy**

MODIFIER	EXAMPLE	
Water Regime	Nontidal	Permanently Flooded
		Intermittently Exposed
		Semipermanently Flooded
		Seasonally Flooded
		Saturated
		Temporarily Flooded
		Intermittently Flooded
		Artificially Flooded
Water Chemistry	Salinity	Saline
		Mixosaline
		pH
		Circumneutral
		Alkaline
Soils		Mineral
		Organic
Special		Excavated
		Impounded
		Diked
		Partly Drained
		Farmed

From Cowardin, L.M.V. Carter, F.C. Golet, and E.T. Larue. 1979. *Classifications of Wetlands and Deepwater Habitats of the United States*. Washington, D.C.: U.S. Fish and Wildlife Service.

The hydrology of a wetland is the most important element that defines the wetland type and function. Water that creates wetlands comes from several sources, including groundwater discharge, out-of-bank stream flow (flooding), surface runoff of precipitation or snow melt, direct precipitation, or tidal flooding. The roles of groundwater and surface water in creating wetlands are described in Figure 17.3. As expressed in the figure, wetlands are located between the seldom flooded upland and perennial waters of creeks, rivers, and bays.

The presence or absence of wetland hydrology is determined using recorded data and field indicators. Recorded hydrologic data, such as stream, lake, or tidal gauge records, can provide information on the duration of flooding in areas adjacent to the water body. In groundwater-driven wetlands, the county soil survey can be useful in determining duration of soil saturation.

Wetland hydrology in the form of surface water or saturated soils is readily apparent during certain times of the year in most wetlands and in other types of wetlands during most of the year (Federal Interagency Committee for Wetland Delineation, 1989). Evidence of this could be standing surface water or muddy ground surface underfoot. However, at those times of the year when surface water isn't evident, wetland hydrology may not be as obvious. For these reasons, field indicators are used primarily to confirm the presence of wetland hydrology when hydrophytic vegetation and hydric soil are determined to be present. An example of obvious wetland hydrology is shown in the photograph of Figure 17.4, a wetland created by a beaver dam. A much less obvious wetland, created by groundwater discharge, is shown in the photograph of Figure 17.5.

Vegetation that grows in wetlands has adapted to permanently or periodically saturated/inundated soils and is called hydrophytic vegetation. Many plants have adapted to

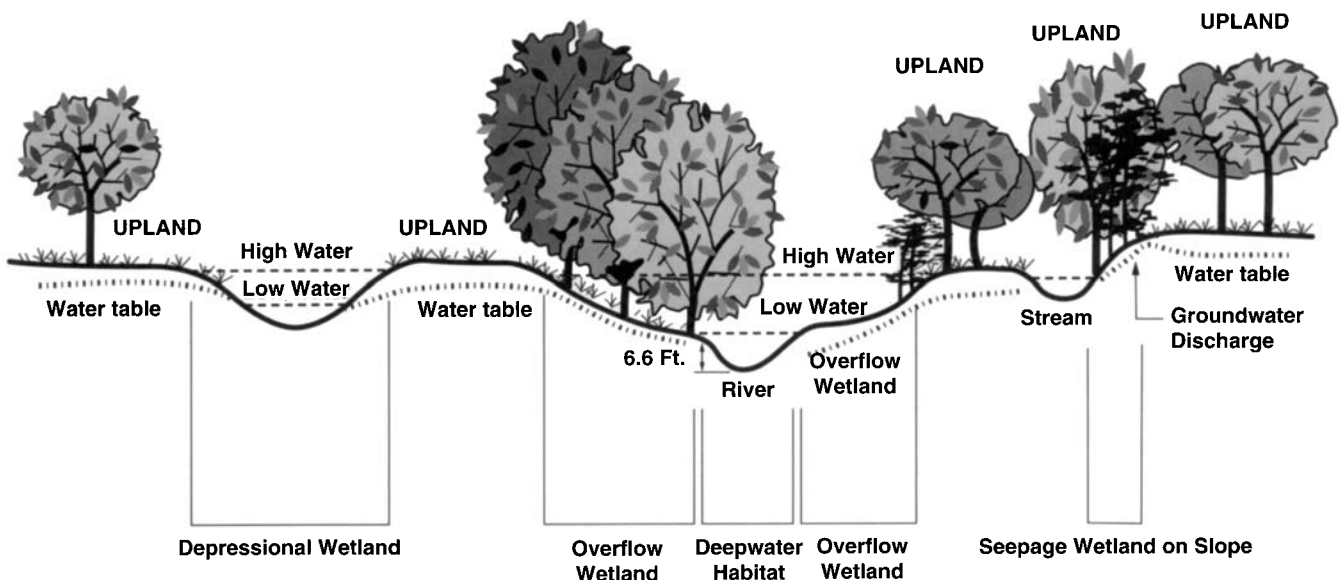


**FIGURE 17.4** A wetland created by a beaver dam built on a stream channel.

these conditions to create the great diversity of plants that grow in wetlands. About 7,000 plant species are included on the Fish & Wildlife Service's *National List of Plant Species that occur in Wetlands: 1988* (Reed, 1988). This list provides habitat information and a wetland indicator for each species. The wetland indicator is an estimate of the probability of a species occurring in wetland versus non-wetland. A hydrophytic vegetation determination is based upon the wetland indicator status of all dominant plant species in an area.

### Wetland Investigation—Off Site

When determining the development potential of a particular piece of land, the developer should first evaluate the land's constraints. One of the first things to do beyond a Phase I environmental site assessment is to determine the presence of wetlands and other waters of the U.S. on site.



**FIGURE 17.3** Schematic diagram showing wetlands, deep-water habitats and uplands on landscape. (Adapted from Tiner, 1984.)



**FIGURE 17.5** A palustrine emergent wetland created by ground water discharge at the surface.

Typically, this process begins with an office investigation of available information. At a minimum, an office investigation should be undertaken to determine the likelihood of wetlands on the site.

Examples of readily available information are discussed below.

**U.S. Geological Survey (USGS) 7.5 Minute Topographic Series Maps.** The largest scale these maps are available in is 1:24,000, directly from the U.S. Geological Survey. Locate the site boundaries on the map and note any marsh symbols or lakes, ponds, rivers, and other water bodies on the site. Keep in mind that the USGS usually bases these hydrologic symbols on data from the driest time of year (Environmental Laboratory 1987). In addition, topographic detail is limited on these maps because the contour intervals are typically 5 or 10 ft.

**Wetlands Mapping.** Much of the U.S. has been mapped by the U.S. Fish and Wildlife Service (FWS), National Wetland Inventory (NWI). The USGS 7.5 Minute Topographic Series map (Quadrangle) is the base for these maps. The NWI map is prepared from aerial photos, and wetlands are designated according to the Fish and Wildlife Service's *Classification of Wetlands and Deepwater Habitats of the United States* (Cowardin, et. al. 1979). If the NWI map indicates wetlands on the site, there is a high probability (>90%) that they are jurisdictional wetlands, unless the water regime is temporarily or intermittently flooded. Wetlands with either of these two water regimes may not be wet long enough to be considered jurisdictional wetlands (Environmental Laboratory, 1987). Water must be present for approximately 12.5% of the growing season or greater in order for a wetland to be considered to exhibit the hydrology criteria. Growing seasons range from county to county and can be found within your local Soil Conservation Service (now the Natural Resources Conservation Service) soil maps. State and local wetland programs may have created wetlands mapping that includes the site area. For example,

the state of New York has mapped all of its wetland resources of at least 12.4 acres (about 5 hectares), or less, if of unusual local importance.

**Soil Conservation Service (NRCS) County Soil Surveys.** The U.S. Department of Agriculture, SCS soil survey of a county, if available, will indicate the soil map units that occur on the site. If there are hydric soil map units or any soil map units with hydric soil inclusions indicated, then one could assume that these areas are potential wetlands. This should be field verified. Greater confidence in that determination would be obtained if the NWI map also showed wetland in this area. If no hydric soils or hydric soil inclusions are indicated in the study area, they can probably be eliminated from further off-site investigation, as they will likely lack the required soil criteria, but they should be field checked with a quick walkover for verification. Remember, the SCS did not likely walk every square inch of your site, and omissions of some hydric soils could occur.

**Aerial Photographs.** Aerial photographs can provide information on vegetation and hydrology in making off-site, or desktop wetland determinations. Possible sources for aerial photographs are county or city mapping or planning offices and the U.S. Department of Agriculture, Agricultural Stabilization and Conservation Service (ASCS) or NRCS offices located in each county.

Many counties, especially those with growing populations, will regularly commission aerial photo surveys to update county land-use mapping. Color infrared (CIR) photography is the most useful when identifying wetland areas from the desktop with aerial photographs as it can highlight differences in vegetation and soil moisture. The county ASCS office may have available CIR photos for the farmed portion of a county. These are used to determine farm subsidies and to enforce the Swampbuster provisions of the latest version of the "Farm Bills." They can also assist in your off-site or desktop wetland delineation.

In addition, local governments will obtain aerial photos to create topographic maps or to update older land-use mapping. Similar photos, possibly at a larger scale, may be available for the site if it has been photographed for the development of detailed topographic mapping.

### Preliminary Wetlands Investigation

A preliminary wetland investigation starts where the office investigation left off. The information from the office investigation is used in the field to identify and to delineate wetlands on the site more accurately. Ideally, the field study should be done by a team of at least two individuals familiar with the soils, vegetation, and hydrologic indicators in the region. Wetland boundaries should be field sketched onto the best available topographic mapping. Field notes, and/or wetland data sheets should record the hydrologic conditions, soil characteristics, and dominant vegetation in each of the wetland types encountered. Notes on the rationale behind the boundary delineation should be made,

including characteristics of the upland areas at the wetland/upland boundary. Some surveyor's flagging may be tied to vegetation or wire flags placed in the ground to delineate roughly the wetland boundary for later capture by field survey.

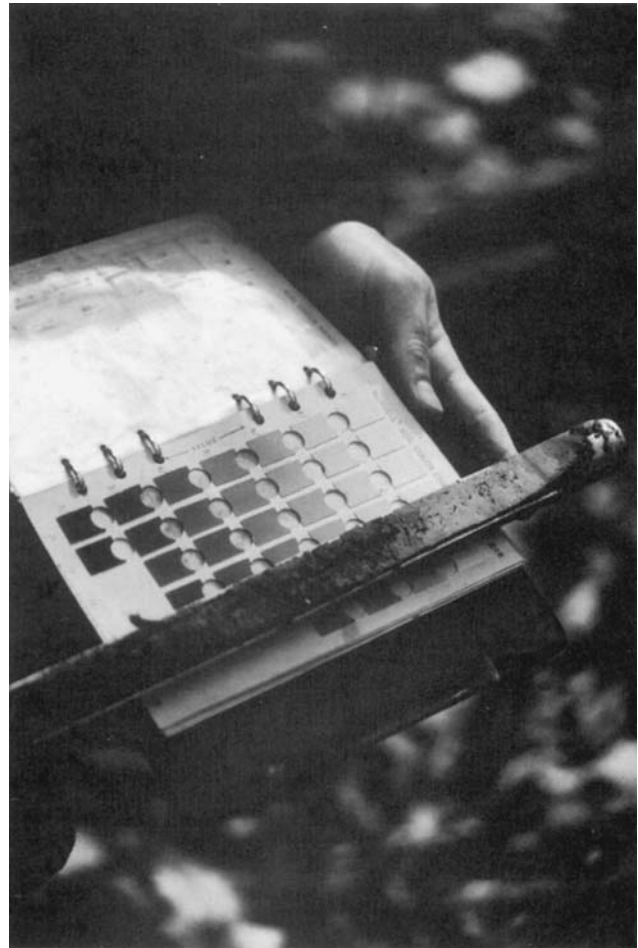
Although the preliminary field investigation should focus upon areas mapped with hydric soils and hydric soil inclusions, and/or areas mapped as wetland on NWI maps. All of the drainage swales and low-lying areas should be visited to identify any and all wetlands and to identify the stream channels that qualify as waters of the U.S.

### Wetland Delineation Criteria

In the field, professionals are utilizing the approved field delineation manual to evaluate the three primary characteristics of a wetland—that is, the things that make a wetland “wet,” as described earlier. In order for an area to be classified as a wet, the anaerobic conditions in the soil, which create certain chemical and morphological properties, are readily observed. For example, soil color is recorded utilizing Munsell Soil Color Charts (Munsell Color, 1992). The color of a soil is described with three variables: hue, value and chroma. The hue variable is used to relate the color of the soil as either red, yellow, green, blue, or purple. The value indicates the color's degree of lightness, and the chroma is its strength. In the Munsell Soil Color Charts book, each hue has its own page. Value and chroma are shown on the vertical and horizontal axes, respectively, of each page. A numeric scale on each axis increases with increasing value (lightness) and increasing chroma (strength).

Typically, mineral soils subjected to prolonged saturation or inundation express the condition as gleization and mottling. Gleization is seen as a predominant neutral gray to sometimes greenish or bluish gray color (low chroma) in the upper part of the soil profile. Reduced compounds are easily leached from the soil under saturated anaerobic conditions, and the low chroma indicates an absence of iron. Mottling represents incomplete reduction of the soil's iron and is seen in the upper part of the profile as mosaics of brown or red oxidized soil interspersed with low chroma-reduced areas. Typically, the grayer or more gleyed the soil, the longer the duration of soil inundation or saturation. The dominant color of the soil (>50%) is called the matrix and the remainder, mottles. In most cases, when the matrix has a chroma of two or less with mottles present, or one or less without mottles, the soil is hydric. In some problem soils, such as sandy soils and soils derived from red parent material, the gleization may not be exhibited due to the extremely low (sandy) or extremely high natural concentration of iron. Wetland identification in these problem area soils may take slightly longer to identify in the field, and require detail beyond the scope of this chapter.

Organic hydric soils are largely an accumulation of non-decomposed plant remains (peat). These soils are easily recognized as hydric soil by a thick organic surface layer that is saturated for most of the year.



**FIGURE 17.6** A gleyed hydric soil sample being compared to the Munsell Soil Color Chart

Hydrophytic vegetation is plant life that has adapted to saturated soil conditions. The field indicators of hydrophytic vegetation include the identification of the dominant tree, sapling-shrub: herbaceous, and woody-vine species. A wetland indicator status from the *National List of Plant Species that Occur in Wetlands: 1988* (Reed, 1988) is assigned to each species of plant to determine the probability of finding that plant growing under saturated conditions. This status indicates the frequency of occurrence of a species in wetlands as shown in Table 17.3.

Under the current *Corps of Engineers Wetlands Delineation Manual*, more than 50% of the dominant species must have an indicator status of OBL, FACW, or FAC for an area to meet the hydrophytic vegetation criterion.

Wetland hydrology, generally the most difficult of the three criteria to verify in the field, is normally confirmed through the use of field indicators. The wetland delineator must keep in mind that while some indicators are not necessarily indicative of the presence of wetland hydrology during the growing season, they do provide evidence that inundation or soil saturation has occurred. Field indicators listed in the Corps manual include the following:

**TABLE 17.3 Indicator Categories Showing the Estimated Probability of a Species Occurring in Wetlands vs. Non-Wetlands**

INDICATOR STATUS	PROBABILITY OF OCCURRING IN WETLANDS
Obligate Wetland (OBL)	>99%
Facultative Wetland (FACW)	67–99%
Facultative (FAC)	34–66%
Facultative Upland (FACU)	1–33%
Upland (UPL)	<1%

- Visual observation of inundation or saturation
- Watermarks indicating prior inundation
- Driftlines of water-carried debris
- Sediment deposits
- Drainage patterns within wetlands
- Oxidized rhizospheres

The current Corps delineation methodology requires that positive indicators of all three wetland criteria be present to delineate an area as wetland. The wetland-upland boundary is determined where one or more of the criteria falls out. The only exception to this is the situation where one or more of the criteria is not present due to a disturbance to the area. A common example of a disturbed condition is the situation where hydrophytic vegetation is not present because of farming activities that have temporarily replaced the natural vegetation with planted crops or pasture.

The accuracy of a preliminary wetlands investigation is lessened when the topographic relief is low and the wetland is not confined to distinct stream valleys. A lack of landmarks in areas of low relief also limits the accuracy of the preliminary study because sketching the wetland boundary in the field is an approximation and landmarks aid in placing it correctly. All of these factors influence the reliability of the preliminary wetlands study.

### Detailed Wetland Investigation

A detailed wetland investigation offers the most accurate delineation and measurement of the jurisdictional area on the site. The detailed study builds upon the office wetlands investigation and the preliminary study, if one was done. This level of effort is necessary when the reliability level of the preliminary study is not sufficient or the exact location

of the boundary and/or total wetland acreage must be accurately known. A wetland boundary would need to be accurately known when designing a project so as to avoid wetland impacts. Accurate wetland acreage is also necessary when determining the proposed wetland fill impacts of a project for the permitting process.

An accurate acreage can be used to confidently determine eligibility for general permits (discussed in the following section), for example. Furthermore, if wetland creation is part of the permit conditions, precise wetland acreage is important because of the high cost of creating replacement wetland.

As described earlier, the basic premise in defining the wetland-upland boundary is to determine the point where one or more of the mandatory technical criteria is not met. In its simplest form, at Point A, indicators of all three criteria are present and the area is determined to be a wetland; at Point B, one or more of the criteria is not met and the area is upland. Delineating the boundary is a matter of determining where, between Point A and Point B, one of the parameters changes.

In some areas, especially where an abrupt change in topography or a distinct difference in vegetation occurs, the boundary delineation is straightforward. However, the situations where the change is very gradual generally are the most difficult areas to delineate. A detailed investigation requires a much more intensive sampling effort, with more sampling points. The boundary is then flagged with surveyors ribbon and located by field survey.

### The Regulatory Process

The simplest strategy for dealing with wetlands on a particular project is to know the location of the jurisdictional areas (i.e., those areas regulated by the Corps) and avoid them in the development process. However, in many in-

stances, total avoidance is not practical (e.g., access and utilities), and some involvement with the regulatory process is inevitable.

The degree of involvement in the regulatory process depends on the amount of jurisdictional area that is proposed to be affected. Almost all regulatory programs have a minimum threshold of wetland impacts that, once crossed, propel one into the regulatory process. Generally, a zero threshold is established for tidal wetlands and non-tidal wetlands located below the headwaters, and, as such, any proposed fill in these areas requires permits. Headwaters are defined in the Corps regulations as the area upstream of the point where a stream's average annual flow falls below 5 cubic ft per second, or in the Northeast U.S. approximately a 5 square mile drainage area. Currently, the Federal Clean Water Act's Section 404 threshold for non-reporting impacts to non-tidal headwaters including wetlands is  $\frac{1}{10}$  acre, provided you meet the general conditions of the nationwide general permit program. General conditions, essentially the fill activity, cannot affect historic sites, endangered species habitat, FEMA floodplains, etc.

Limiting impacts to less than the  $\frac{1}{10}$  acre of fill might be accomplished by clustering buildings in upland areas connected by road crossings of wetlands with minimal fill impact. Another technique used to minimize wetland impacts is to use the existing wetlands as an amenity to the development, particularly if some open water exists or can be created. Open space, buffers, or green space requirements of some zoning ordinances can be satisfied utilizing the wetland area that is avoided by development.

Many states initiated programs to protect wetlands from a range of activities that the Federal Section 404 regulations did not address. Typically, state wetland protection laws regulated draining, channelization or clearing of vegetation. In addition, some states have gone further to protect those areas by requiring buffer areas adjacent to wetlands to prevent damage to the resource. Many states have used the authority to issue or deny water quality certification under Section 401 of the Clean Water Act to regulate some fill impacts to wetlands. A state can deny water quality certification of a proposed wetland fill, under the premise that the state's water quality standards are not being met.

The types of wetlands involved and the degree and magnitude of the potential impacts will dictate the type of wetland permit required. At the Federal level, the Corps can authorize projects through either individual or general permits. Individual permits undergo a "public interest" review by the Corps, where factors such as property ownership, economics, aesthetics, environmental concerns, historic values, fish and wildlife values, flood damage prevention, land use, navigation, recreation, water supply, water quality, energy needs, safety, and food and fiber production, must all be considered and balanced. Such individual permits are granted by the Corps unless their issuance would be contrary to the public interest.

General permits are an authorization from the Corps issued on a regional or nationwide basis for activities that cause only minor individual or cumulative environmental impacts. Authorization of a project by general permit often requires little or no review by the Corps, provided specific conditions can be met. Nationwide permits are a type of general permit implemented through the Corps Nationwide Permit Program Regulations (33 CFR Part 330). Generally speaking, in the interest of time and cost, it is more desirable to design a project in such a manner that impacts are authorized by general permit, rather than individual permit.

The individual states each have a role in the Federal permitting process through the authority of Section 401 of the Clean Water Act. The state must certify that the granting of a 404 permit by the Corps will not violate state water quality standards. Depending on the particular state and the type of project, an individual water quality certification may be required or the project may qualify for a "blanket" certification issued to the Corps for certain activities. Some states have their own wetland permit program based in part on their authority and responsibilities under Section 401.

The Federal review of a permit application to fill or alter wetlands follows the guidelines of Section 404(b)(1) of the Clean Water Act. The use of these guidelines is addressed in the February 6, 1990 Mitigation Memorandum of Agreement (MOA) between the Corps and the EPA. This MOA sets forth the sequencing procedures to be followed by the agencies in determining whether a proposal avoids, minimizes, and compensates for impacts.

Avoidance, as used in the 404(b)(1) Guidelines, allows for the issuance of a permit only for the least environmentally damaging practicable alternative. The Guidelines also introduce the presumption that alternatives for non-water dependent activities exist that do not involve impacts to special aquatic sites, including wetlands.

Once an alternatives analysis has demonstrated that the proposed action is the least environmentally damaging option, the Guidelines require that practicable steps to minimize unavoidable impacts be investigated. Minimization may include project modifications or permit conditions. For example, time-of-year restrictions are often imposed as a permit condition for bridge or culvert construction to minimize impacts to the movement of aquatic life or to spawning fish. The Guidelines define "practicable" as being available and capable of being done after taking into consideration cost, existing technology, and logistics in light of overall project purposes.

Finally, when alternatives, avoidance, and minimization have been considered to the reviewing agency's satisfaction, compensation for unavoidable impacts is normally required. Compensation types can be very diverse, ranging from the use of vegetated buffers adjacent to waterways onsite, in the form of restoration of degraded wetlands, or the creation of man-made wetlands. Man-made wetlands can be created on or off site, although they are typically



preferred within the same river basin. In addition, mitigation can also occur in an approved "mitigation bank." Mitigation banks are large man-made wetland areas that offer mitigation credits for sale. The numbers of credits you will need will be based on the types and amount of your wetland impacts. These banks have gone through rigorous regulatory approvals to insure the created wetland success. Also, if approved by the agencies, the developer could opt to utilize an "in lieu fee." This is a fee based on the cost per acre of creating mitigation sites in the general vicinity of the project. The "fee" is placed in a fund to pay for the creation of mitigation banks within the watershed of impact. This is especially useful in highly developed watersheds where potential mitigation creation areas are scarce. It should be noted that federal regulatory agencies typically prefer on-site mitigation to mitigation banks or in lieu fees, when it is feasible, and can provide similar benefits to the wetlands lost.

The primary goal of compensatory mitigation is to replace the functional values of the resource to be impacted. Creating new wetlands is expensive and opinions vary widely on the true potential for success of man's attempts to replicate a natural system. Thus, wetland creation should not be viewed as a simple trade-off for the right to affect natural wetlands. The regulatory agencies do not view it in this manner, and a successful permit application for a land development project involving wetland impacts will have considered these steps of alternatives analysis, avoidance, and minimization throughout the design process.

### Possible Consequences of Violation

Administrative and civil action or criminal prosecution are possible with violations of the regulations and have historically been pursued in that order. Most violations are corrected through administrative procedures, such as orders to restore affected areas to their original condition, according to a 1989 Memorandum of Agreement between the EPA and Corps. After-the-fact permits cannot be issued "until all administrative, legal and/or corrective action has been completed." Flagrant or repeat violators can be setting themselves up for fines of \$10,000 per violation up to a maximum of \$25,000 for minor violations. Major violation penalties can reach \$10,000 per day up to a maximum of \$125,000 and/or jail time.

### REFERENCES

- 16 U.S.C. 1531 et seq. Endangered Species Act, 1973.
- 16 U.S.C. 661 et seq. The Fish and Wildlife Coordination Act, 1964.
- 29 CFR Parts §1910.1000 et seq. (1991)
- 40 CFR pts. 260-299 (1991)
- 55 Fed. Reg. 48,065, 1990.
- 56 Fed. Reg. 50,759-50,770, 1991.

Adamus, Paul R., et al., *Wetland Evaluation Technique (WET); Volume Literature Review and Evaluation Rationale*, USAE Waterways Experiment Station, Environmental Laboratory, Vicksburg, Mississippi, 1991.

- Corbitt, Robert A., *Standard Handbook of Environmental Engineering*, McGraw-Hill, N.Y., 1990.
- Cowardin, L. M., V. Carter, F. C. Golet, and E. T. Laroe, *Classification of Wetlands and Deepwater Habitats of the United States*, U.S. Fish and Wildlife Service, Washington, D.C., 1979.
- Darr, Lonnie, *A Technical Manual for Woodland Conservation with Development in Prince Georges County*, Maryland-National Capital Park and Planning Commission, Upper Marlboro, Md., 1990.
- DeChiara, Joseph, and Lee E. Koppelman, *Time-Saver Standards For Site Planning*, McGraw-Hill, New York.
- The Environmental Audit Handbook*, Business & Legal Reports, Inc.: Madison, Connecticut, 1990.
- Environmental Laboratory, *Corps of Engineers Wetlands Delineation Manual*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1987.
- EPA National Pollutant Discharge Elimination System Permit Application Regulation for Storm Water Discharges, 40 CFR pts. 122, 123 and 124, 1990.
- Federal Interagency Committee for Wetland Delineation; U.S. Army Corps of Engineers, U.S. Environmental Protection Agency, U.S. Fish and Wildlife Service and the U.S. Department of Agriculture, Soil Conservation Service, *Federal Manual for Identifying and Delineating Jurisdictional Wetlands*, Washington, D.C.: U.S. Government Printing Office, 1989.
- Feierabend, J. Scott, and John M. Zelazny, *Status Report on Our Nation's Wetlands*. National Wildlife Federation, Washington, D.C., 1987.
- Final NPDES General Permits for Storm Water Discharges from Construction Sites 57 Fed. Reg. 41,176, 1992.
- General Accounting Office, *Wetlands: The Corps of Engineers' Administration of the Section 404 Program*, Washington, D.C., 1988.
- Harris, Richard W., *Arboriculture: Integrated Management of Landscape Trees, Shrubs and Vines*, 2d ed. Prentice Hall: Englewood Cliffs, New Jersey, 1992.
- Klapproth, Julia C., and James E. Johnson, "Understanding the Science Behind Riparian Forest Buffers: An Overview." Virginia Cooperative Extension, Publication Number 420-150. Posted Oct. 2000. See also <http://www.ext.vt.edu/pubs>
- Lerner, Steve, and William Poole, *The Economic Benefits of Parks and Open Space*, San Francisco, CA: Trust for Public Land, 1999. Also see <http://www.tpl.org>.
- Liebman, Lawrence R., *Developer's Guide to Federal Wetlands Regulations*, National Association of Home Builders: Washington, D.C., 1990.
- Macie, Edward A., and Gary Moll, *Shading Our Cities: A Resource Guide for Urban and Community Forests*, Gary Moll and Sara Ebenreck, eds, American Forestry Association, Island Press, Washington, D.C., 1989.
- McDaniel, Allen, "Roots: where are they?" The Virginia Gardener Newsletter, Vol. 6, No. 8. August 1987. See also <http://www.ext.vt.edu/pubs>
- Miller, Robert W., *Urban Forestry: Planning and Managing Urban Greenspaces*, Prentice Hall: Englewood Cliffs, New Jersey, 1988.

- Mitsch, W. J., and J. G. Gosselink, *Wetlands*, Van Nostrand Reinhold, New York, 1986.
- Moll, Gary, and Sara Ebenreck, eds, *Shading Our Cities, A Resource Guide for Urban and Community Forests*, American Forestry Association, Island Press, 1989.
- Munsell Color, *Munsell Soil Color Charts*, Munsell Color, Macbeth Division of Kollmorgen Instruments Corp., Baltimore, MD, 1988.
- National Association of Home Builders of the United States, *Land Development*. National Association of Home Builders of the United States, Washington, D.C., 1987.
- Neely, Dan ed, *Valuation of Landscape Trees, Shrubs, and Other Plants: A Guide to the Methods and Procedures for Appraising Amenity Plants*, 7th ed, International Society of Arboriculture, Urbana, Ill., 1988.
- O'Brien, James P., and William Harris Frank, *Environmental Due Diligence*, The Bureau of National Affairs, Inc., Washington, D.C., 1989.
- Parkin, William P., Marburg Assoc., *Site Auditing: Environmental Assessment of Property*, Specialty Technical Publishers, Inc.: Vancouver, B.C., Canada, 1991.
- Reed, P.B., Jr., *National List of Plant Species That Occur in Wetlands*, in cooperation with the National and Regional Interagency Review Panels, U.S. Fish and Wildlife Service, National Wetlands Inventory, St. Petersburg, FL, 1988.
- Salvesen, David, *Wetlands: Mitigating and Regulating Development Impacts*, ULI—The Urban Land Institute, Washington, D.C., 1990.
- Tiner, Ralph W., Jr., *Field Guide to Nontidal Wetland Identification*, Maryland Department of Natural Resources, Annapolis, Maryland and U.S. Fish and Wildlife Service, Newton Corner, Massachusetts, Cooperative publication, 1988.
- Tiner, Ralph W., Jr., *U.S. Fish and Wildlife Service Wetlands of the United States, Current Status and Recent Trends, National Wetlands Inventory Project*, U.S. Department of the Interior, Fish and Wildlife Service: Washington, D.C., 1984.
- U.S. Environmental Protection Agency, *America's Wetlands: Our Vital Link Between Land and Water*, Washington, D.C., 1988.
- U.S. Department of the Interior, National Park Service, *National Register Bulletin: How to Apply the National Register Criteria for Evaluation*, Publication #15, Washington, D.C.
- U.S. Environmental Protection Agency, *Guidance Manual for the Preparation of NPDES Permit Applications for Stormwater Discharges Associated with Industrial Activity*, Office of Water Enforcement and Permits, Washington, D.C., 1991.
- U.S. Environmental Protection Agency, *NPDES Storm Water Sampling Guidance Document*, Office of Water, Washington, D.C., 1992.
- U.S. Environmental Protection Agency, *Storm Water Management for Construction Activities: Developing Pollution Prevention Plans and Best Management Practice*, Office of Water Enforcement and Permits, Washington, D.C., 1991.
- U.S. Environmental Protection Agency, Office of Water.
- U.S.D.A. Soil Conservation Service, *Hydric Soils of the United States*, in cooperation with the National Technical Committee for Hydric Soils, USDA-SCS, Washington, D.C., 1991.



## CHAPTER 18

# HISTORIC PRESERVATION AND ARCHEOLOGY

*Elizabeth Couture, MURP, MARCH*

### INTRODUCTION

“Preservation” seeks to understand our human fabric as defined by the physical structures built and left by people before our time. Archeology tells the story of people at a particular time or within a particular place based upon the physical “things” left behind. Together they provide history. Moreover, such history may be extremely important to a specific land parcel, location, group of people (community), jurisdiction, state and/or our nation.

History energizes our goals for tomorrow. We look to the past for understanding, direction, patterns, and avoidance of mistakes.

### Example of Preservation

President John Adams enacted construction of the C&O Canal on July 4th, 1828, with the belief it would expand economic trade and land development west from Washington D.C., the Nation’s capital, along the Potomac River to locations in western Virginia and West Virginia. (See Figure 18.1.) Construction ended in 1850. Public and private investment exceeded 7.5 million. The canal was 184.5 miles long, up to 60 ft wide, 6 ft deep, one tunnel 3,118 ft long, and 74 locks. Each lock location spurred the economic growth of a small community. Severe floods in 1889 and



**FIGURE 18.1** C&O Canal.



**FIGURE 18.2** Photo of C&O Canal lock reconstruction.

1924 closed the canal. Re-construction failed after the 1924 flood.

Use of the canal diminished and it quietly closed years later. Preservation efforts began in 1971; canal remnants were determined significant to our Nation's history. Various sections of the canal in Maryland and Virginia were restored as public parks. Thirty years later, archeological and building reconstruction efforts remain strong.

Figure 18.2 shows the C&O Canal lock under reconstruction. Dewberry & Davis architects, engineers and resource staff started in 1998 inspecting and documenting all structures, parts of and whole, along the 184.5 path. Over 2,100 photographs and inspection finding reports are now on file for public use.

It is important to note that, even today, any land development effort with views of and/or a location near the canal hold accelerating economic value; people value the historic context and aesthetic setting. Clearly, President Adams' goal for expanded economic growth and development along the Potomac River has been realized.

### The Process

Land development is a continual process that reshapes our physical environment in response to the needs of people. By serving such needs, land development contributes to the inventory of historic resources for preservationist and archeological professionals tomorrow.

When the developer takes time to research a land parcel (or parcels) relative to the people it will serve, then the planning and creation of a new "place" will yield a product worthy of respect for generations to follow. This initial goal to understand people, their history and sense of place lays the foundation for developing a research file important to land development efforts.

### WHAT IS A HISTORIC RESOURCE?

The preservation movement has clearly shown with time that a clear-cut definition or set of terms for a historic re-

source does not exist. Age, typically defined as over 50 years, coupled with a profile of historic events, persons, architectural style, and/or the innovative use of building materials, once guided professionals to identify historic resources. Today, the task of definition is more complex. Other factors of local interest, vernacular styles, land use, quantity (supply) and ever-changing technological advances affect and modify traditional terms and resource definitions.

One example is the Quonset hut. (See Figure 18.3.) Manufactured by the thousands for military and later industrial use during World War II, the Quonset hut is on the edge of becoming a historic resource. It represents a building type unique to a time in history that now stands in diminishing quantities nationwide.

The Quonset hut, like railcars or diners, exemplifies the broadening of what defines a historic resource. Case studies nationwide, whether resolved by court rulings, financial investment decisions, citizen input, or public regulatory controls, support broader terminology to define historic re-



**FIGURE 18.3** Quonset Hut.

sources. Key terms like rehabilitation, restoration, repair, replacement, and/or reconstruction define different outcomes in a development effort. Similarly, terms of maintenance, stabilization and upkeep can translate into different costs and/or penalties where a historic resource is involved. Last, location terms like “historic setting,” “environment,” “district,” “conservation zone,” “distinct architecture,” “historic event,” and/or “landmark” indicate immediately that public government involvement is inevitable.

The point is simple: know the meaning of terms used in documents, plats, and conversations regarding a parcel or tract of land. Education yourself about local, state and federal publications pertinent to your development location. Secure and read publications by the U.S. Department of the Interior, The State Preservation Officer, and local municipalities pertinent to your property location that define a historic resource.

### THE FORMAL AND INFORMAL PUBLIC AND QUASI-PUBLIC SYSTEM

All levels of government that govern to a particular location define the “public” or formal system. The “quasi-public” or informal system centers on non-profit, organized, and often incorporated, community associations that play a variety of roles, direct and indirect, in historic preservation and archeology. Often their role is defined by the perceived effect that a development project will have on a location.

Government involvement with the land development process is exercised in three ways:

- *Legislature*: laws and ordinances
- *Taxation*: payment, credits, penalties, liens
- *Eminent Domain*: acquisition or condemnation for government use with “just compensation”

The extent and use of each regulatory power varies with state and local concerns, funding sources, permit requirements, penalties and precedent examples of similar preservation situations. Discussion of the public sector is divided into three parts: (1) Agencies; (2) Tools; (3) Strategies.

### Public Agencies: A Brief Description

An important federal government agency is the U.S. Department of the Interior which includes:

- *The National Park Service*: created under the Secretary of the Interior for the purposes of identifying, establishing and managing resources within our landscape of national importance during the late 19th Century.
- *The Historic American Buildings Survey*: born out of a “work program” for skilled and unemployed persons during the Great Depression that called for the preparation of architectural drawings, field notes, photography and site documentation of specific buildings

meriting national recordation. The survey was permanently established by the Historic Sites Act in 1935, mandating its administration to the Secretary of the Interior. Today the survey encompasses over 30,000 case records, providing the greatest repository of physical artifacts and architectural data in our nation.

- *The National Register of Historic Places*: created under the 1966 National Historic Preservation Act by Congress, the National Register lists over 60,000 properties, structures, districts, archeological materials, and elements of the landscape that are determined significant to the cultural, architectural, environmental, technological, and archeological heritage of our nation. The Register is maintained by the National Park Service and works in cooperation with The National Trust for Historic Preservation.

- Another important agency is the *U.S. Geological Survey*, which provides an in-depth database with maps and photography for the study of land. Agency locations are nationwide.

- *The Advisory Council on Historic Preservation*: established to assist the President and Congress, this appointed Commission serves to advise and arbitrate in cases where federal funds are proposed for a particular use that could result in an adverse impact to properties listed on “The National Register of Historic Places.”

- *U.S. Housing & Urban Development (HUD)*: activities at all levels of government and private development endeavors that utilize federal funds secured through HUD require an environmental assessment with specific provisions focusing on the safeguard of unique historic and archeological resources.

- The U.S. Department of Transportation (DOT): transportation projects as well as transportation corridor studies are subject to an environmental and historic resource assessment. This requirement has significant consequence given the obsolescence, aging and anticipated replacement needs of the transportation infrastructure nationwide. The importance of this is heightened by the location of many of these elements in close proximity to land uses and structures of historic and archeological merit. These may include designated historic districts, national parks, cemeteries, monuments, and landmark properties as well as individual bridges, roads, subways, tunnels, or paths significant in themselves as a function of design, location, age, materials, style, technology, and use.

State agencies include:

- *State Preservation Officer*: each state has a State Preservation Officer, who is responsible for review, evaluation, and recommendation regarding activity associated with properties having existing or potential historic resources.

■ *Administration and Planning:* administrative responsibility for the planning, programming and budgeting of activities related to historic properties and archeological surveys may be housed exclusively within one administrative or operating unit of state government. Or, a range of services associated with such considerations may be represented by individuals within several different departments charged with select functional duties, such as research, documentation, project development, land-use, budget formulation or administrative review of applications. Each state also has planning divisions and staff responsible for issues of preservation and archeology in context with zoning, master planning, and environmental assessment. To decipher which agency best addresses a particular development concern, start with the State Preservation Officer.

*County and Local Governments:* Responsibility at the local municipal and county level may be extremely varied, depending on the particular locale and the size of the governmental entity. Generally, control over historic and archeological resources is accommodated within the planning functions governing land-use controls. The professional planning staff may be complemented by appointed history or architectural commissions, or historic boards, who act in formulating local policies for adoption by elected officials. Architectural and historic review boards may also be appointed and serve to review individual land development proposals for conformance with local standards and criterion.

### The Tools: Public Policies

Every piece of property is subject to public review, whether under the guise of taxation, zoning or land use. Public review pivots off public policies, enacted or directed by elected officials at all government levels. Such policies are the working tools of public control over land development. Federal and state regulations play both an indirect role by underscoring the intent and direction of local enactments and a direct role in cases involving the use of federal funds, environmental assessment, or site proximity to a national landmark and/or district. A synopsis of those tools likely to be experienced during the planning and development of land that pertain to preservation resources follows.

**Master Plans:** Master Plans are public documents that forecast and designate appropriate land-uses, areas for conservation, and often places of historic significance over a time span, typically five, seven, ten or twenty years.

**Tax Policies:** Taxation policies usually address value assessments, rate credits or penalties designed to protect or promote a particular type of resource.

**Zoning:** Founded on the principle of directing land changes over time, zoning often works in conflict with preservation, as a resource once identified may introduce a land-use incompatible with the existing zone. Changing a land-use typically requires zoning review and public ap-

proval. During this process the issues of preservation and site resources are commonly broached.

**Permits:** Permits are issued for demolition, remodeling or new construction. Permits frequently flag preservation resources and issues, and immediate attention is drawn to a particular land tract where existing improvements, based on the age or location of an improvement, merit further study.

**Tax Assessments:** Every local jurisdiction assesses property to define land values relative to taxes due. Tax assessors field-inspect properties in support of land-value definition. Those inspections often identify potential resources for file notation. Such documentation aids in the public review and identification process.

**Land Use Planning:** Two sides typically include:

■ *Environmental Assessment:* the process of study to determine if land use or prospective changes will affect existing physical and natural environs of a locale. Part of this process specifically includes the examination of archeological and historic resources.

■ *Environmental Impact:* the location of a building or site adjacent to or nearby a historic site, landmark, conservation district, or historic district frequently warrants preservation assessment as a function of potential negative impact.

### The Public Process: A Strategy for Working Together

Public involvement and participation within the land development and historic resource processes have been institutionalized in the United States. They include legislated laws mandating that certain actions take place often within specific time periods. Select financial and tax incentives, as well as penalties and fees, assist in the enforcement and cost of such activities. Moreover, the power of eminent domain fosters the government's right to "take" property for a public good for "fair" and "just" compensation. All are examples of how participation is enacted.

The public process also includes provisions for community involvement. It is the government's responsibility to provide avenues for community participation, and it is prudent to expect (and foolhardy to think otherwise) that the land development process will involve a high degree of community involvement. It is precisely this mechanism that provides individual citizens and other quasi-public entities with an entrance into the public arena.

Neighborhood citizen groups, homeowners' associations, tenant organizations, historical societies, and others can, through the government review process, find opportunities to communicate concerns. Often, such concerns can serve to completely change the direction of a development program. Perhaps the most prudent advice lies in trying to incorporate community concerns and participation within the land development process at the earliest possible date.

Regardless of the simple or complex government system, the increasing use of federal monies to fund local infrastruc-

ture and community improvements has prompted the emergence of professional and community mechanisms at all levels of government to insure compliance with federal mandates regarding the safeguard of historic and archeological resources. Mindful of this mission, land development professionals should initiate preliminary research on a property when the occurrence of historic or archeological resources is remotely anticipated. Start with the local government level. Open informal dialogue with local authorities; however, enter into this dialogue prepared and informed of the review and assessment process and of the property in question. Land development professionals should be prepared in most cases to discuss the following information items:

- Is the property of known importance to the heritage of our nation, state or local region?
- Were there important historic events, people, etc. associated with the property?
- Are there any physiographic features that warrant further site study (cemeteries, land depressions, mounds, caves, swamps, etc.)?
- Is the land improved with structures? Do those buildings 50 years or older have distinctive architectural styles, building materials or structural features?

The designated local government representative can generally offer guidance as to the likelihood that noted resources or site observations might require further investigation. If a resource is identified, guidance should be solicited as to the extent of action that is necessary for compliance with local concerns and the extent of varied government involvement. Finally, it is always prudent to study the availability of public assistance with research, funding, historic nomination application, archeological site studies, and community participation that may assist in defraying costs occasioned by the unique circumstances of a property.

Local government historic and archeological personnel can be extremely helpful. While consultants can be retained to research, document, prepare assessments, file historic designation nominations, conduct building and site surveys, draft architectural plans, secure soil samples, excavate, and photograph, often government staff are willing to assist with some of these tasks. In many cases, part or all of the research needed for a particular property may exist on-file within a public agency and will facilitate the work effort. The key is to ask!

### **ENVIRONMENTAL SITE ASSESSMENT AND HISTORIC AND ARCHAEOLOGICAL PRESERVATION**

Briefly discussed earlier under Phase I-Phase II: Environmental Site Assessment, the point is clearly made that “Environmental Site Assessment is an integral part of site feasibility and is critical in minimizing a client’s environ-

mental liability exposure.” Most properties undergo varying levels of environmental assessment.

This is occasioned for a variety of reasons, including project financing, site proximity to an existing historic property or district, conformance with rezoning or permit requirements, compliance with tax credit usage, and/or containment of a potential resource. Environmental assessment is an increasingly common part of the development sequence and serves as an added safeguard against otherwise unforeseen circumstances that could have adverse financial, scheduling and/or legal consequences to a project.

Early assessment should seek to uncover sufficient property information to determine if elements such as waste disposal pits, dumps, ruins, endangered vegetation or physical elements exist to warrant further resource investigation. In each instance, the focus of early assessment should have two objectives: (1) to identify any unique property characteristics that may render a site unsuitable or unsafe for the intended use; and (2) to identify potential historic and archeological resources. To augment this assessment, the following information sources and data should be examined:

#### **Property Data**

Property settlement and deed records should be researched for site plats or legal descriptions that reference boundaries to specific, physical site features. Older property surveys and deeds in particular may reference buildings, fences, trees, rocks, rock formations, and physical markers to establish a metes and bounds description. Many of the documents noted can be obtained in public land records. Additional information sources for property research can be found in the United States Geographic Survey’s (USGS) topographic maps, aerial photographs, and property tax assessment records.

#### **Initial Field Survey**

The site should be investigated in the field and ideally subjected to a rigorous walking survey. Knowing the land and paying attention to physical forms such as mounds or depressions may give an early indication that the presence of archeological artifacts or subsurface conditions warranting investigation exist. Again, public tax records often note field findings, and land surveyors can be retained at a low up-front cost to field-inspect a property with a more trained eye. Recall Phase II-Field Observations in the preceding section.

#### **Corroborative Research**

The property data and field survey should be reviewed by an individual with experience in architecture and landscape cultural resources to assess the importance of on-site land forms and improvements. Individual structures need to be evaluated in terms of their architecture and an appropriate environmental setting. The same holds true for land forms and material remains. Increasing attention is being placed



on the issue of environmental setting. Buildings and site features exist within a spatial and location context; often it is the context—the environmental setting—that determines whether a property will undergo historic review.

### Revisit the Deed Records and Start Documentation

If previous efforts uncover potential on-site resources, then initiate a more detailed research and documentation process. Revisit the land records and search back through the property deeds, as they offer the most accurate source of information regarding dates of ownership change, persons involved, buildings, and site modifications. Land records will generally reference site improvements, roadways, paths, orchards, caves, water bodies, or similar features. Tracing the deed of ownership may clarify issues of building dates, cemeteries, landforms, and the presence of important persons or events. Reference personnel in any land records office can be extremely helpful. Compile all the deed, site, and interpretative research findings that are germane to fulfilling requirements established by public standards for a particular jurisdiction or locale, a funding source, or client discretion. The investigation should not lose sight of its purpose.

### Former Owner Use

Last, make sure that former land uses and ownership of the property under development are defined. Surprises do occur. For example, fire insurance records may uncover improvement conditions or events important to an owner's use of the property.

## LEGAL ISSUES PERTAINING TO PRESERVATION

### Questions and Information to Gather

There is abundant legal precedent affording protection to historic and archeological resources. The law clearly sets forth limits regarding the disturbance of land and the improvements thereon. Land ownership and development carry the responsibility of being knowledgeable of such limits, especially with regard to the demolition of site improvements, the disturbance of cemetery plots, and the excavation of earth. While it is often difficult for federal, state and local administrators to police every resource, penalties, fees and criminal charges can result where deliberate action has resulted in the loss of property or artifacts that may be of historic or archeological importance.

It is prudent to secure legal advice at the earliest stage of land acquisition and program planning efforts to gain a clear understanding of the rights and responsibilities affiliated with a development program. Advice pertinent to preservation issues within the endeavor should include:

1. Ask that title reports and deeds searched for land acquisition be reproduced for file records and that appropriate notations be made regarding ownership and property improvements over time.

2. Request legal interpretation of references and notations on deed copies provided.

3. In the event the deeds reference the presence of improvements, relics, or site activities implying possible historic or archeological resources or subsurface conditions (former waste or burial), ask for a definition of such ramifications.

4. Lastly, maintain communication. Most attorneys involved in land development have prior experiences in the legal implications of historic resources and can offer guidance based on the outcome of similar or related endeavors. Attempt to benefit from the experiences of others.

## ENGINEERING—ARCHITECTURE—LANDSCAPE ARCHITECTURE: CONTRIBUTING TECHNICAL SERVICES

The technical services of engineering and design professionals play an integral role in the preservation and archeological process. Such services include: the preparation of site surveys, assessment of sub-surface conditions, the evaluation of the structural integrity of building and material remains, and land planning. Initiating the process of historic and archeological identification with a site survey that includes the following information is of immeasurable assistance:

1. Instruct the survey crew to identify all site features remotely perceived as having potential historic or archeological significance.
2. Ask for field notations of material changes, vegetation, water, pits, fences, rocks, and similar elements, all of which could aid in the future design studies and findings for environmental study.
3. Ask for "footprint" measurements of on-site improvements, including buildings, walls, wells, and ruins.
4. Take pictures of existing conditions.

Architecture and landscape architecture, as professions, have a long record in producing products of noted preservation value. Of current and increasing importance are the products of engineering. Rising attention is being paid to engineered accomplishments as a new, evolving repertoire of historic resources within the American landscape. The plethora of highways, tunnels, bridges, subways, and water dam construction, often affiliated with buildings and distinct landscaped settings, and most notably built more than 50 years ago, are often subject to major upgrading and reconstruction. The age and location of many engineered structures now raises question of their historic and archeological resource value.

Many of these engineered feats occur in or adjacent to older buildings and historic districts, and this heightens the

importance of their preservation. Two case examples offer evidence of resource potential and assessment challenges.

The Rip Van Winkle Bridge Rehabilitation project in Catskill, New York required the engineering redesign and reconstruction of a structure crossing the historic Hudson River Valley. (See Figure 18.4.) It also called for the rehabilitation of an older Dutch Colonial highway administration building and toll facilities. The bridge and building were over 50 years old. This coupled with the site location within a historic region lead to design and landscape studies of a preservation nature.

A second example involves the ongoing inspection, evaluation, and rehabilitation of 60 bridges in the State of Connecticut. Most of the bridges date between 30 to 75 years old; most connect places of historic significance, and many were either designed by or influenced by the works of Frederick Law Olmsted and his firm. Olmsted, by act of Congress, is reputed to be the father of landscape architecture in this nation. Central Park in New York City is perhaps Olmsted's most renowned work, yet of quiet and equal historic importance are many of those streets and bridges he helped orchestrate during 1857–1883. These were his years of active practice while living in the city before moving his office to Brookline, Massachusetts.

Riverside Drive, a primary transportation link that ultimately leads from Manhattan to the Merritt Parkway in Connecticut, was built with design influence from Frederick Law Olmsted as documented in six sets of plans (dated from 1875) on file at the National Olmsted Historic Archives in Brookline. The importance of this drive, and the maintenance and reconstruction of the viaduct (see Figure 18.5) cannot be over emphasized, as the highway overall extends past New Haven, Connecticut under various highway names and includes historic consideration of over 60 bridges.<sup>1</sup>



**FIGURE 18.4** Rip Van Winkle Bridge.

<sup>1</sup> "Landscape into Cityscape: Frederick Law Olmsted's Plan for a Greater New York City". Pg 343–347. By Albert Fein. Van Nostrand-Reinhold Co; NYC-1967. "Civilizing American Cities": A Selection of Frederick Law Olmsted's writings on city landscapes. Pg 43–52. By S.B. Sutton. MIT Press; Cambridge, Massachusetts-1971. "The Columbia Historical Portrait of New York". Pg 295–296, 303, 419. By Columbia University and John Kouwenhoven. Doubleday and Company; NYC-1953.



**FIGURE 18.5** Riverside Drive Viaduct (124th Street southwest, circa 1927).

By virtue of Olmsted's participation in the original design of the parkway, the structures and related settings have also occasioned archeological interest as many of the bridges traverse water bodies along which colonists settled during the 15th and 16th centuries. The importance of this example is two-fold. First, the aging infrastructure of cities (particularly roads, bridges, and dams) points to a tremendous number of future historic resources, or at minimum to a deluge of historic evaluations and documentation with regard to the reconstruction of engineered structures important for modern living, yet clearly representative of past technical accomplishments. The second point to note is that private development plays an ever-increasing role in communal infrastructure and services. Whether as part of a large, planned community, or as part of a zoning proffer commitment, the increasing number of structures privately built for general public use has grown steadily since Olmsted's first subdivision for private development, Riverside, near Chicago, in 1858, and the enactment of zoning during the 20th century. Land developers, contractors, engineers, and designers should be aware of this growing preservation repository and the implications it has for all future private land development efforts.

## HISTORIC PRESERVATION AND ARCHEOLOGY DOCUMENTATION

### The Process of Recordation

Documentation is a process that involves the written, drawn, and photographic record of a particular place, property, building or relic. It is a time-consuming, detailed, fact-gathering process, which serves to clarify cultural, material, architectural, landscape and physical characteristics of a site, district, person, or location. The documentation process started with specific building studies prepared for the Historic American Buildings Survey under the management of the U.S. Department of the Interior during the 1930s.

Through the years the required contents of official information needed to compile a thorough description of a place, artifact, or building have been revised by the Secretary of the Interior. State and local municipalities may reinterpret this process and vary requirements to meet area needs, but regardless of the variations, the documentation process will conform within guidelines established by the Department of the Interior.

Property documentation can serve two purposes. First, recognizing that every building and artifact of our past will not be preserved, recordation of historic resources provides a paper archive for future generations. Second, documentation provides information beneficial for the preparation of an environmental assessment study, or a historic nomination application for designation on local, state, or the Na-

tional Register of Historic Places on a particular land parcel or improvement.

Many locations afford abundant quantities of aging buildings. Municipalities recognize that preservation of each building to its original state is often not realistic in light of contemporary safety requirements, mechanical systems, use needs, and economics. In such instances the use of documentation to record a building prior to rehabilitation or demolition serves to provide historic and architectural information important to the heritage of the area. While every building is not historically significant, rapid urbanization and suburban growth in many jurisdictions has facilitated the need to have select vestiges documented for posterity.

Similarly, local governments may require sites of suspected archeological significance to be inspected, and often

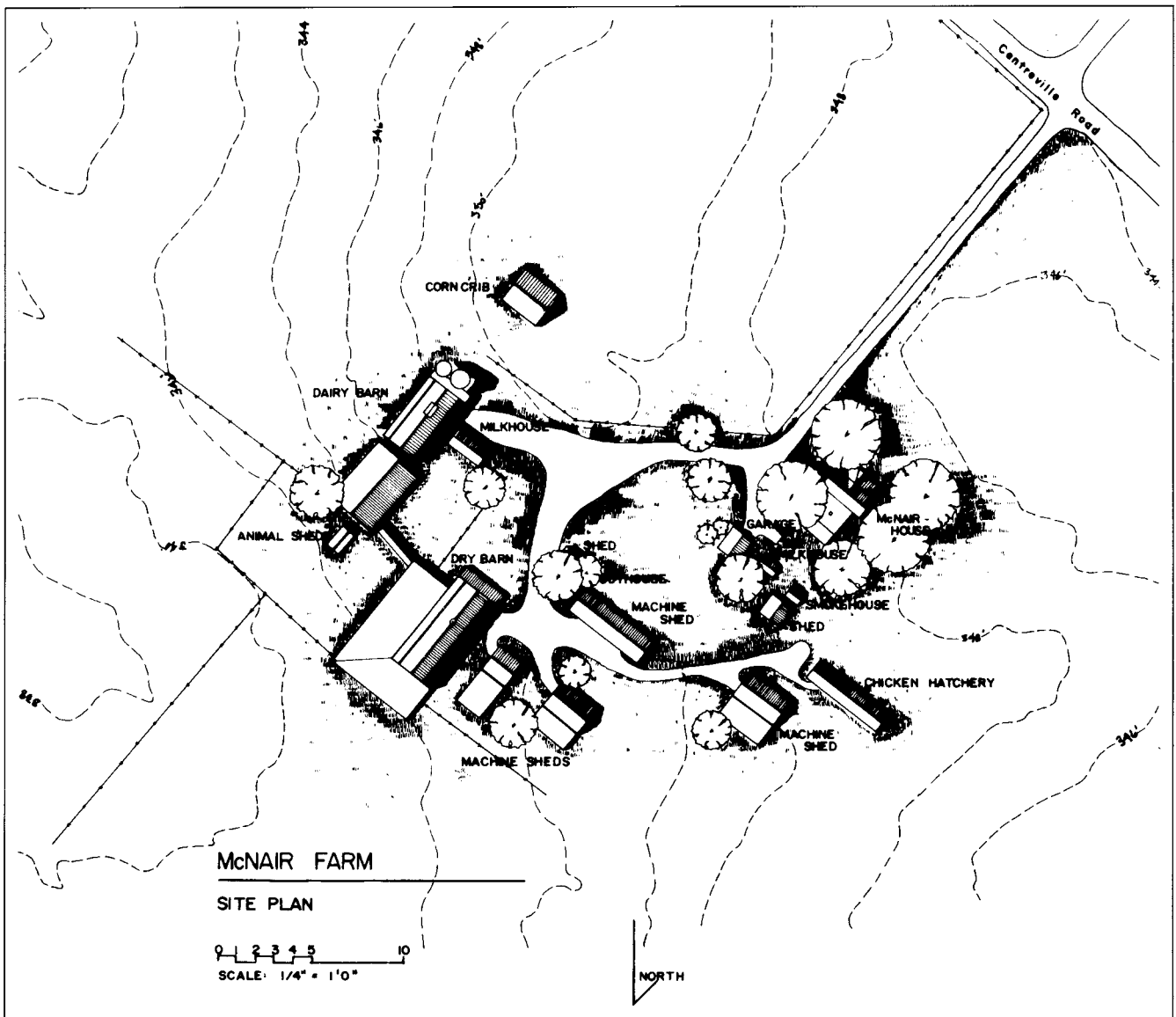


FIGURE 18.6 McNair Farms.

with some sub-surface soil sampling, prior to any site disturbance. Each of these activities, and the preparation of suitable report documentation of these concerns, is an increasingly common element to land development. The guidelines, procedures and specific standards that should be employed in a documentation effort can be obtained through the Historic American Buildings Survey, the U.S. Department of the Interior, in Washington, D.C.

Following is a list of topics typically included in the documentation process as set forth by the U.S. Department of the Interior.

- *Legal Description of Subject Property:* the property should be defined by its' deed number and tax plat or recordation numbers. It should have a street address and be referenced within a specific county and state. Maps should be included; in particular, the U.S. Geological Survey (USGS) map is recommended.
- *Historic Profile:* the history of the property, events and people associated with its "life," should be documented. This description should include an historic profile of names, potential adjustments in property boundaries, improvements, events, social trends, and site appearances.
- *The Site:* a site plan, or plans will be required. The plan should be defined by an engineering survey with an accurate description of the land area, its boundaries, fence lines, and physical features like vegetation, waterways, and paths. The site plan should focus on the historic character of the property at the time period prompting its historic significance. A second site plan may be required to record contemporary conditions. Notations and/or attached plans should define topography, vegetation, soil, rock, water, and other visual or materials qualities, all of which afford the site its character. Aerial photographs are encouraged.
- *Archeological Resources:* two levels of study may be required with regard to the documentation of archeological resources. First, the site should be inspected visually by a trained archeologist to determine if any potential elements exist prompting further study. Soil samples may need to be taken, or partial excavations of specific site locations may be desirable. In cases where significant historic information may be gained by a cordoned-off, full excavation investigation of a determined or perceived archeological resource, the archeologists will generally try to accommodate the land development schedule, as the process of study is time-consuming and highly detailed.
- *Photographic Documentation:* photo documentation should include all site improvements, not just buildings. Elements like bridges, dams, tunnels, ruins, machinery, partial buildings, power lines, railway lines, roads, paths, wells, and dumps, to note a few, are all

are considered physical improvements. Age is a determining factor. If it looks old, include it in the photographic documentation. Four kinds of photography are needed to document a property. These include:

1. *Contact Sheets.* Initiate the photographic documentation process with a roll of black and white print film processed into a contact sheet. All of the photographs grouped together help to determine which elements should be subjected to additional photo documentation.
  2. *Standard black and white prints,* preferably 4 in by 6 in to show a general view of property improvements and the setting. Two exterior (outside) elevations (side views) of improvements and, at minimum, two interior (inside) views are necessary. Attention should be given to capturing the details of material use, spatial change, building additions, roof and foundation materials, and the method of construction. Each photo should be mounted and labeled with a description of the image, view or detail, together with the project name, location and date.
  3. *Enlarged black and white prints,* 5 in by 7 in are adequate. These prints are required to show site improvements and features of determined historic significance. Generally, two exterior elevations, two perspective site views, one general (distant) view, four interior detail, and one exterior detail photograph are required for each improvement. Each photograph should be mounted and labeled.
  4. *Color slides* should be secured for the same subject material required in black and white prints. The slides should also be used to expand the opportunity to convey the site's environmental setting. Each slide should be labeled and marked in plastic sheet holders in numerical sequential order with the property name, location, and subject matter.
- *Architectural Description:* all physical improvements on the subject site should be described with an appropriate narrative supplemented with graphics and plans, as necessary. The architectural documentation is the 'body' of the process, and calls for written and drawn interpretation of property improvements. A professional trained in architecture should be involved in this process.
    1. *Style Descriptions.* The architectural narrative should focus on the definition of each building style, size, materials of construction, methodology, detail characteristics, exterior features, interior spaces, the roof, the foundation, and the site setting.
    2. *Field Notes.* On-site inspection is required with a field notebook where sketches of improvements

and land features can be drawn. The importance of this requirement is that the field notes, in cases of detail and accuracy, may fulfill parts of the following requirement to provide measured drawings.

3. *Measured-Scale Drawings.* Existing improvements determined to be of historic significance must be measured in the field with scale drawings prepared for select buildings. Two kinds of drawings are likely: pencil and ink. The drawings should include two exterior elevations and floor plans. In cases where an improvement is likely to be filed with the Historic American Buildings Survey, compliance with specific requirements calling for ink on mylar film and Leroy letter is necessary. The Department of the Interior will provide the mylar sheets. These drawings are prepared at specific scales and must contain specific title labeling.

4. Added requirements and variation in the presentation of information listed above exists with the use of computers and computer drawings. Documentation of any type should be reviewed first with the agency; a simple telephone call will suffice.

One case example involving documentation is offered to illustrate how a local jurisdiction handled the demolition and redevelopment of a former dairy farm property. In response to a developer's request for the rezoning of two tracts of 203 acres and 10 acres respectively, Fairfax County, Virginia required that the improvements associated with both parcels be documented in a manner consistent with the U.S. Department of the Interior standards. Early in the project's planning stages, the County determined that despite the property's ineligibility for state and national registers, the improvements were among the few remaining vestiges of the County's once abundant agricultural industry and, therefore, of historic importance. The County stipulated that both land tracts, proposed for development as a planned residential community with supporting commercial entities under the former name of the principal land-use—the McNair dairy farm, renamed McNair Farms, be documented for historic records.

At the time of original developer purchase, the McNair Farm (shown in Figure 18.6) was one of four remaining dairy farms in the County. By September, 1988, when land planning for the project was well underway, there was only one remaining 471-acre dairy farm in Fairfax County. The rapid urbanization of Fairfax County can be clearly understood in context with agriculture. In 1969, there were 218 farms, including 200 dairy farms, with an average size of 130 to 146 acres in Fairfax County. While a hard way of life, dairy farming was economically successful and the backbone of the County's agricultural heritage.

Fairfax County's urban growth can be measured in population changes, from 25,000 residents in 1930, to 40,929

in 1960, to 455,021 in 1970, and ultimately to a current density exceeding 1,000,000. In less than 30 years, the peaceful rural character of the County transformed into a thriving urban extension of Washington, D.C. The pace of land activity and the real and perceived loss of the County's agricultural heritage prompted public requirements that the McNair Farm and the adjacent Hoffman property be documented.

A site survey was conducted to record the location and dimension of all improvements and land forms. This included 23 structures, extensive fence lines, two ponds, and several groves. Land records were searched, a historic profile was drafted, and former landowners and neighbors were interviewed. Site artifacts were identified and photographed. Measured drawings were prepared for seven buildings, and field note sketches were drawn for all improvements. Four site plans were generated based on the engineering survey. The U.S. Department of the Interior assisted in providing the mylar sheets and technical assistance. Photographs, slides, field notes, and drawings were prepared with the assistance of students and part-time employees.

The outcome of this documentation effort proved beneficial in several ways. Not only did the information fulfill requirements set forth by the County, but it augmented site design, farmhouse relocation and re-use planning, and business relations. The process brought people together, opened communication, and broadened the base of project information for future environmental and funding requirements. The cost of the study was less than 1% of the planning and design fees for the development project.

### Architectural and Landscape Preservation Design Examples

The opportunities to use information generated from the investigation of preservation and archeological resources in a manner beneficial to land design are diverse. Two case studies are cited to emphasize the contribution such information can make to design development and project marketing. In each case, the identification of unique land features, artifacts, archeological or historic resources helped shape the land development project design response. Historic resources became a component of the larger development program or, by virtue of association, became incorporated into a unique marketing tool, like name recognition.

An example is the planning and design of Franklin Farm (see Figure 18.7), a planned residential and commercial community in western Fairfax County, Virginia. This 828-acre project sought to perpetuate the agricultural farm imagery long associated with the property to distinguish it from the surrounding suburban housing developments. Site design and landscape elements incorporated rural agricultural imagery with the retention of three farm buildings, ponds as part of storm water management, and tree groves



**FIGURE 18.7** Aerial View, Franklin Farm with silo and barn evident (Photo courtesy of Larry Olsen).

as visual buffers between residential homes. Fencing and signage details were also designed to augment the imagery.

More strikingly, community building and retail elements were designed with finish materials to evoke an imagery associated with agricultural vernacular architecture. The development program for Franklin Farm included 1,300 single-family residences, 330 townhomes, several community recreation centers, parks, and a shopping center, all of which was not dissimilar to other planned residential communities of comparable size located nearby.

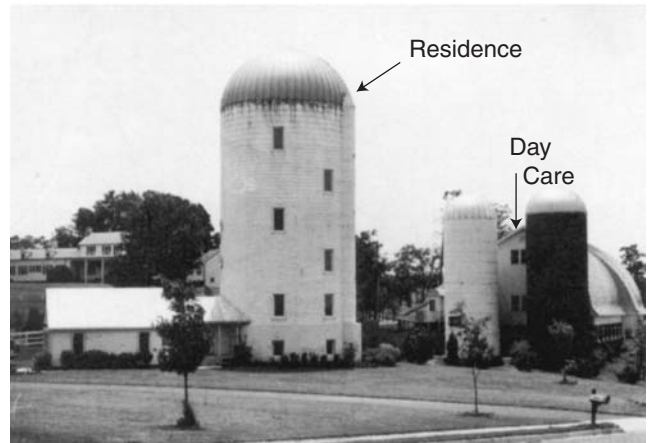
Two former farm buildings, the dairy barn and milkhouse, sited at the entrance of the large community, were rehabilitated for contemporary use as a daycare center. The silo (shown in Figure 18.8) was converted to a private residence. Each required extensive study, design, and public review. Each offered distinct challenges in terms of design requirements and permit processing. Each stands today as a landmark image within the community of Franklin Farm and a tribute to the agricultural heritage of Fairfax County. The silo was converted into a five-level residence of 3,100 ft<sup>2</sup>. It has received numerous awards for architectural design, preservation and innovative use of building materials.

The rehabilitation of the dairy barn and milkhouse as a daycare center was an equally challenging tribute to architectural adaptive reuse. Given the stringent safety and health requirements associated with its principal occupants—children—and the unique requirements for recreation, administrative, handicap, and educational facilities, the 6,000-ft<sup>2</sup> center provides a high-profile community resource distinguished by numerous awards.

Together, these two structures, the community name, and the site development theme are a continual reminder to the greater community that there is a unique history to the site. That history is perpetuated in a contemporary land development response. The 75-ft silo is a visual landmark, symbolizing joint public and private efforts to retain images of and appreciation for the agricultural heritage of the County.



Before

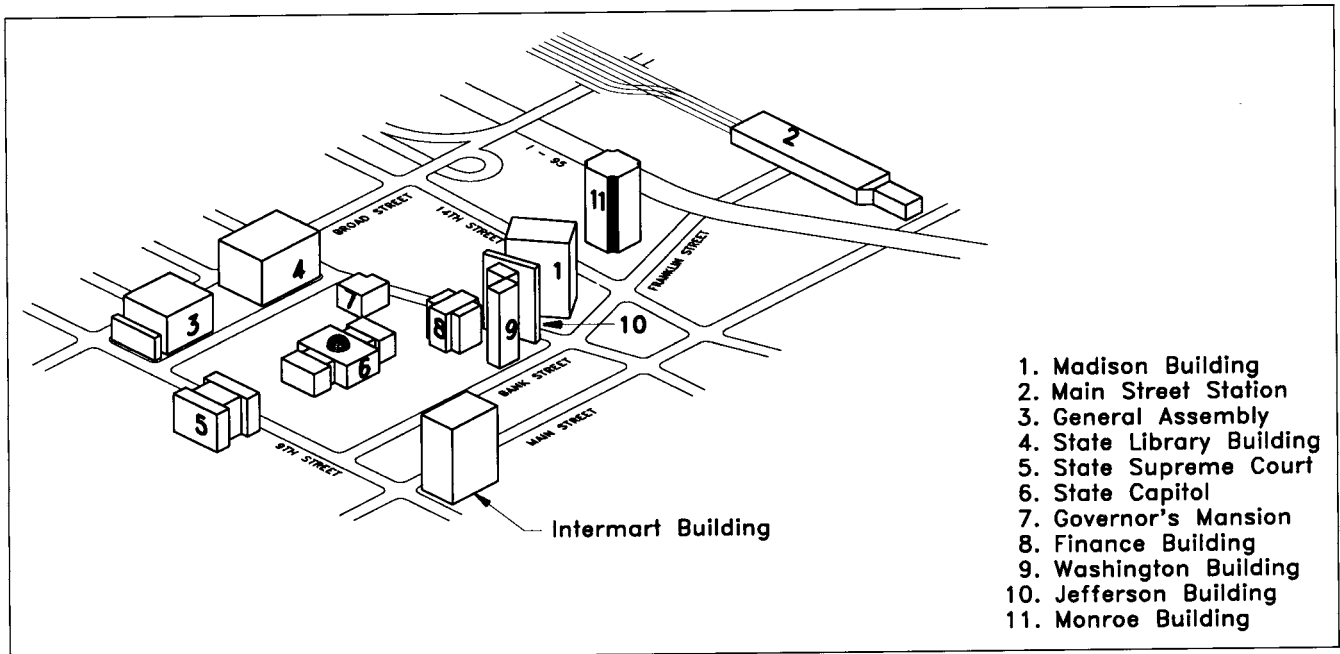


**FIGURE 18.8** Before and after view of silo with dairy barn (photo courtesy of Joel Grimes).

The second example, the historic assessment required to rehabilitate the 1960s international-style Intermart Building in Richmond, Virginia in 1988 exemplifies how site proximity near a national landmark alters the development process. (See Figure 18.9.) The Intermart Building faces the historic grounds and buildings of the Virginia state capital. As a result, all changes to the 25-story, 252,890-ft<sup>2</sup> high-rise underwent historic review based on the potential of a visual impact to a national landmark. The 11 million dollar rehabilitation project won numerous awards for the innovative use of materials and streetscape pedestrian design solutions.

## CONCLUSIONS

The role of historic preservation and archeological resource assessment within the land development process is continually growing and expanding. The traditional concerns of a property being historic in terms of age and national importance have branched into a wider arena where contemporary issues of culture, heritage, community, environment, material use, and archeology are of significant importance. Most land development endeavors recognize and plan for inclusion of these concerns. The emphasis of this presen-



**FIGURE 18.9** Location of Intermart Building to Virginia State Capitol Historic property.

tation has been to increase the level of understanding of and appreciation for the role of historic preservation and archeological as a function of land change.

As development interests understand their increasing role as a part of preservation, the preparation and the formulation of strong advocacy plans toward the realization of land changes will take place with greater ease. Such increased awareness will open opportunities for reasonable public and community participation with fewer gridlocks. New insight will also be gained into the options of rehabilitation, documentation, relocation, or excavation of a particular resource if identified on a property. Financial opportunities with creative avenues suited to saving all or part of a resource, or studies undertaken in support of resource identification and environmental assessment, will become a routine part of development planning.

Last, preservation resources when discovered should challenge development and design professionals to a higher level of design, marketing and financial solutions. While issues regarding the "fair and reasonable" costs associated with preservation activity, as well as the perception of abridged rights and control over private property, continue to be in contention, ask yourself: what are the time and financial costs of neglecting history?

## REFERENCES

Couture, Elizabeth, for the Batman Company and U.S. Department of the Interior, McNair Farms. Herndon, Virginia, 1988.  
 Darr, Lonnie, *A Technical Manual for Woodland Conservation with Development in Prince Georges County*, Maryland-National Capital Park and Planning Commission, Upper Marlboro, Md, 1990.

DeChiara, Joseph, and Lee E. Koppelman, *Time-Saver Standards For Site Planning*, McGraw-Hill, New York, 1984.

Fairfax County, Va., *Comprehensive Plan: Fairfax County, Virginia Area III*, 1986.

Fairfax County, Va., *1987 Standard Report*, 1987.

Fitch, James Marston, *Historic Preservation*, McGraw-Hill, New York, 1982.

Gillette, Jane Brown, *Historic Preservation Magazine*, "Elvis Lives," May 1992.

Macie, Edward A., and Gary Moll, *Shading Our Cities: A Resource Guide for Urban and Community Forests*, Gary Moll, and Sara Ebenreck, eds., American Forestry Association, Island Press, Washington, D.C., 1989.

Marois, Denise, Farming: "A Way of Life under Seige," *The Gazette*, Vienna, Va., November 10, 1988.

Miller, Robert W., *Urban Forestry: Planning and Managing Urban Greenspaces*, Prentice Hall: Englewood Cliffs, New Jersey, 1988.

Morris, Marry, *Innovative Tools for Historic Preservation*, Planning Advisory Service, Report #438, 1992.

Moll, Gary and Sara Ebenreck, eds., *Shading Our Cities, A Resource Guide for Urban and Community Forests*, American Forestry Association, Island Press, 1989.

Moloney, Noral, *Archaeology*, Oxford University Press, England and New York, 1996.

The Mount Vernon Ladies Association of the Union, *Mount Vernon: A Handbook*, Mount Vernon, Va., 1985.

National Association of Home Builders of the United States, *Land Development*, National Association of Home Builders of the United States, Washington, D.C., 1987.

*Natural Resources Defense Council Inc. v. EPA*, Nos. 90-70671 and 91-70200, slip op. at 6217 (9th Cir.), June 4, 1992.

Netherton, Ross, and Nan Netherton, *Fairfax County in Virginia*, Donning Company, Norfolk, Va., 1986.

Robinette, Gary, *Local Landscape Ordinances*, Agora Communications: Plano, Texas, 1992.

Smardon, Richard C., James P. Karp, *The Legal Landscape*, Van Nostrand Reinhold: New York, New York, 1993.

Travers, Naomi S., "Last Fairfax Dairy Framer Holds Out for Love of the Land," *Washington Post*, Sept. 6, 1988.

U.S. Department of the Interior, National Park Service. National Register Bulletin: *How to Apply the National Register Criteria for Evaluation*, Publication #15, Washington, D.C.

United States Department of the Interior, National Park Service, *Guidelines for Preservation*, Washington, D.C.

Upton, Dell, and John Michael Vlach, *Common Places: Readings in American Vernacular Architecture*, University of Georgia Press, Athens, Ga., 1986.

Weinberg, Nathan, *Preservation in American Towns and Cities*, Westview Press, Boulder, Colorado, 1979.

Wienczek, Henry, *The Smithsonian Guide to Historic America*, Stewart, Tabori & Chang, New York, 1989.

Wood, Joseph S., "Shaping The Present: Geographical Essays on Historic Preservation Policy," *North American Culture, Society for the North American Cultural Survey*, Vol. 4 No. 2., 1988.





## CHAPTER 19

# ENVIRONMENTAL CONSIDERATIONS

*Jeff Moran, P.E. / Fred Eberle*

### INTRODUCTION

This chapter focuses on environmental considerations that may affect the purchase or development of a property. As described in the previous environmental chapters, environmental impacts may exist as a result of previous or current activities, development activity, and/or future use of the property. Environmental regulations exist to protect human health and the environment. The material presented in this chapter addresses the various types and sources of chemical contamination that may be encountered during an assessment of a property, the environmental requirements for the management and corrective action relating to underground storage tanks, environmental considerations for demolishing buildings, air quality issues, and noise.

### CHEMICAL CONTAMINATION

Chemical contamination impacts air, water (groundwater and surface water), and soils. This contamination is managed and regulated through a variety of environmental regulations, as described in the chapter entitled Environmental Regulations. Chemical contamination sources come in three major categories:

- Properties or facilities used for the direct disposal of materials
- Sites of indirect disposal of materials
- Non-point sources

Hazardous waste is likely to be the most serious chemical contamination on a parcel of land and is typically related to industrial activities. Hazardous waste refers to discarded waste materials from institutions, commercial es-

tablishments and residences that poses an unacceptable risk to human health and safety, property values, and the environment.

Before purchasing property for development, an environmental site assessment should be performed as described in Chapter 9. As part of these environmental assessments, a search into the history of the property is performed as well as an actual physical site survey and field reconnaissance. If the assessment identifies potential environmental concerns, environmental sampling (soil, groundwater, air, etc.) is usually recommended. If the environmental sampling identifies contamination it may need to be appropriately defined and remediated prior to development. These detailed studies can take several months to complete and cost as much or more than the property was initially valued. While the contributing sources of environmental contamination are highly variable, the processes for addressing the contaminant are fairly standardized. There are numerous written protocols for performing environmental site assessments. The protocols include checklists that document areas and items to consider. The list of potential contributing sources of chemical contamination of property is too lengthy to discuss in this chapter; however, several of the more common sources of contamination found today are presented in the following paragraphs.

### Chemical Contamination Sources

**Surface Impoundments.** Surface impoundments have been recognized as a major source of groundwater contamination. Before the enforcement of environmental regulations, generators of solid and hazardous wastes commonly disposed of the waste in surface impoundments. The waste would either evaporate, possibly leading to air pollution, or

percolate into the groundwater, thereby contaminating a potential source of drinking water.

**Landfills.** Municipal and private facilities used for burying wastes are also sources of groundwater contamination. Many materials received by landfills in the past are now recognized as hazardous materials. Construction of some older landfills, as well as the operation of these facilities, failed to provide even minimal protection from the potential migration of their contents. Typically, ordinary municipal garbage has been exempt from hazardous waste regulations, yet this waste inevitably contains amounts of hazardous materials discarded by households and industries. These wastes include paints, solvents, pesticides, cleaners, and other household products that contain hazardous constituents and pose a threat of subsurface pollution when large amounts are collected in a landfill. Additionally, liquid wastes may be disposed of directly to the landfill, or they may arise from chemical or biological transformations of landfill material. Often the leachate from the landfill percolates through the waste mass, picking up hazardous constituents along the way and eventually affecting the groundwater.

**Spills.** Accidental spills can release chemicals and create major problems. With the current attention being given to hazardous materials handling, spills from overturned trucks or ruptured drums are now managed with specific protocol. In the past, however, response was often limited and chemical spills were not properly mitigated.

**Tanks.** The wide use of underground storage tanks for fuel and waste storage throughout the United States has resulted in significant contamination to our nation's groundwater. Tank systems without the proper leak detection, spill and overflow protection, and corrosion protection are a continued threat to human safety and the environment. Since there is no comprehensive inventory for all the tank systems installed, the possibility exists that leaking tanks exist on "undeveloped properties."

**Septic Tanks.** Many households and industrial facilities have used septic tanks for on-site disposal of waste from wash areas and bathrooms. These systems are designed to hold the wastes in an underground tank, where sedimentation and anaerobic microbial activity remove some of the contaminants. The partially treated water is released through perforated pipes to a subsurface "leach field." The soils in the leach field filter the waste and provide an environment appropriate for further microbial degradation. The system is specifically intended to promote infiltration of water, which is the bulk of the waste. Septic tanks are successful and appropriate systems where population density is low, soil conditions are good, and the waste being treated is only water with biodegradable contaminants. However, public health problems occur where septic tanks are too numerous, the water table is too high, or subsurface channels carry leachate directly to a water supply aquifer. On many occasions, the transport of infectious organisms has caused epidemics, making septic tanks the number one

source of waterborne disease outbreaks in the United States. Septic tanks become a hazardous waste problem when materials other than ordinary sewage are put down the drains. Some household products are toxic. Tanks at industrial facilities may receive the usual spectrum of chemicals, including toxins that are not quickly biodegraded. Additionally, these toxins may be relatively mobile in soil and not subject to the processes that attenuate the concentrations of sewage organics. Because the septic tank system is specifically designed to promote subsurface migration of the water, the included hazardous materials may become a serious problem for groundwater quality.

**Agriculture.** The pesticides and fertilizers used in farming have contaminated many water supplies. Ethylene, dibromide, aldicarb, and others have become common contaminants in farming areas, and many wells have been contaminated with nitrates intended for plants. Fertilizers and pesticides are spread widely on the soil in large amounts. Irrigation water dissolves these and carries them downward as it infiltrates. Manufacturing, loading, and mixing sites serving agriculture have produced some highly localized point sources of contamination. Loading of planes for aerial application of pesticides, for example, has polluted some airport areas.

**Urban Runoff.** Modern cities are built on the assumptions that separate systems for wastewater and stormwater collections are best. In this design, sewage from homes and industries is collected in an enclosed system and treated, while the much larger amount of clean rain water is collected in an open system and channeled to rivers, lakes, or the ocean. Recent studies are casting some doubt on this assumption. Runoff from cities contains significant quantities of contaminants. Lead, discharged in automobile exhaust collects in dust on surfaces, may appear in high concentrations in the first flush of runoff at the beginning of the rainy season. Petroleum from spills and drippings in the street may also be present. Runoff water will stand on the soil for some period, and may infiltrate to contaminate groundwater.

**Deep Well Injection.** Hazardous waste is sometimes pumped into the ground. The wells used for this purpose are thousands of feet deep, and some effort has been made to establish the fact that there is no connection between the strata receiving the chemicals and shallower aquifers being used for water supply. In some cases, the connections have in fact been present, hidden by the complexity of the subsurface geology and the difficulty of making observations. The wastes may return to the shallow water supplies through fissures, abandoned wells with corroded castings, or sometimes through the annular space surrounding a poorly installed injection well.

**Illegal Dumping.** Probably the most common form of illicit disposal is the inclusion of hazardous materials in municipal garbage. If the amounts are small, it is relatively easy to conceal containers of chemicals in the solid waste going to the dump. Disposal of this kind is a significant factor in

the production of hazardous leachates by municipal landfills. Illegal dumping often produces large acute releases. Unethical individuals may wait for dark to deposit barrels of material in the countryside. These people rarely take care to see that the barrels are intact and closed so they can be picked up; more often they are tipped over and leaking. Smaller-scale, chronic dumping occurs. Homeowners or businesses may carelessly discard used motor oil or contaminated gasoline. It has been suggested that “every company has its back 40” where materials were dumped in the past.

### Composition of Hazardous Chemicals

Among the most obvious characteristics of matter is its physical state. While solids retain their shape and volume, liquids retain their volume but not their shape. As a result, liquids may flow through soils. Gases retain neither volume nor shape and flow readily. These characteristics are fundamental in determining the fate and transport of pollutants that enter the soil. Solids may be large objects, such as those often disposed of in municipal landfills. They may also be finely divided particulate matter like soils. Liquids may be solvents or petroleum products, or more commonly, water that is carrying dissolved pollutants. Gases are sometimes released directly to the soil, but they will more often be involved in subsurface migration of hazardous wastes because evaporation, microbiological activity, or chemical reactions generate them.

**Solids.** Solids have the unique characteristic that only their surfaces will participate in chemical reactions or support microbiological growth. Solids in either a buried waste mass or natural soils strongly influence the composition of water passing through their pores, since the solid itself may dissolve. Where the waste is contaminated, the solids may absorb and release pollutants. Because the specific surface area of the solid may be large, the total effects on water quality can also be large. An important characteristic of solids is their relative immobility. Since solid contaminants rarely move far from the site of disposal, the disposal of solid material, if it does not later dissolve, threatens groundwater quality less than the disposal of liquids.

**Liquids.** The movement of liquids is the most common kind of subsurface migration of hazardous wastes. Movement occurs in response to several forces. Gravity will cause liquids to move downward. If the soil is saturated, liquid waste will move with the slope of the water table. Since water has an affinity for dry soil, it will move from wet soil to dry soil, just as it soaks into a dry sponge set against a wet sponge. The most common kind of subsurface pollutant is a contaminant dissolved in water. Where it is not removed by interaction with the soil, the contaminant can pollute large volumes of soil and groundwater. A stream of contaminants moving away from a point source of pollution gradually broadens, just as a plume of smoke spreads after it leaves the chimney. Other groundwater problems arise from the presence of a pure liquid. Gasoline and other petroleum hydrocarbons are common examples. If they are

immiscible, they will form a layer of pollutant on top of the water table and move according to their own pressure gradient. A layer of pure hydrocarbons contains a large mass of pollutants, which can contaminate correspondingly large amounts of water as it moves and dissolves.

**Gases.** Gases move faster than water under comparable pressures. They respond readily to pressure and can be pushed outward from a point of generation. Since a volume of gas is much lighter than the same volume of water, it is less affected by gravity. It moves up, if it is less dense than air, or down if it is more dense. Molecular diffusion, from areas of high concentration toward areas of low concentration, may also occur. Methane, generated by biological activity, is commonly encountered in landfills that receive municipal waste or other decomposable organic matter. Houses near landfills have often been threatened by methane in basements or other enclosed areas. Accidents have even occurred at the exposed surfaces of landfills. In one case, children playing with matches ignited the gas and were burned. Methane migration also has the undesirable secondary effect of carrying other gases along with it. Vinyl chloride, which is a microbial breakdown product of the common solvent trichloroethylene, is often present in landfills. Transportation of this gas with methane is particularly alarming because it is a known human carcinogen (cancer causing). Methane has carried vinyl chloride into the basements of houses near landfills. Benzene, xylene, and other toxic compounds have also been detected. Water vapor also moves in the pores of soil and waste, often driven by temperature differences. A mass of organic matter undergoing microbial decomposition will warm considerably. Water evaporating from the mass will spread to cooler regions and condense.

**Hazard Classification.** Wastes deemed hazardous are regulated extensively. The Environmental Protection Agency (EPA) defines hazardous waste by its characteristic, process, and use. Hazardous waste characteristics are defined by their ignitability, corrosivity, reactivity, and toxicity. There are specific analytical methodologies for determining these characteristics. A waste is considered ignitable if it has a flashpoint less than 140°F or if it can ignite through friction, moisture, or spontaneous chemical changes. Wastes with a pH of less than 2.0 or above 12.5, or the ability to corrode steel (SAE 1020) at a rate greater than 0.250 inches per year at a test temperature of 130°F, are classified as corrosive. A waste that can explode, react violently with water, or can form potentially explosive mixtures or toxic gases, is classified as reactive. Since solid wastes may produce toxic leachates, proper waste characterization is critical prior to removing, transporting, and disposing of a waste. The EPA outlines specific test procedures to determine the amount of extractable hazardous materials in a waste. For example, the toxicity characteristic leaching procedure (TCLP) is a common analytical method for determining a waste's toxicity. When performing a phase II or phase III environmental site assessment, information about the type

of contaminant is crucial. If the source and nature of the material are known, the sampling and analysis are limited to the parameters of the known contaminants and regulatory requirements. Otherwise, a full spectrum of analyses may be required, consisting of analysis for hazardous substances.

### UNDERGROUND STORAGE TANKS

The regulation of underground storage tanks (USTs) in the United States has become a very critical issue and a matter of national concern. There are a great number of USTs in use and many have had undetected leaks of hazardous substances. More than one million underground storage tanks in the United States contain petroleum or hazardous substances regulated by EPA. Many of these tanks have leaked or are currently leaking. Given the contents of these tanks, releases from USTs can threaten human health and safety, since releases can contaminate soil and drinking water supplies. Leaking USTs continue to pose a serious threat to groundwater and drinking water in many parts of the country. Historically, tanks were placed underground to store materials that were hazardous because of their flammability or combustibility. Many tanks were installed underground on the premise that they would never develop leaks.

Until 1984, only state and local regulations covered USTs. In many cases, they only covered the installation, fire safety, and prevention of explosion hazards but did not consider groundwater protection. In November 1984, amendments to the Resource Conservation and Recovery Act (RCRA) directed the EPA to establish regulations for monitoring leaks and cleaning up contamination from existing tanks, and to establish standards for the construction and installation of new tanks. The amendments also required tank owners to notify various state agencies of the existence of their tanks. Several factors have combined highlighting the problem of leaking USTs before the public; programs now mandated by the EPA are resulting in more reported release incidents; the ability to detect hydrocarbons in the groundwater and soils has greatly increased over the last several years; and lending institutions, property owners, and developers are now realizing the extent of potential liability under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), or Superfund law. Since state or local regulations may differ from the federal requirements, it is important to determine which requirements apply to the USTs in the appropriate state. Figure 19.1 shows the geographical distribution of the number of reported USTs in the U.S. and its territories.

### Regulated Tanks

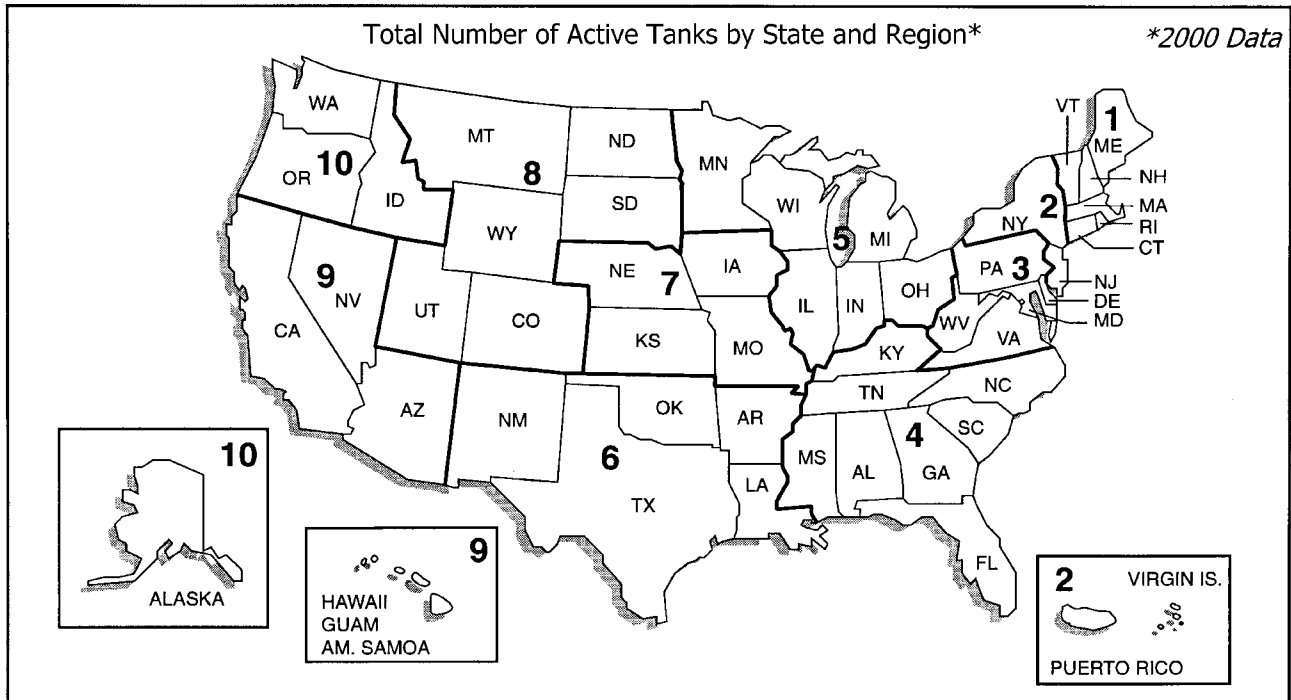
Since property that is purchased for development may contain an underground storage tank, the purchaser and developer should be aware of the regulatory requirements associated with them. RCRA defines USTs as “any one or a combination of tanks (including underground pipes connected thereto)” that contain a regulated substance and the

“volume of which (including the volume of the underground pipes connected thereto) is ten percent or more beneath the surface of the ground.” Excluded from the definition are:

- Farm or residential tanks with the capacity of 1100 gallons or less holding motor fuel used for noncommercial purposes
- Tanks storing heating oil used on the premises where it is stored
- Tanks on or above the floor of underground areas, such as basements or tunnels
- Septic tanks and systems for collecting storm water and wastewater
- Flow-through process tanks
- Emergency spill and overfill tanks

Regulated substances include hazardous chemical products regulated under CERCLA, and petroleum products, including crude oil and refined products that are liquid at standard conditions of temperature and pressure. All new tanks must have leak detection when they are installed. The EPA has identified the following methods that owners and operators may use to meet the federal leak detection requirements:

- *Secondary containment with interstitial monitoring:* Secondary containment often uses a barrier, an outer wall, a vault, or a liner around the UST or piping. Tanks can be equipped with inner bladders that provide secondary containment. Any product leaked from the inner tank (or piping) is directed towards an “interstitial” monitor located between the inner tank (or piping) and the outer barrier. Interstitial monitoring methods range from a simple dipstick to a continuous, automated vapor or liquid sensor permanently installed in the system.
- *Automatic tank gauging system:* A probe permanently installed in the tank is wired to a monitor to provide information on product level and temperature. These systems automatically calculate the changes in product volume that can indicate a leaking tank.
- *Vapor monitoring:* Vapor monitoring measures product “vapors” in the soil around the UST to check for a leak. This method requires installation of carefully placed monitoring wells. Vapor monitoring can be performed manually on a periodic basis or continuously using permanently installed equipment.
- *Groundwater monitoring:* Groundwater monitoring measures the presence of liquid product floating on the groundwater. This method requires installation of monitoring wells at strategic locations in the ground near the tank and along the piping runs. To discover if



<b>Region 1</b>		<b>Region 4</b>		<b>Region 7</b>	
Connecticut	13,339	Florida	33,574	Missouri	11,039
Massachusetts	12,122	North Carolina	32,012	Kansas	8,971
Maine	3,709	Georgia	29,959	Iowa	8,433
New Hampshire	3,061	Alabama	18,560	Nebraska	7,135
Vermont	2,926	Tennessee	17,219	<b>Total</b>	<b>35,578</b>
Rhode Island	1,795	Kentucky	14,797		
<b>Total</b>	<b>36,952</b>	South Carolina	12,727		
		Mississippi	9,533		
		<b>Total</b>	<b>168,381</b>		
<b>Region 2</b>		<b>Region 5</b>		<b>Region 8</b>	
New York	32,928	Ohio	29,037	Colorado	7,990
New Jersey	23,250	Illinois	26,615	Utah	4,181
Puerto Rico	4,637	Michigan	23,067	Montana	3,610
Virgin Islands	114	Wisconsin	17,262	South Dakota	3,292
<b>Total</b>	<b>60,929</b>	Minnesota	13,574	North Dakota	2,407
		Indiana	9,842	Wyoming	2,071
		<b>Total</b>	<b>119,397</b>	<b>Total</b>	<b>23,551</b>
<b>Region 3</b>		<b>Region 6</b>		<b>Region 9</b>	
Virginia	32,267	Texas	61,966	California	44,688
Pennsylvania	28,842	Louisiana	15,948	Arizona	8,613
Maryland	8,784	Oklahoma	12,067	Nevada	3,490
West Virginia	6,664	Arkansas	9,952	Hawaii	2,056
Delaware	1,744	New Mexico	4,293	Guam	577
District of Columbia	754	<b>Total</b>	<b>104,226</b>	N. Marianas Islands	76
<b>Total</b>	<b>79,055</b>			American Samoa	45
				<b>Total</b>	<b>59,545</b>
				<b>Region 10</b>	
				Washington	11,332
				Oregon	7,370
				Idaho	3,479
				Alaska	1,220
				<b>Total</b>	<b>23,401</b>

FIGURE 19.1 Geographical distribution of active underground storage tanks (USTs).

leaked product has reached the groundwater, these wells can be checked periodically by hand or continuously with permanently installed equipment.

- *Statistical inventory reconciliation:* In this method, a trained professional uses sophisticated computer software to conduct a statistical analysis of inventory, delivery, and dispensing data, which must be supplied regularly.

- *Other methods meeting performance standards:* Any technology can be used if it meets a performance standard of detecting a leak of 0.2 gallons per hour with a probability of detection of at least 95% and a probability of false alarm of no more than 5%. Regulatory authorities can approve another method if it can be demonstrated that it works as well as one of the methods above and complies with any condition the authority imposes.

### Identification/Detection

Determining the existence of USTs on an undeveloped tract of land can be a difficult task, since records of installation and use may not exist, as there was no requirement to notify state agencies of the existence of USTs. This is one of the reasons why phase I environmental site assessments are so important for properties planned for development. This assessment should be performed as early as possible in the development planning stage. Ideally it will be performed prior to property acquisition to ensure that any identified environmental concerns can be economically mitigated or controlled during property development. Interviews with past and present owners may reveal that USTs were installed on the property. Searches and reviews of local and state regulatory agency records may reveal that USTs were installed and the installation work was inspected by the local fire marshal or a leak was investigated by county or state environmental agencies. Historical research into prior land use may indicate that past operations such as farming, quarrying, or air strip landing that are suspect of potential UST use, occurred at the site. Finally, visual inspection of the property may identify situations or locations suspect of the presence of USTs. Visual evidence of the presence of USTs may include vent or fill pipes protruding from the ground; the presence of fill materials such as sand or gravel or other materials not indicative of native soils or geology; mounds of earth with no explanation of origin; evidence of staining; dead or dying vegetation; or hydrocarbon type odors.

With the lack of historical or visual evidence of UST existence, there are methods of determining the subsurface presence of metallic structures. Underground tanks can be found in various sizes, shapes, and construction materials. Metal tanks are usually of welded construction and some may have some type of exterior coating or protection against corrosion. Newer tanks may be fabricated of fiberglass, epoxy, or other nonmetallic materials. Depth of the tank burial varies; however, the top of the tank is usually

not more than one tank diameter below finished grade. Tanks can be found in mounds, partially buried below grade in some installations. If traffic travels over or above the tanks, they may be paved over. Manholes, caps, or other devices to cover or protect tanks and associated piping systems, such as fill connections or gauge pipes, may be present. Some large tanks may have a manhole to allow access to the tank.

### Releases/Mitigation

Corrosion is the most common cause of UST leakage for steel tanks and piping. Even as late as the 1970s it was common practice to bury unprotected steel tanks. Surprisingly, studies have indicated little correlation between the product stored in USTs and their corrosion. Yet, soil and groundwater conditions can speed the corrosion of USTs. In northern climates, road salt percolates into the soil and groundwater and accelerates the corrosion of USTs. Metal failure is frequently observed at connections and pipe fittings; although mechanical stresses are contributing factors, the most common cause of failure is corrosion from the contact of two dissimilar metals. Overfills, spills, and other accidental releases resulting from poor oil delivery transfer procedures are another major contributing factor to environmental contamination from UST operations. Large releases due to overfill, or small releases due to connect-disconnect spillage over a long period of time, can be equally serious.

Generally, contamination from USTs will leach into the vadose zone and become adsorbed onto clays or form a thin film on the larger grain-size fraction of soils. Often, if the spill is small, the contaminant will remain in the surrounding soils of the UST and remain above the groundwater table. Sometimes in very clayey soils the contamination will pool under the UST, captured by the clay layer. For large releases, the contamination may migrate to the zone lying immediately above the water table. Depending on the physical properties, such as density, viscosity, and water solubility, the contaminant may sink to the bottom of the groundwater aquifer or float on top of the water table. Other factors, such as groundwater flow velocity and soil porosity, have a definitive effect on the migration, areal extent, and concentrations of contamination in groundwater. For example, denser, less volatile compounds, such as No. 2 diesel fuel, will spread slower than the more volatile chlorinated solvents like trichloroethylene.

Typically, investigations of the degree and extent of subsurface contamination from UST leakage begins with a hydrogeologic investigation and sampling survey. Soil borings, sampling, and the installation of groundwater wells are well-established techniques that give direct observation of subsurface contamination conditions. If the contamination is extensive, a detailed, exhaustive, and expensive investigation and sampling program is required to define the areal extent of subsurface contamination plumes. Upon determination of the extent of contamination, remedial actions

and requirements can be determined. Depending on the extent of contamination, remediation can range from the removal of free product within the soils surrounding the UST pit to long-term pump and treat methods of contaminated groundwater remediation. The estimation of remedial costs is practically impossible without detailed delineation of the extent of contamination. Small fuel oil releases may be reduced to acceptable levels through continuous pumping or even hand-bailing. In other cases, partial product recovery, followed by the excavation of contaminated soil may be required. The disposal of contaminated soils is dependent on the level and types of contamination. Highly contaminated soils may require transportation and disposal at an approved hazardous waste treatment or disposal facility, while soils having acceptable levels of contamination may be transported to locally approved landfills for disposal.

### Demolition of Structures

Often, the development of land will require the demolition of the existing structures. Similar to environmental site audits, surveys of the existing structures should be performed to determine if hazardous materials are present.

Although there are several reasons to perform these surveys, the predominant reason is that regulations in most states require the safe handling and disposal of hazardous materials. This is particularly true of asbestos, lead-based paint and ozone-depleting substances, for which most states and/or the federal government have regulatory requirements that affect disposal. Other reasons for knowing the location(s) of hazardous materials within the existing structures are:

- To protect the developers'/contractors' workers
- Reduce potential financial liability to the developer ("not knowing" is not an excuse)
- Minimize the amount of hazardous waste generated (and disposal costs)
- Ensure that hazardous materials are not spread throughout the site
- Protect the public and environment at large
- Eliminate public outcry and bad press by doing the right thing

Based on the previous use of a property and its structures, there can be many types of hazardous materials in a structure. A qualified environmental firm should be hired and consulted not only to identify these materials, but also to recommend a method for the proper demolition and disposal of these materials. The treatment and disposal of hazardous materials in structures varies by region and state, so it is recommended that the environmental firm be knowledgeable with respect to all regulations, but particularly to state and local regulations. Pertinent federal regulations ap-

plying to hazardous material demolition will be cited in this chapter.

Although asbestos and lead-based paint tend to have the most significant cost implications when demolishing existing structures, the following hazardous materials should also be identified and handled in accordance with the most stringent prevailing regulation for the site:

- PCBs (typically found in light ballasts)
- Chlorofluorocarbons (typically found in refrigerators and air conditioning units)
- Mercury (typically found in thermostats)
- Radioactive sources (typically found in smoke detectors)
- Household chemicals, cleaners, and pesticides (typically found in kitchens, bathrooms, basements, garages, and out buildings)

Disposal criteria of the above (or other) materials may vary widely between states and local jurisdictions. For example, state and local jurisdictions may consider some of the above-listed materials as falling under "conditionally exempt small-quantity generators" (or even falling under the homeowner exclusion), and the materials need only be disposed of properly in a state-licensed or permitted solid waste management facility, along with all the other construction debris. While other state and local jurisdictions may require that these materials be removed and separated from the construction debris and taken to a hazardous waste disposal facility. The danger in not determining disposal criteria before demolition, in addition to harming humans or the environment, is that a large quantity of demolition debris with a relatively small amount of hazardous material could result in authorities declaring the entire wastestream as hazardous, thereby requiring more expensive disposal. In 2001, it cost approximately \$750 to dispose of 40 cubic yards of construction debris and \$4500 to dispose of 40 cubic yards of hazardous waste.

Under Section 608 of the Clean Air Act of 1990, the EPA established regulations (40 CFR Part 82, Subpart F) that require service practices that maximize recycling of ozone-depleting compounds (chlorofluorocarbons and hydrochlorofluorocarbons) during the servicing and disposal of air-conditioning and refrigeration equipment. A certified technician must perform recovery of the refrigerant, and recovery can be performed on site or at a disposal facility (Reference: <http://www.epa.gov/docs/ozone/title6/608fact.html>).

Since the presence of asbestos-containing materials (ACM) and, in some cases, lead-based paint (LBP) in structures to be demolished (or renovated) can place a greater financial liability on the developer for the risks to workers, the public, and the environment, these materials will be discussed in greater detail.



## ASBESTOS

Asbestos concerns in land development consist primarily of the control of airborne fiber releases during the demolition of existing structures that possess ACMs, excavation and disturbance of naturally occurring asbestos, and the removal of asbestos-containing debris from dumps or piles located on the property. In each case, provisions for the control of airborne asbestos fiber releases, exposure to workers, and the proper containerization, transportation, and disposal of asbestos-contaminated materials are required.

“Asbestos” is a broad term for a group of naturally occurring hydrated silicates that crystallize in a fibrous habit. Asbestos fibers in ores cannot be inhaled until released and made airborne during excavation, mining, and processing. The family of asbestos minerals is subdivided into two major groups, serpentine and amphiboles. Chrysolite asbestos, also known as white asbestos, which accounts for over 90% of the world’s production of asbestos, is the sole member of the serpentine asbestos group. Amphibole asbestos includes crocidolite, amosite, anthophyllite, actinolite, and tremolite. Two of the amphiboles amosite (or brown asbestos) and crocidolite (or blue asbestos) are commercially valuable forms of asbestos. Although asbestos use dates back centuries, it was not until the Asbestos Hazard Emergency Response Act, effective in 1987, that public awareness and attention became focused on managing and mitigating ACMs. The mineral was attractive for use because of its ability to insulate against heat, cold, and noise, incombustibility, good dielectric properties, great tensile strength, flexibility, and resistance to corrosion. Subsequently, it is found in many building materials. Additionally, asbestos is naturally occurring and can be found in many parts of the United States.

### Health Effects

Asbestos is a known human carcinogen (cancer-causing substance). Exposure to asbestos fibers can cause several different types of asbestos-related diseases. The primary exposure route of concern is inhalation; however, asbestos fibers can be ingested. Asbestos is not normally absorbed through the skin. Exposure to large concentrations of asbestos can result in the disease asbestosis. Asbestosis, a chronic scarring disorder of the lower respiratory tract, occurs when asbestos fibers lodge in the lungs. The latency period (or period of time it takes to develop) for asbestosis is 5 to 10 years with heavy exposure but can take up to 20 to 40 years to develop. Lung cancer is another disease that can be developed through exposure to asbestos. The latency period for lung cancer caused by inhalation of asbestos is usually at least 15 years with a peak of 30 to 35 years. A unique form of cancer, which is directly related to asbestos exposure, is Mesothelioma. Mesothelioma tumors are the uncontrolled growth of cancerous cells in the lining of the chest cavity (pleura), between the chest walls and lungs, or

in the lining of the abdominal cavity (peritoneum). It is important to note that many cases of Mesothelioma are associated with relatively low levels of asbestos exposure as well as exposures of even short duration. Additionally, the latency period for Mesothelioma is long, with cases reported with as many as 40 years passing between exposure and the onset of the disease. Mesothelioma is an incurable form of cancer and usually fatal within 1 to 2 years from diagnosis.

The EPA requires the control of airborne visible emissions of asbestos under the National Emission Standards for Hazardous Air Pollutants (NESHAPS) regulations (40 CFR 61, Subpart M). For the most part, it will be the NESHAPS regulations that a developer will be required to follow when demolishing structures. Many local jurisdictions ensure that asbestos concerns are addressed during building demolition (or renovation) by requiring an asbestos survey as part of the demolition permit process. It is important to note that under NESHAPS, certain building material forms of ACMs within a structure do not need to be removed before demolition. Rather, under certain conditions, they can be included in the wastestream with the other building demolition materials and taken to an approved construction debris landfill for disposal. It is highly recommended that a developer contact a local knowledgeable environmental firm to assist in this process, as they should know local regulations and understand what the regulators require with this process. The EPA also requires the proper containerization, transportation, and disposal of asbestos-containing or asbestos-contaminated materials at approved landfills. In addition, the EPA, through its regulation 40 CFR 763, among other items, requires training for asbestos inspectors and workers, which most states regulate through a licensing program.

The Occupational Safety and Health Administration (OSHA) also has published regulations for the protection of workers from exposures to asbestos. These include control measures that establish controlled work areas, wetting ACM to preclude fiber releases, the wearing of personal protective equipment in the form of full body protective suits and respiratory protection, decontamination of workers and equipment, and the collection of worker exposure and environmental air samples. These control requirements can be found in OSHA regulations 29 CFR 1910.1001 and 29 CFR 1926.1101.

Although asbestos has been used in many products over the years, the construction business has been the main consumer of asbestos-containing materials. Table 19.1 lists ACMs that are common in a structure and the approximate cost to abate.

Identification of asbestos or ACMs consists of the sampling and laboratory analysis of suspect materials. The primary methods of asbestos identification involve microscopic inspection of bulk samples or pieces of suspect materials. Polarized light microscopy (PLM) is probably the

**TABLE 19.1 Common ACM and Associated Abatement Costs**

<b>ASBESTOS CONTAINING MATERIAL</b>	<b>COST TO ABATE<sup>1,2</sup></b>	<b>UNIT<sup>3</sup></b>
Floor tile and mastic	\$3.50	SF
Floor tile	\$2.50	SF
Mastic	\$1.50	SF
Pipe insulation	\$17.17	LF
Pipe joint insulation	\$29.00	EA
Boiler insulation	\$12.00	SF
Roofing materials	\$4.00	SF
Asbestos siding shingles (transite)	\$5.50	SF
Vapor barrier	\$3.50	SF
Wall board	\$5.50	SF
Surfacing material (troweled or spray-applied)	\$10.25	SF
Ceiling tile	\$4.00	SF
Bulk insulation	\$12.50	CF

<sup>1</sup>Cost varies widely between areas of country, and time of year. These costs only reflect an approximation of a typical unit cost to remove ACM and should not be relied upon to cost or estimate a project. Not included in these costs are such items as mobilization, monitoring, and other costs outside the actual removal of ACM.

<sup>2</sup>In 2001 dollars, based on historical records at Dewberry & Davis LLC.

<sup>3</sup>SF = square feet, LF = linear feet, CF = cubic feet, EA = each.

most common type of laboratory identification of asbestos and the least costly type of analysis.

Phase contrast microscopy (PCM) is used in determining asbestos exposure concentrations to workers occupationally exposed to asbestos. The analysis of air samples for asbestos involves collecting the fibers on filters connected to air sampling or suction pumps that are set at a given flow rate to collect an adequate volume of air for analysis.

The more sophisticated types of microscopic analysis such as transmission electron microscopy (TEM) and scanning electron microscopy (SEM) are used to detect asbestos that is present in a very tight matrix or very bound ACM. TEM is also used in the analysis of air samples collected to determine airborne asbestos fiber concentrations. Note: TEM and SEM analysis costs are several times that of PLM and PCM, and generally are not required for demolition projects.

Before development or property acquisition, or even the digging or excavation of a piece of property, investigation into the area geology should be performed as part of the environmental assessment and survey. The county soils science office should be contacted and the county soil survey reviewed to determine if soils or geology within the area have characteristics of naturally occurring asbestos. Again,

a local environmental firm knowledgeable in local conditions and able to assist a developer in determining the presence of naturally occurring asbestos and the process of working with it should be consulted prior to construction activities commencing.

### **LEAD-BASED PAINT**

Lead-based paint (LBP) concerns in land development consist primarily of the determination of whether LBP in a structure's demolition debris (wastestream) will cause that wastestream to be classified as a hazardous waste. Soil should also be tested around structures to determine if it is contaminated by LBP from the structure. It will be more cost effective to determine if lead-contaminated soil is a result of LBP from a structure versus other sources of contamination.

### **Exposure and Risk**

A highly toxic heavy metal, lead was a major component of household paint until banned from residential use in 1978. As LBP ages, lead dust, chips, or flakes may enter the environment and can be ingested or inhaled. Medical research has determined that lead exposure—even at extremely low levels—can decrease intelligence, inhibit school

performance, and impair memory attention and language function. Young children, pregnant women, and women in the childbearing years are particularly at risk from toxic lead. As a result, lead-based paint and dust is recognized as a potential health hazard in buildings and other structures. Figure 19.2 shows peeling lead-based paint on a structure.

Because of these concerns, Congress passed “The 1992 Residential Lead-Based Paint Hazard Reduction Act” (42 U.S.C. 4851 *et seq.*, PL 101-550). This Act is commonly known as “Title X” (ten) by the industry. Title X established the framework for informing the public, the training and certification of inspectors, contractors and laboratories, and protection of abatement workers. Therefore, states have and will be reacting to Title X and other concerns for protecting our nation’s children from lead-based paint and dust by enacting legislation. The developer is probably most affected by the legislation promulgated by Title X with respect to protection of workers during the demolition of structures, and the proper disposal of demolition debris. The Housing and Urban Development’s (HUD) webpage regarding LBP is probably the best to access, review, and link to other websites, such as the EPA and the OSHA websites on LBP facts, programs, and regulations (<http://www.hud.gov/offices/lead/>, enter their “reference library”).

### Testing and Mitigation

The objective, as with the other hazardous materials listed above, is to predict prior to demolition the disposal criteria of a wastestream from the demolition of a structure(s) and to do everything legally possible to dispose of the wastestream in a properly permitted construction and demolition landfill, as opposed to a hazardous waste landfill. The test, which determines if LBP in a structure will cause the demolition debris (wastestream) of that structure to be considered hazardous, is called the Toxicity Characteristic



**FIGURE 19.2** Lead-based paint peeling from a structure.

Leaching Procedure test (TCLP). This is further discussed in 40 CFR 261.24.

Basically, performing a TCLP test to determine the disposal criteria involves the estimation and quantification (by percentage) of all building materials (regardless whether painted), and the collection of these building materials in their representative quantities for submission to a laboratory for analysis. The laboratory will take these various materials and composite them (in their specified representative quantities) in a 100-gram sample and perform the TCLP test. If the result equals or exceeds 5 mg/L, the wastestream is considered hazardous, and if under 5 mg/L, the wastestream is nonhazardous and can be taken to a construction and demolition landfill for disposal, in accordance with state and local regulations.

If the TCLP test equals or exceed 5 mg/L, it is probably more economical for the developer to perform specific LBP testing to determine which components contain the LBP, and abate (remove) those components prior to demolition. If a wastestream is predicted as hazardous due to LBP, then abatement of the LBP is generally recommended prior to the demolition of the structure to reduce the amount of hazardous waste. As noted above, it costs approximately \$750 to dispose of 40 cubic yards of demolition debris in a construction landfill as opposed to \$4500 per cubic yard in a hazardous waste landfill. At present (Spring 2001), there is proposed legislation that could eliminate the TCLP testing for lead-based paint in renovation or demolition projects at a federal level. It is important to note that different state and local jurisdictions and landfills may impose various requirements on disposal material. It is highly recommended that developers consult and hire an environmental firm that understands this issue—federal, state, and local requirements—and can perform any required sampling and analysis.

Regardless of impending regulations for disposal, OSHA requires the protection of workers during the demolition of structures containing lead in paint. These requirements can be reviewed in 29 CFR 1926.62, Lead in Construction. An environmental firm can assist a developer in meeting these requirements.

The developer should also consider testing soil around structures for lead, resulting from the leaching or deterioration of LBP from the structure. This is a simple test where soil samples are taken from the drip lines around a structure, and bare soil areas in yard areas. These samples are taken from the top 0.5 inch of soil, typically composited, and sent to a laboratory for *total lead* analysis. A lead hazard is present if sample results equal or exceed 1,200 parts per million. Lead contamination in soil resulting from LBP can be treated under the LBP regulations found in TSCA as opposed to the requirements under CERCLA, which tends to be more costly and may take more time to abate and monitor. Again, an environmental firm should be hired to consult the developer regarding this process, as different states and local jurisdictions may view this process differ-

ently. The environmental firm should also be knowledgeable in collecting, processing, and analyzing the soil samples.

If a developer intends to reuse a structure (i.e., renovate), surface by surface inspection should be performed to determine exact locations of LBP. This will be necessary to protect workers performing the work during renovation, as well as to inform the future occupants of the structure (if the LBP was not removed during the renovation). In addition, like asbestos abatement, LBP abatement requires strict clearance criteria. If the structure is to be reused as a residence, the clearance should be conducted in accordance with the risk assessment protocol found in 24 CFR Part 35. The developer is required to disclose to the buyer any known locations of LBP. Since most developers will want to work with raw land, this process is beyond the intent or scope of this book. However, regulations and guidance on this topic can be found in 24 CFR Part 35, 40 CFR Part 745, and the "Guidelines for the Evaluation and Control of Lead-Based Paint Hazards in Housing." These documents can be accessed on the HUD webpage noted above. If LBP remains in a structure after renovation, the location of LBP must be disclosed to the buyer. If the developer intends to reuse structures on a property, it is recommended that an environmental firm be hired to perform the testing and risk assessment, and if LBP is present, design any required LBP abatement, and assist with the disclosure requirements.

If a structure is "historic" (generally defined as being on or eligible for the historic register) and to be included as part of the development, other regulations and requirements will apply. (Because of their age, most historic structures contain LBP.) If this is the case, and the developer is truly interested in restoring an historic structure, it is recommended that the developer immediately hire a knowledgeable "historic architect," generally found in a cultural resources firm, and an environmental firm and LBP abatement contractor with specific expertise with renovations in historic structures containing LBP. A historic renovation, especially in those structures containing LBP, has become a specialty expertise in the LBP marketplace and is beyond the intent and scope of this book.

## **RADON**

When considering land development and the sale of new construction, the developer, architect, and builder can face numerous areas of potential liability. High radon levels may breach warranties concerning the habitability of a property or good workmanship. Builders are advised not to include, contractually, any warranties covering radon. All express warranties should be clear and specific, leaving little or no room for interpretation to include radon.

Negligence claims can be made against the developer or builder for failure to use reasonable care when selecting the site for construction. An architect may be sued for negligence in his design of the property. Furthermore, the builder could be found negligent for not alerting buyers to the possible existence of, and dangers of, a radon gas prob-

lem. Additionally, this failure to warn can be interpreted as fraud and/or misrepresentation. It is suggested that builders make specific reference to radon in the context of warning about the dangers of radon exposure. It is then up to the buyer to decide if they wish to pursue the issue further. A builder can further protect his interests by also stating that the buyer cannot make the sale contingent simply on the results of short-term screening.

The EPA has developed a web page that goes into greater detail for radon issues and that should keep the reader/developer apprised of new information regarding radon. This webpage can be accessed at the following address: [www.epa.gov/iaq/radon](http://www.epa.gov/iaq/radon).

Radon is a naturally occurring colorless, odorless, tasteless gas produced from the radioactive decay of radium, which in turn results through several intermediate steps from the radioactive decay of uranium. Radon occurs naturally in geologic formations containing uranium, granite, shale, phosphate, or pitch blend. Since uranium and radium are common elements throughout the entire earth's crust, radon occurs naturally in varying concentrations throughout most soils. In addition, water from underground aquifers or wells may carry radon produced from the surrounding rock. Radon is a chemically unreactive, inert gas. Since radon is not chemically bound or attached to other substances, it can move easily through all gas permeable materials. Radon and its by-products are a lung cancer risk and may cause genetic damage. Typically, exposures to radon gas occur in confined areas such as buildings and basements where the gas can accumulate. Naturally emitted radon gas is unlikely to accumulate to hazardous concentrations in outdoor air or in the general atmosphere of a development site.

## **Health Risk and Effects of Exposure**

The primary known health risk associated with exposure to radon is an increased risk of developing lung cancer. Determining actual risk estimates for radon exposure is a difficult task. Scientists still disagree on appropriate risk models. Many factors influence if any one individual will develop lung cancer, including the amount and duration of exposure, smoking history, genetic predisposition, and many others. Most scientists agree that radiation exposure of any kind should be kept as low as reasonably achievable.

One thing of which we are certain is how exposure to radon and its decay products causes damage to lung tissue. The EPA estimates that 5000 to 20,000 lung cancer deaths per year in the United States alone may be directly attributed to radon exposure. In 1988, the U.S. Surgeon General's office called radon "the nation's number one environmental health hazard." It is the alpha radiation particles released by radon and its polonium daughters that pose the major risk. Since alpha particles are large and slow moving, they are stopped by our first layer of skin and pose little threat to us externally. However, when radon gas is inhaled, whether unattached or attached to dust, smoke, or other

aerosols, it easily adheres to lung tissue. The alpha particle then decays, resulting in one of two occurrences, either the cells are completely destroyed or the DNA of the cell is damaged. When this happens over a prolonged period, cell damage or mutations can occur, eventually leading to a cancerous condition. Scientists have also found that the combination of radon exposure and smoking greatly increases the risk of cancer.

### Detection

EPA has set an action level of 4.0 picocuries per liter (pCi/L) for continuous long-term exposure to radon gas. Additionally, the Radon Abatement Act of 1988: Amendment of Toxic Substance Control Act (15 U.S.C. 2661 *et seq.* PL 100-551) calls for a long-term national goal for all buildings nationwide to have indoor radon levels as low as the ambient air levels outdoors. When initial short-term testing shows levels above this guideline, EPA suggests a follow-up long-term test.

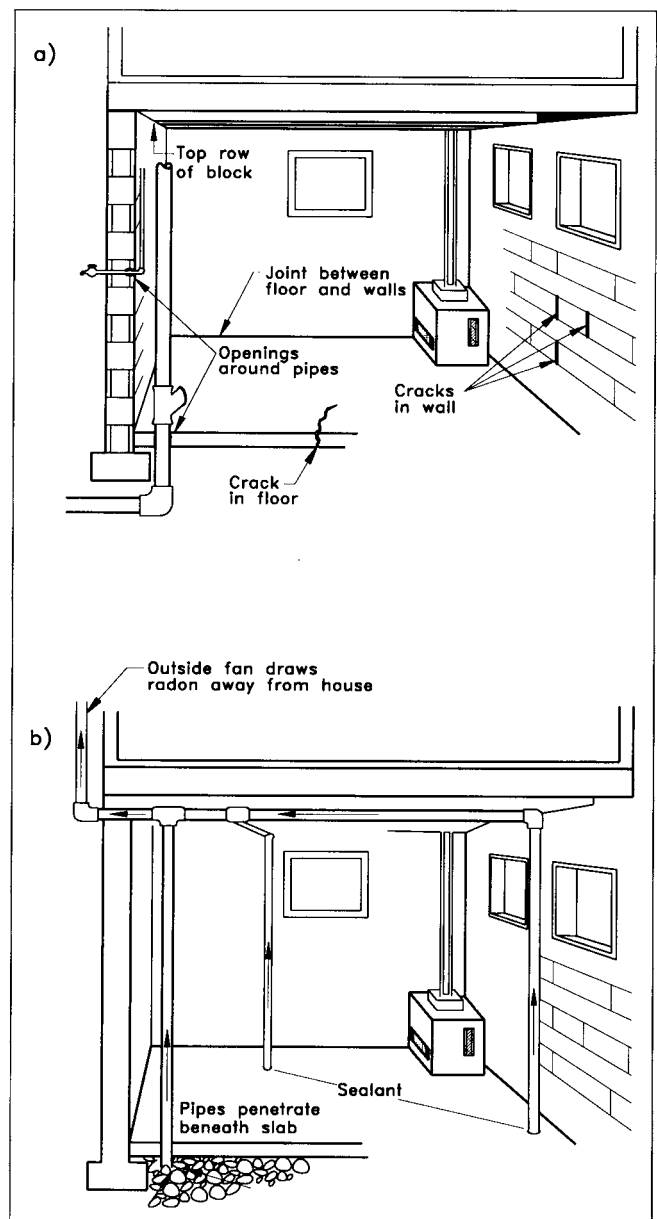
As the radon industry develops, an increasing number of detection devices and instruments are becoming commercially available. They are categorized as passive and active methods of short- and long-term measurement. Passive methods usually require no external power. They take advantage of normal indoor airflow to obtain the necessary sampling. Some short-term passive measurement methods are charcoal absorption canisters, electrets, and continuous monitors. The electrets can be used both for short- and long-term sampling. The alpha track detector is a long-term passive monitor. Active methods draw air through them to collect a sample through a scintillation cell or to allow for deposit of radon decay products onto a filter paper. These devices are used primarily for shorter-term testing (less than one week).

When planning development at an undeveloped site, it may be difficult to predict whether a potential radon gas problem will exist in the structures that are to be built on the property. In this case, some investigation as to the potential for radon in the site geology and neighboring property structures will be required. In addition to the above-referenced EPA website, other sources of information in determining site radon potential include the local, or State Health Departments and the U.S. Geological Survey. Typically, State Health Departments maintain a listing of reported radon testing results within buildings throughout the area. The county's soil science office may also possess radon gas potential data for the area geology. The U.S. Geological Survey develops regional radon potential maps or maps defining geological formations, which may be indicative of potential radon gas production. If testing or site investigation confirms the potential for elevated concentrations of radon gas in development and/or new construction, remedial steps or design modifications are usually advised to maintain radon levels below the EPA "action level" of 4 pCi/L of air.

### Mitigation

The most practical approach to radon mitigation is to prevent radon from entering a building. This can be done in a variety of ways, most involving combinations of techniques.

For example, before installing a concrete floor, prepare the subgrade with 4 inches of crushed stone and a plastic membrane. Crawlspace should be isolated from the rest of the structure and vented to the outside. Major entry routes; e.g., plumbing and electrical lines, floor crack, etc., should be sealed with a polyurethane caulk (see Figure 19.3). This sealing process, however, normally only reduces levels a few picocuries and is more commonly used as a precursor to other mitigation techniques as opposed to being used alone.



**FIGURE 19.3** (a) Points of entry of radon into a building; (b) Subslab depressurization system.

The most common method used today involves a combination of sealing and a sub-slab depressurization system (see Figure 19.3). In existing buildings, PVC pipe (normally 4 in.) is inserted vertically through the concrete slab into the aggregate layer below. A fan is fitted into the piping network and the piping exits the building, preferably through the roof or out the sidewall. Activation of the fan causes soil gas (including radon) to be drawn through the stone under the slab into the piping and vented outdoors. This method can be modified for use in venting hollow cinder block walls as well as crawlspace areas. Sometimes, it is necessary to reverse the airflow, thereby pressuring the subslab rather than depressurizing. This particular technique has proven to reduce indoor radon levels up to 99%.

There are a number of other methods for reducing radon in buildings. Depending on the source or route of entry, a combination of methods such as natural or forced ventilation, drain tile suction, block wall ventilation, or subslab suction may be employed. EPA document OPA-86-005, *Radon Reduction Methods—A Homeowners Guide*, describes these methods in detail and can be obtained by calling the EPA's hotline at (202) 554-1404 or by accessing EPA's webpage for Radon (<http://www.epa.gov/iaq/radon>).

## AIR QUALITY

Air pollution sources to be considered in land development consist of new processes or sources resulting from the completion and start-up of facilities, generation of air pollutants during development and construction, and air pollution sources originating from surrounding or nearby properties. Air pollutants generated during development may include asbestos fiber releases, lead dust emissions, hydrocarbon emissions from heavy equipment, or dust as a result of demolishing structures or disturbing soil. New processes or operations upon completion of the development such as exhaust stacks, incinerators, welding or auto body shops, photo processing labs, manufacturing facilities, etc., must include the installation of appropriate air pollution control systems in land planning and design. A thorough environmental assessment will identify which activities or processes will require air pollution control measures upon completion of development. Local regulatory agencies can identify any requirements for restriction or control of air pollution emissions that may be generated during development and/or construction operations. The environmental assessment will also identify any off-site sources contributing pollutants to the air quality of the development property. Building designs can then incorporate the appropriate controls in building heating, ventilation, and air conditioning systems to insure the indoor air quality of structures.

Air contaminants (pollution) are categorized as particles and gases, including dusts, smoke, fumes, mists, and vapors. Some examples of common air pollutants are carbon monoxide, hydrocarbons, lead, nitrogen oxides, ozone, particulate matter, and sulfur oxides. Air pollution can be defined as the presence of one or more contaminant in the

air in concentrations that can be harmful to human health or the environment.

The EPA, through the Clean Air Act (CAA) [42 U.S.C. 7401 *et seq.*], has divided the United States into Air Quality Control Regions (AQCRs). The EPA also establishes National Ambient Air Quality Standards (NAAQS) for air pollutants. Each AQCR is evaluated for compliance with the NAAQS. Each state is required to develop and submit to the EPA State Implementation Plans (SIPs) for complying with NAAQS. The CAA amendments of 1990 divide areas not in compliance with NAAQS into categories and extend deadlines for compliance. These "non-attainment" areas are classified in order of significance as marginal, moderate, serious, severe, and extreme. To map a state's compliance with the standards set by the 1990 amendments, visit the EPA's environmental mapping website, Air Graphics, at <http://www.epa.gov/agweb>, or to map the current air quality by region or state, visit EPA Air Now at <http://www.epa.gov/airnow>.

The Clean Air Act also includes the regulation of air pollution generated from "mobile" and "stationary" sources through pollution emission limitation standards. It also includes requirements for the prevention of significant deterioration of air quality in regions where the ambient air quality is well within limits. Existing pollution sources are covered in the SIP. The EPA develops standards for categories of new sources. These standards are referred to as New Source Performance Standards (NSPS). The CAA amendments of 1990 require air emission operating permits nationwide and cover primarily all stationary sources. The CAA designates 189 substances as Hazardous Air Pollutants (HAP). Periodically, the EPA reviews the HAP list and may add pollutants. Air quality control in land development involves the evaluation of three major categories of sources:

- Air quality control during actual development construction must be considered. Fugitive dust during excavation and grading activities and the excavations of naturally occurring asbestos or the demolition of existing structures with asbestos-containing materials or lead-based paint, are also examples of potential air pollutants during development.
- Air quality control in the design of new processes or facilities is required to ensure compliance with emission standards and NAAQS. Dry cleaning, photo-labs, and auto painting are examples of these types of operations, which may affect development objectives.
- Land design and development must consider air pollution originating from adjacent properties or other sources that may affect the new development.

The effects of air pollution are dependent on many environmental factors, including topography, wind speed and direction, temperature and vertical changes in atmospheric temperature, sunlight, precipitation and elevation at which

air contaminants are released. These factors are all considered when predicting air pollution concentrations and their effect on human health. Condition, age, sensitivity, duration of exposure, and individual susceptibility are some of the factors that affect health conditions related to air pollution. With so many variables, it is very difficult to predict the end effect of air pollution on communities, populations, or the environment. The safest way to control adverse effects is to minimize to the extent possible (zero is most preferred) the emission of hazardous air pollutants at the source.

Air pollution effects and control must be considered early in land planning and development design. Along with specific strict regulatory requirements for the control of air pollution emission, the effects of air pollution originating off-site from adjacent properties to the development structures and future occupants of the property must be investigated and controls included in the design.

Since various states or local areas may have different compliance criteria, it is important for a developer to consult with a firm specializing in air quality compliance. This is especially so if development permits require the calculation of various air pollutant loads into the environment and appropriate methods to minimize those pollutant loads.

## NOISE

Land development engineers must be aware of the importance of noise impacts associated with construction and the surroundings of the property being developed. In addition, noise-generating neighbors such as airports, railways, highways, and light industrial facilities can adversely affect the development potential of property and must also be considered. The engineer should become thoroughly familiar with the noise-related requirements of the jurisdiction in which the project is located, since many communities now have specific requirements, especially for residential housing. These requirements might involve something as complex as the initiation by the developer of noise-impact studies, detailing the impact of development on adjacent areas and on how existing and anticipated noise levels are handled to lessen impacts to the proposed development.

Many local jurisdictions have noise impact legislation in the zoning ordinances or may even have specific noise ordinances. In addition, during the site approval process, the jurisdiction may make specific requirements relating to attenuation/mitigation of noise. This may come in the form of development requirements incorporated into the design.

Proper land use planning and the judicious use of zoning regulations can have a significant impact on preventing the development of noise-related problems. Several land-use strategies have been utilized to restrict the development of land that borders the right of way of the noise-generating source. One such strategy is to allow noise compatible development. For example, constructing structures that normally are unoccupied, such as warehouses and storage facilities, or constructing structures that contain activities

that are also noise generators, such as shopping centers and light industrial facilities in the vicinity of airports or highways.

Therefore, it is important for the engineer to understand the basic principles of noise production and abatement and understand thoroughly the local jurisdictional requirements imposed on the project site. The purpose of this section is to provide information on noise pollution: types of noise; measuring noise; and noise control measures.

## Noise Pollution

Noise can be a nuisance and is considered pollution because of its potential impacts on human health. These impacts include hearing impairments and psychological stress. The major sources of community noise are transportation and construction related. Noise does not need to be ear-piercing to be annoying. Studies conducted have produced varying results, but it has been generally found that:

1. Hearing damage from exposure to excessive noise is a cumulative process; both level and exposure times are important factors.
2. At a given level, low-frequency noise tends to be less damaging than noise in the mid-frequency range.
3. Individuals are not all equally susceptible to noise-induced hearing loss.
4. The hearing loss that results from noise tends to be most pronounced near 4000 Hz and above, but it spreads over the frequency range with increased exposure time and level (Peterson, 1980).

Exposure to a relatively high level of noise will result in some immediate hearing loss. However, if the individual is not exposed to that level of noise for a period of time, gradually one recovers the lost hearing ability. Yet, if one is repeatedly exposed to that level, the recovery will not be complete and hearing loss becomes permanent. Although many factors interrelate to cause this loss, it is most affected by the level of noise and the duration pattern of exposure. Often hearing loss is not detected since damage is progressive. Currently the OSHA has set permissible noise exposures as shown in Table 19.2 (Harris, 1992).

*Background noise* is defined as noise from all sources unrelated to a particular sound that is the object of interest. Background noise may include airborne, structureborne, and instrument noise (Harris, 1991). It is, therefore, all-inclusive. *Ambient noise* is defined as the composite of airborne sound from many sources, near and far, associated with a given environment. No particular sound is singled out for interest (Harris, 1991).

**Community Noise.** Community noise is an all-encompassing term used to identify noise intrusions from several sources. Traffic or highway noise is probably the most prominent. Current and future highway and trans-

**TABLE 19.2 Permissible Daily Noise Exposures**

NOISE LEVEL (dB)	ALLOWABLE DAILY EXPOSURE (hr)
85	16 (but periodic testing is required)
90	8
92	6
95	4
97	3
100	2
102	1.5
105	1
110	0.5
120	1 minute or less

portation planning must include a detailed evaluation of noise during design and routing of new facilities. Land use planning, through land use controls, can also have a major impact. National standards exist that are intended to guarantee quieter vehicles by legislating allowable noise emission. In addition, legislation is being instituted at various levels of government to protect residents by way of noise codes, noise ordinances, or specific requirements during the plan approval process. Aircraft flyover noise and runway noise operations are also monitored and legislated. All major airports have available noise contour maps that identify noise contours based on runway geometry, flight operations, and aircraft noise. These are frequently updated to account for the trend to lower noise levels of newer plane fleets.

Day-Night Average Sound Level (DNL) is generally considered as a uniform description of cumulative noise exposure, and research shows that it correlates with health and welfare effects. It is known as a metric and is employed to determine community (population) dose-response and annoyance of sound events. The DNL is intended to take into account both the noise levels of all individual events that occur during a 24-hr period, as well as the number of times these events occur (Federal, 1989). The DNL includes a 10-dB penalty for all noise events between 10 P.M. and 7 A.M.

Typically, the response to noise is annoyance and can be considered a general adverse reaction. It can include speech

interference, sleep disturbance, or simply the interference with a desire for a tranquil environment. The most commonly used measure of this is the percentage of the area population characterized as "highly annoyed" (% HA).

**Highway Noise.** Numerous research efforts have been directed to all of the major components of motor vehicle noise sources; these include engines, exhausts, tires, and transmissions. In addition, special studies have been directed at truck noise propagation. Standards have been adopted for new vehicles and generally have been effective in reducing levels of noise.

The DNL procedures are used to compare community response estimates based on characteristics of the intruding noise, consideration of background noise levels, and of community factors (Gordon, 1971). The use of computer-based highway noise prediction models has developed into a specialty within the engineering profession. It is recommended that the land design engineer rely on the services of such individuals, or firms, that provide expertise in this field, when situations warrant.

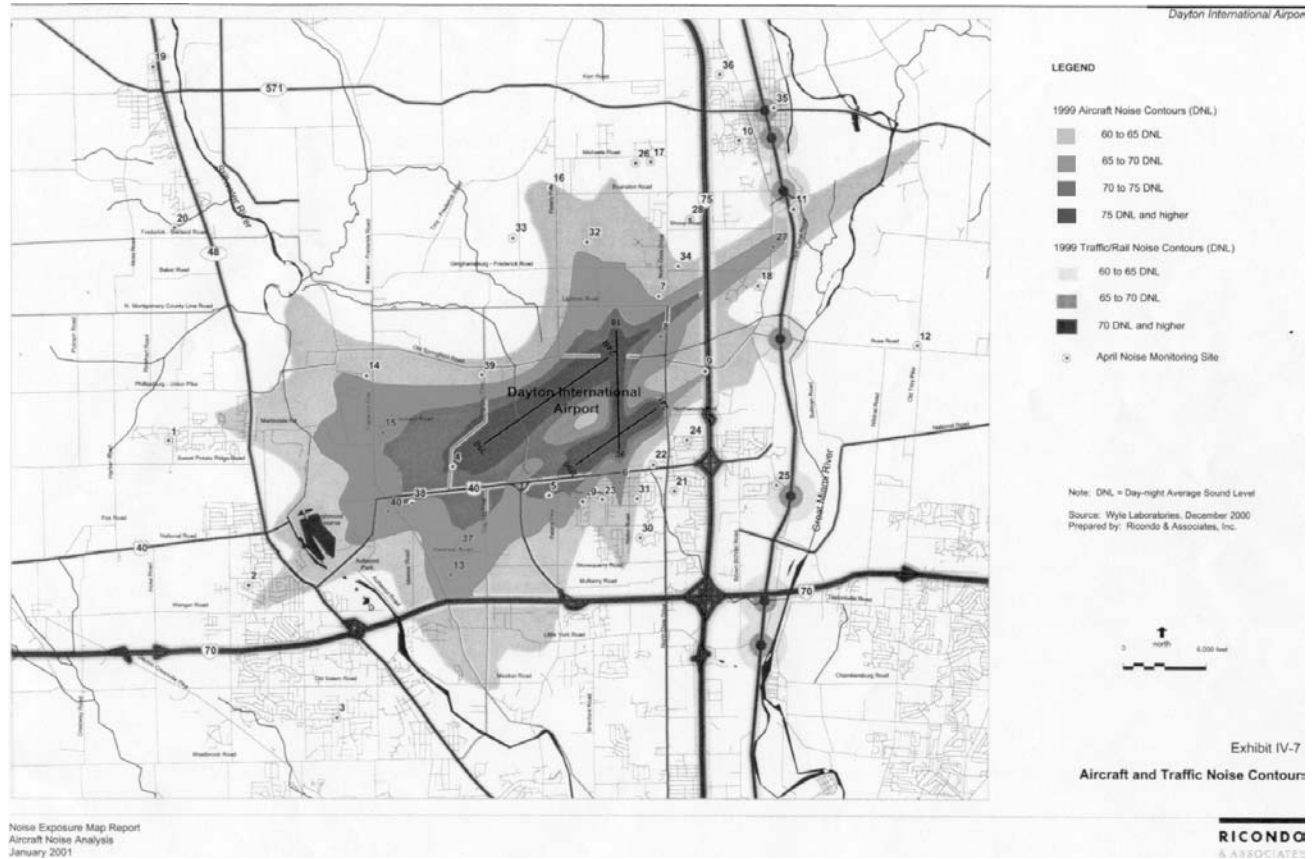
**Aircraft Noise.** There is little the land development engineer can do, in terms of noise abatement, regarding aircraft noise. It is recommended that if the project under consideration lies within 15 miles of an airport, the engineer obtain initial data to ascertain if there are any impacts to the site. Estimates of the noise exposure near an airport are usually available from the airport manager or from the Federal Aviation Administration (FAA). Figure 19.4 shows a typical aircraft contour map for a major airport.

Part 150 of the Federal Aviation Regulation (FAR Part 150) sets up the recommended standards for airport noise exposure maps. It identifies land uses that are normally compatible with various levels of DNL exposure. Levels below DNL 65 dB are usually acceptable for all land uses and can accommodate structures without noise protection modifications. When more levels are expected between DNL 65 and 75 dB, restrictions are usually placed on residential and educational use facilities. Above DNL 75 dB, all residential uses, even with noise attenuation measures, are considered unacceptable. A variety of nonresidential open spaces, commercial, and manufacturing land use are considered as compatible in areas with DNL greater than 75 dB.

Several computer programs have been developed to predict noise levels at airports. NOISEMAP and Integrated Noise Model (INM) take specific input data and produce DNL values and contour lines similar to elevation contours on topographic maps. These contours are usually at 5-dB intervals.

In addition, the engineer should ascertain from the airport operator if any new runways are planned. Information should be sought on the number of nighttime operations and the flight paths of each runway. Also, the engineer should ascertain if there are any planned changes for future operations. This information should be transmitted to the client.





**FIGURE 19.4** Typical airport noise contour map. (Provided by Dayton International Airport)

While there is little the land development engineer can do on the site for abatement of aircraft noise, some mitigation can result from incorporating the noise suppression techniques for building construction that are used for highway noise. These are discussed in detail in the previous section on highway noise abatement and in the section later in this chapter on building construction.

**Railway Noise.** Noise from railway operations can be mitigated through many of the same techniques used to suppress highway noise. The land development engineer should develop a basis of data if the subject site is within 3000 ft of a railway line, including rapid-rail transit lines. If the site falls within 3000 ft, the rail operator should be contacted and information obtained on the number of nighttime operations and any plans for future operations.

### Measuring Noise

Noise measurement is performed to provide definitive quantities for description and to rate sounds. The measurements that are taken can be used to improve building acoustics or to permit precise, scientific investigation of annoying sounds (noise). Due to the psychological and physical differences between individuals, the degree of annoyance cannot be scientifically measured; however, noise measurements provide an objective way of comparing annoying sounds. Such measurements can also provide an indication of when a sound can cause hearing loss. These

measurements provide levels of a person's hearing sensitivity. Measurement of noise serves to support and evaluate community noise abatement programs.

The most common measurement of noise is the A-weighted sound level, frequently abbreviated dB(A) or dBA. It is measured with a sound level meter using the A-weighting filter. This filter is used as a standard in sound measuring equipment since it follows closely the response characteristics of the human ear.

The evaluation of noise impact on hearing loss requires more than a single measurement of sound level. Noise exposure measurements on individuals need to be measured in doses and the devices that perform such measurements are called *noise dose meters* (or dosimeters). Dosimeters can be worn on an individual for an extended period (e.g., carried in a shirt pocket for several days), or they can be fixed, mounted models that record for extended periods. The most common dosimeters measure noise in dB(A).

### Noise Control

The source and impacts of noise must first be characterized in order to identify noise control measures. Incorporating noise abatement elements into the design of the project can eliminate the need to retrofit such elements after major design elements have been locked-in or, in a worst case, retrofitting abatement measures after a project is under construction or completed.

Basically there are three ways to control noise:

1. Control noise at the source
2. Control noise along the path from the source to the receiver
3. Control noise at the receiver

Noise control terminology often makes reference to SPR (Source, Path, Receiver). The source of noise can be controlled through better muffling, realigning or relocating the source, or through use of different tires on trucks, for example. Acoustical materials can be installed along the noise path to block or redirect the sound, such as sound walls or acoustical windows. For transportation noise, little can be done at the receiver since in most cases it is the inner ear.

Generally, the most cost-effective approach in terms of dollars/decibel reduction is to control noise at the source. Many times, however, the only practical approach may be to provide control along the path. In highway noise abatement, it will take years of research to develop quieter tires for trucks (control at the source), so the only real option is to block or dampen the noise along the path via earthen sound berms, barriers, or acoustic windows. The engineer sometimes can control noise at the receiver by reorienting buildings during the initial design stage.

**Noise Barriers.** Noise or sound barriers are the most commonly used abatement measure. A sound barrier is a device that is placed between the noise source and the receptor in order to cause transmission loss or diffraction. The barrier causes diffraction of sound waves, either around or over it, and causes transmission loss of sound through it. Most barriers in the U.S. are constructed of wood, metal, concrete, earth (berm), or some combination of these.

An important element in the selection of the type of noise barriers to use is the perceived effectiveness. Appearance or the attractiveness of the materials used to construct the barrier affects the perceived effectiveness of the barrier. It is generally concluded that the most attractive barriers are earth berms. As detailed later, earth berms require significant horizontal land area to enable the height to be sufficient to provide abatement. Table 19.3 (Cohn, 1981)

**TABLE 19.3 Acceptance Ranking for Various Types of Barriers**

BARRIER MATERIAL	ACCEPTANCE RANKING
Earth berm	1
Wall/berm combination	2
Wood	3

Reproduced with permission from Cohn, Louis F. 1981. *Highway Noise Barriers N.C.H.R.P. #871*. Washington, D.C.: Transportation Research Board.

identifies acceptance ranking for barriers of different materials.

The aesthetics of the barrier design also has a major effect on the perceived effectiveness as well as on its general acceptance. Aesthetical treatments include type, color, material selection, concurrent landscaping, and surface texture. Recent trends in concrete barriers have resulted in attractive installations that complement the total landscape treatments. One state, Colorado, has found that a psychological attenuation occurs of up to 7 dBA in perceived effectiveness from well designed aesthetics. Placing murals or graphics on the barrier can also be used to increase attractiveness.

Figure 19.5 through 19.8 show typical installations of various types of barriers. It is important for the engineer to give consideration to the following list when considering the design of barriers:

#### Earth Berms

- Figure 19.5 shows a typical cross-section drawing for a 12-ft high earth berm barrier. If the side slopes are 2:1, the base width is 51 ft. For maintenance purposes many consider 2:1 slopes to be too steep for safe mowing, with a preferred slope of 3:1. This increases the base width to 75 ft. This loss of land area can cause severe impact to development. A combination of a short barrier wall on top of the landscaped berm might provide a good compromise. Figure 19.5 shows such an installation.

- Concern should be given to proper compaction, particularly if a steep slope is allowed.

- Earth berms can provide a nearby site for disposal of excess site dirt. However, if the site does not provide such an excess, the transportation and placement cost must be taken into account for project budgeting.

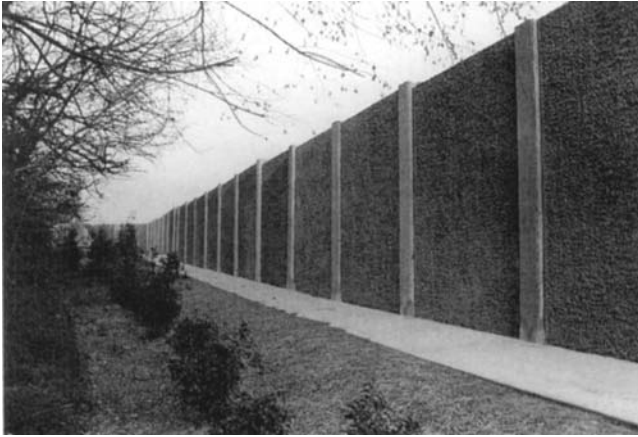
- Earth berms can also provide an attractive project approach feature, particularly when properly landscaped.

#### Concrete Barriers

- The access to the barrier site is important if large panels are used. If heavy precast panels are used, con-



**FIGURE 19.5** Noise barrier: combination earth berm and wall.



**FIGURE 19.6** Noise barrier: exposed aggregate concrete.

struction costs can be excessive if the contractor expects difficulty in transporting. On-site storage of panels can be a problem. Figure 19.6 shows a typical concrete barrier.

- Sufficient soil boring information is needed to allow proper footing design.
- Difficulty can result when lateral support is needed, particularly in unstable soil conditions.
- Significant costs can result if deep piles are needed.

#### *Metal Barriers*

- Problems can result if care is not given to barrier material color during fabrication. Figure 19.7 shows a typical metal barrier.
- Panels can be damaged during installation and particularly during inventory storage on site.
- Expansion and contraction after installation can result in detached panels.



**FIGURE 19.7** Noise barrier: metal.

- Damage (denting) can occur after installation. This can be significant if replacement parts and materials become unavailable.

#### *Wood Barriers*

- Problems can develop with wood shrinkage, warping, and separations. Figure 19.8 shows a typical wood barrier.
- In some cases extra vertical bracing might be needed to keep the decking from separating.

#### *All Barriers*

- Care must be exercised to identify any existing utilities that might require relocation.
- The later retrofit of new utilities can result in high costs if the utilities have to be installed around the barriers.
- Difficulty in mowing can occur, particularly if the slopes are in excess of 3:1.
- Litter accumulation and excessive weed growth can cause higher maintenance cost.
- Vandalism, such as graffiti, can also cause high maintenance cost.
- It can be costly to provide maintenance services behind barriers. In some examples of barriers in use, a break to allow maintenance vehicles to pass can be effective. Care must be given to block noise through a second duplicate barrier section.

*Trees as Barriers.* Trees or landscaping as a technique for noise abatement is generally over-rated and can be exaggerated. A dense growth of trees can result in as much as 5 dBA per 100 ft of depth, up to 10 dBA total (Cohn, 1981).

It is generally felt that as much as 500 to 1000 feet of such foliage is needed, but this can cause severe impact to site development. As indicated previously, if a certain amount of open space (or undistributed area) is required,



**FIGURE 19.8** Noise barrier: wood.

this area, if heavily treed, can serve such a purpose. Care must be taken to ensure that such foliage has a high proportion of nondeciduous variety, otherwise, the noise impact will be high in winter months. Trees, in combination with a noise barrier such as a wall or an earthen berm, can be attractive and can lessen noise impacts.

It is important to note here that psychological impacts can be important in developing an attractive project. The spatial relationships of the project can be important, especially with regard to site elements such as pedestrian facilities (hiking or biking trails, for example). Generally, this is considered visual enclosure of space. If one can design an attractive area, including elements of landscaping, attractive views, the emphasis of changing topography, and the inclusion of hydrological events such as water ponds, the visual impact can psychologically lessen noise impacts. Such water elements can include required stormwater facilities to result in an additional positive result.

**Noise Abatement in Building Construction.** As indicated above, there are several means of abatement for noise. We have discussed the use of physical barriers to achieve such abatement. Also discussed above is the suppression of noise through land use and zoning and other means of good planning and innovative design techniques. This section covers a third means, that is, reduction of interior noise in structures through sound treatment in building construction. Covered here are building sound treatments to reduce noise through modifications to existing buildings as well as the use of design details for future buildings. One study has found that up to 15 dBA of interior noise reduction is possible for structures through proper sound-insulating construction techniques (Cohn, 1981).

A standard rating system has been developed to rate the speech privacy provided by a barrier. It rates the sound transmission loss for typical building materials. This sound transmission class (STC) is used to rate the acoustical performance of building construction elements. It is based on air tightness, mass, and how noise absorbent the core material is.

In general, measures to lessen noise impacts can be both exterior and interior. For exterior building construction elements, consideration should be given to the structural frame, exterior walls, windows, exterior doors, roof, ceilings, and construction of floors or specifically underfloors. For interior elements, consideration should be given to interior surfaces of exterior walls and ceilings and environmental controls such as air conditioning.

Windows and doors are the most effective areas for sound insulation, since they are receptor areas where most of the noise enters a structure. Conventional window design has low-noise (sound) transmission loss because of the single thickness of the glass, and the fact that glass is not a good insulator by itself. Doors allow transmissions of noise through air gaps around the sashes and through single-thickness door panel construction.

There are numerous elements of building construction that can be incorporated in reconstruction of existing buildings or in new construction. These include the use of gypsum board for exterior walls, building insulation, double-pane glass for windows, heavy weight doors, sealant for air gaps, weather stripping and gaskets. Note: the acoustical benefits of insulating glass are not as effective as double-glazed construction, since the airspace between the panes of insulating glass is only a fraction of an inch; however, newer, specially-designed sound insulating windows are available. These are usually two panes of glass in fixed and sealed sashes separated by a 2- to 4-inch airspace. These sashes usually contain a minimum  $\frac{3}{16}$  inch thick glass.

Typically, basements are considered to be unoccupied and do not include insulating elements. However, if basement areas are to be used as living areas, it is recommended that insulating materials be utilized, particularly for windows.

Modifying existing structures or constructing new buildings with air conditioning can have a positive effect on internal noise abatement. The best units are central station equipment, which, with an appropriate distribution system and/or terminal units, provide year-round air treatment. Acoustically-lined fresh-air intake and exhaust-air ducts to central fans are recommended (Cohn, 1981).

Additional benefits can result in using high sound insulation materials. Construction elements that provide protection from noise also can reduce heating and air conditioning cost by as much as 50%. It should be noted that although the use of such installations can reduce winter maintenance costs, if non-opening windows are used, the summer maintenance costs can increase since open-air ventilation is denied.

Exterior noise can be transmitted through building components such as connecting passageways or through fresh air intakes and exhaust ducts. Therefore, it is important to consider insulating attic space vents and interior duct systems.

Highway noise and its abatement is probably the one area of this subject that has been researched the most, and, consequently, the area where positive impacts have been seen to the greatest extent. This section is not intended to cover all of the technical aspects of noise analysis and abatement; it is intended, rather, to provide information so that the land design engineer can include measures in the planning and design of the project. At the same time, it is important for the engineer to recognize when a problem exists or may exist that requires the services of others more highly trained in this field.

## REFERENCES

- 24 CFR Part 35, 2001.
- 29 CFR Part 763, 2001.
- 29 CFR Part 1910.1001, 2001.

- 29 CFR Part 1926.62, 2001.
- 29 CFR Part 1926.1101, 2001.
- 40 CFR Part 61, Subpart M, 2001.
- 40 CFR Part 82, Subpart F, 2001.
- 40 CFR Parts 260-299 2001.
- 40 CFR Part 261.24, 2001.
- 40 CFR Part 763, 2001.
- 40 CFR Part 745, 2001.
- The American Heritage Dictionary, Houghton Mifflin Company, 1985.
- The 1992 Residential Lead-Based Paint Hazard Reduction Act (42 U.S.C. 4851 et. seq. PL 101-550), 1992.
- Barry, T. M., and J. A. Reagan, *FHWA Highway Noise Prediction Model*, U.S. Department of Transportation, Washington, D.C., 1978.
- Bolt Beranek and Newman Inc., *Highway Noise Generation and Control*, Transportation Research Board, Washington, D.C., 1977.
- Bowlby, William, ed., *Sound Procedures For Measuring Highway Noise*, U.S. Department of Transportation, Washington, D.C., 1982.
- The Clean Air Act (CAA) (42 U.S.C. 7401 et. seq. PL 101-549), 1990.
- Cohn, Louis F., *Highway Noise Barriers N.C.H.R.P #87*, Transportation Research Board, Washington, D.C., 1981.
- Cooke, Susan M., and Matthew Bender, *The Law of Hazardous Waste*, Matthew Bender & Co. Inc., New York, 1992.
- Corbitt, Robert A., *Standard Handbook of Environmental Engineering*, McGraw-Hill, New York, 1990.
- Deviny, Everett, Lu, and Stollar, *Subsurface Migration of Hazardous Wastes*, Van Nostrand Reinhold, New York, 1990.
- The Environmental Audit Handbook*, Business & Legal Reports, Inc., Madison, Conn., 1990.
- Gordon, Colin G., William J. Galloway, B. Andrew Kugler, and Daniel L. Nelson, *Highway Noise, A Design Guide For Highway Engineers N.C.H.R.P #117*, Highway Research Board, Washington, D.C., 1971.
- "Guidelines for the Evaluation and Control of Lead-Based Paint in Housing," 1995, Revised 1997.
- Harris, Richard W., *Arboriculture: Integrated Management of Landscape Trees, Shrubs and Vines*, 2nd ed., Prentice Hall, Englewood Cliffs, N.J., 1992.
- Harris, David A., *Noise Control Manual*, Van Nostrand Reinhold, New York, 1991.
- Harry M. Freeman, editor in chief, *Standard Handbook of Hazardous Waste Treatment and Disposal*, McGraw-Hill, New York, 1989.
- Hopcroft, Vitale, and Anglehart, *Hazardous Material and Hazardous Waste*, R.S. Means Company, Inc., Kingston, Mass., 1989.
- Lieter, Jeffrey L., editor in Chief, *Underground Storage Tank Guide*, Thompson Publishing Group, Salisbury, Md., 1988.
- O'Brien, James P., and William Harris Frank, *Environmental Due Diligence*, The Bureau of National Affairs, Inc., Washington, D.C., 1989.
- Parkin, William P., Marburg Assoc., *Site Auditing: Environmental Assessment of Property*, Specialty Technical Publishers Inc., Vancouver B.C. Canada, 1991.
- Patty's Industrial Hygiene and Toxicology, Third Revised Edition*, Wiley-Interscience: New York, 1978.
- Peterson, Arnold P.G., *Handbook of Noise Measurement*, GenRad, Inc., Concord, Mass., 1980.
- Radon Abatement Act of 1988 Amendment of Toxic Substance Control Act (15 U.S.C. 2661 et. seq. PL 100-551), 1988.
- Rom, William N., M.D., 1992. *Environmental and Occupational Medicine, Second Edition*. MPH: New York, 1992.
- Rothenburg, Eric B., and Dean Jeffrey Telego, *Environmental Risk Management*, RTM Communications Inc., Alexandria, Va., 1991.
- Salvato, Joseph A., P.E., *Environmental Engineering and Sanitation*, 3d ed., Wiley-Interscience, New York, 1982.
- Transportation Research Board, *Issues in Transportation Noise Mitigation: Highway and Railway Studies TRR #983*, Transportation Research Board, Washington, D.C., 1984.

## PART V

# FINAL DESIGN

*Dennis Couture, A.S.L.A., R.L.A.*

### **STEP 5: FINAL DESIGN**

After the client and local governing agencies have reviewed and accepted the schematic design and preliminary engineering study, the civil engineers prepare the final design. The final design reflects the detail necessary for project re-

view and approval by local governing agencies, which authorizes the final commencement of construction. The site plan developed during the final design represents the final documentation of the land development design process. The main components of final design are chronicled in the ensuing pages.



## CHAPTER 20

# SUBURBAN STREET DESIGN

*J. Thomas Tanner, P.E.*

### INTRODUCTION

Most of the design criteria for high-speed, high-volume highways has been established by organizations such as the American Association of State Highway and Transportation Officials (AASHTO), the Institute of Transportation Engineers (ITE), and the Federal Highway Administration (FHWA). Design criteria for low-speed roads is typically established by local municipal or state agencies by modifying the criteria for highways. However, recent revisions to the popular AASHTO design manual, *A Policy on Geometric Design of Highways and Streets* have incorporated discussions and provided design guidance regarding residential type streets in urban settings. Many local agencies have adopted AASHTO criteria entirely or with modifications to reflect regional philosophies and conditions. AASHTO criteria represent the consensus of state transportation agencies and the FHWA. They provide a consistent basis for highway design with appropriate variations for climates terrain and economic factors.

The FHWA publishes the Manual on Uniform Traffic Control Devices (MUTCD). This manual has been officially adopted by most states and has become an important legal document because it assures that pavement marking signing and signaling devices will be uniformly and consistently applied. Local jurisdictions, in most instances, are required by state law to conform to the MUTCD. Use of this standard applies to both the selections of appropriate traffic control devices and to design and application of markings and signs.

High-speed, high-volume highway design is mostly governed by vehicle and driver characteristics. Local urban street design is governed by the same criteria but to a lesser

degree. More emphasis is placed on pedestrian safety, local service, land accessibility, and the preference by residents for a pleasant environment.

In many localities the actual construction and the cost of construction of the street network of a development project is the responsibility of the developer. Once the streets are constructed, they are then accepted as part of the local or state street network system, and the costs of repair and maintenance become the responsibility of public agencies. Maintenance costs, in most cases, are funded through local and state taxes. Higher order roads on the other hand are typically constructed with state and federal funds and maintained with state funds.

It becomes imperative for the streets to be designed and built according to the governing criteria. Failure to do so results in additional costs in design and construction in order to have the street accepted into the state or local system. Nonstandard design may also cloud liability in accident cases. Some developers choose to provide private streets. The cost burden for repair and maintenance will then be the responsibility of the private owner(s).

### HIERARCHY OF STREETS

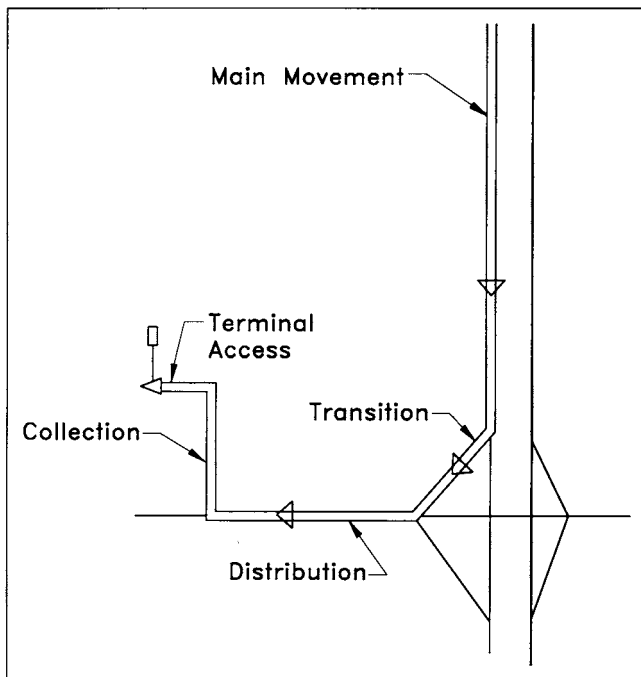
Several different classification systems exist for grouping highways and streets. The various systems are used by different agencies and other interest groups to establish a common base according to a particular interest, function, or operation of the road. The various systems include: grouping for administrative purposes to identify the levels of government responsible for construction and maintenance; grouping according to eligibility for various sources of funding; grouping by traffic volumes for traffic operations; grouping according to geometric design for setting design



standards and priorities; and grouping by character of service provided by the road as a functional classification. Geometric design category is of interest to design engineers and the lowest category for residential, commercial, or industrial access is one of particular interest to land planners for development projects.

The American Association of State Highway and Transportation Officials (AASHTO) handbook *A Policy on Geometric Design of Highways and Streets 1990* includes a detailed discussion of functional highway systems. AASHTO describes a typical trip as including the following six stages of travel movement or service to facilitate vehicular movement: (1) main movement, (2) transition, (3) distribution, (4) collection, (5) access, and (6) termination. The roadway functions and traffic volumes are directly related to the movement hierarchy served. Figure 20.1 illustrates the hierarchy of movement and depicts the use of roadways within a typical trip. In many land development projects the majority of the planning and design focuses on the roads pertaining to the latter parts of the travel movement; i.e., terminal access, collection, and occasionally distribution.

The highway and street network provide travel mobility and access to property. Mobility is an all encompassing description that includes riding comfort, absence of speed changes, operation speed, and travel time. At one end of the functional category is the high-access, low-mobility local roads and streets. The other extreme is the high-mobility, low-access arterial highways such as freeways.



**FIGURE 20.1** Hierarchy of movement. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Between these two extremes are the collection and distribution roads that link the two functional extremes.

Functional classification is divided into rural and urban systems, which possess different vehicle and traffic characteristics. The systems differ in travel patterns, the density of the streets and highways within the network, and the type of land served by the network. As might be inferred, the population density sets the distinction. Comparatively dense areas, as designated by state or local officials, with populations of 5000 or more people are considered urban. Population is used to further divide this category into two subcategories: *urbanized areas*, those with populations of 50,000 and over, and *small urban areas*, defined by a population between 5000 and 50,000. As shown in the AASHTO handbook, the urban functional system divides streets into principal arterial, minor arterial, collector, and local functions.

The urban principal arterials provide service to major centers of activity and are the corridors that accommodate the highest traffic volume. A high proportion of the urban area travel occurs on the limited arterial system. In the urbanized areas it is these streets that provide linkage to the rural arterial roadways. The principal arterial system is composed of several types of roadways with full control of access, such as interstate highways and other freeways, and other principal arterial streets with partial or no control of access. The spacing of these principal arterials varies with the density of the urban areas.

Next in the hierarchy of the urban functional system is the minor arterial street system. These distributor streets provide greater access to adjoining land than that afforded by the principal arterial system. Traffic movement, as compared to the principal arterial system, is greatly impeded. Minor arterial streets provide travel between communities and possible connections to rural collector roads. Typically, such roads provide the boundaries of identifiable neighborhoods. Their spacing relates to the density of the developed area. Spacing ranges from  $\frac{1}{2}$  mile to 3 miles. In fully developed areas the spacing normally does not exceed 1 mile.

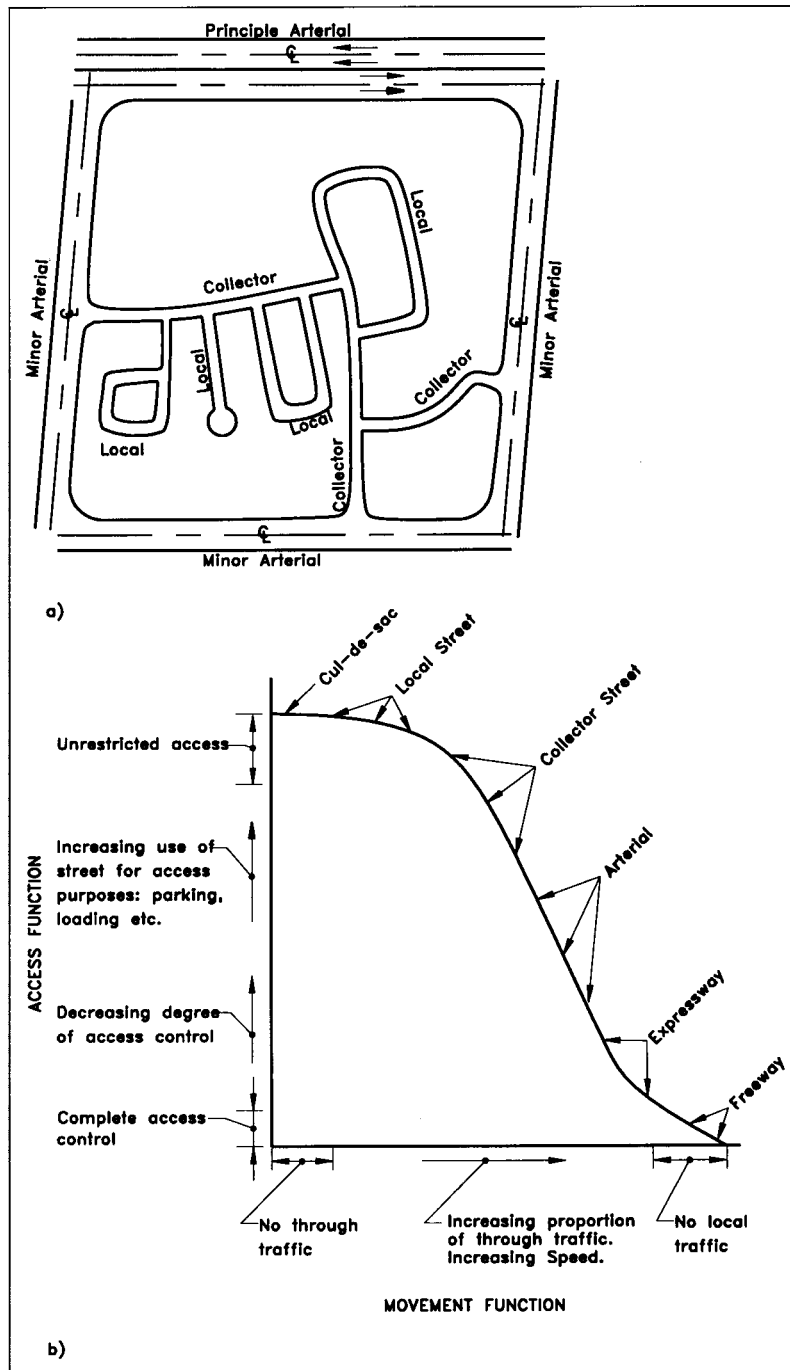
The urban collector street system provides for land access and traffic flow in all land use areas. These streets link the local street system with the principal and minor arterial street systems. Collector streets through predominantly residential areas serve as the main access to the development from the arterial or another collector. The residential collector street has average daily traffic (ADT) counts of 1000–3000 vehicles per day (VPD). Single-family dwelling units may have direct access on these roads where traffic volumes are on the lower end of this range. However, as traffic volumes increase, perhaps due to the length of the collector or the land use it serves, direct access becomes less desirable and left turn movements are often restricted. An option to consider is to design the development project such that the collector accesses multifamily residential dwelling units with off-street parking such as townhouses and condomin-

iums, schools, and other public facilities. As the collector street traffic volumes approach 3000 VPD, access to single-family units becomes undesirable; however, access to small public and local commercial/retail facilities would still be considered appropriate.

Lowest in the hierarchy is the urban local street system. The main function of the local street is access to the land adjoining the roadway and linkage to streets mentioned pre-

viously that are higher in the order within the functional system. Whereas collector streets are the main routes that provide access to the development, local streets provide access to the properties within the development. In residential areas, the volume of local streets is usually less than 1000 VPD.

Figure 20.2(a) schematically shows the functional classification of streets and the interconnection of the street



**FIGURE 20.2** (a) Functional classification at the subdivision scale; (b) Access versus movement. From Institute of Transportation Engineers: *Residential Street Design and Traffic Control*, © 1989, p. 20. Reprinted by permission of Prentice Hall, Englewood Cliffs, NJ. And from DeChiara, J., and L. Koppelman. 1984. *Time Saver Standards for Site Planning*. New York, New York: McGraw-Hill. Reproduced by permission of McGraw-Hill.

hierarchy. For example, it is impractical to connect a local street to a principal arterial. Figure 20.2(b) relates movement to access for each of the functional classifications. A summary of the characteristics of the AASHTO classification appears in Table 20.1.

Other organizations besides AASHTO have developed a functional system that relates to residential and suburban settings. The National Association of Home Builders, the American Society of Civil Engineers, and the Urban Land Institute formed a task force to assimilate data and publish a manual to aid in appropriate local street design. This manual entitled *Residential Streets*, 2nd ed. (1990), defines the street categories as arterial, collector, subcollector, and access. Accordingly, residential streets are included in all groups except arterial. The definition of arterial and collector streets correlates to the AASHTO definition. A subcollector street acts as a partial-collector, partial-access street by providing access to lots and carrying through traffic to the lesser-access street. The access street carries no through traffic. It serves only to access the lots located along it. Streets such as cul-de-sacs, dead-end streets, loop roads, and courts fall into this category.

Ultimately for land development projects, street classification will fall into the domain of the local controlling body. Description and definition of the street categories are usually found in local subdivision ordinances and standards. In all likelihood the categories will be defined according to function and in part by anticipated traffic volumes. Regardless of the basis for grouping, categories should follow a pattern similar to those outlined above.

Placing priority on function when designing local access streets requires that the designer consider many related factors apart from average daily traffic. Land development projects are only pieces in the regional development scheme. Design of streets within a project should, therefore, conform to the overall street system for the entire neighborhood. Such factors as economy, efficiency, safety, and viability lead to functionally well-designed streets.

## DESIGN CONSIDERATIONS

### Traffic Volumes

The design capacity—that is, the traffic volume that can be supported by a street—depends on factors such as width, horizontal curvature, vertical curvature, longitudinal street grades, superelevation, roadside obstructions, intersections, and traffic control measures. These criteria are usually specified as minimums for residential access streets and are determined by the need for fire access and whether on-street parking is permitted. For collectors and arterials, it is necessary to design the street for the traffic it is expected to maintain at some future date.

A traffic impact analysis performed during the early planning stages provides a basis for design. The traffic study includes estimates of additional traffic generated based by the new development. The study's level of detail and the methods used for projecting traffic volumes depend on the type of project, its size, and its location. Of interest in the design are the traffic volumes in terms of peak hour volume and average daily traffic. The projected peak hour traffic volume dictates street geometry.

Methods used for estimating traffic volumes on local and collector residential streets may be different than those used for streets higher in the functional hierarchy or for commercial/industrial projects. Whereas the higher-order streets require detailed traffic analysis and future traffic volume projections, volumes on residential streets are typically based on averages of vehicle trips per type of dwelling unit. Table 20.2 provides estimates on the number of trips generated for various land uses.

The trip rates shown in Table 20.2 are “trip-ends.” A trip-end is “a single or one direction vehicle movement with either the origin or destination (exiting or entering) inside a study site.”<sup>1</sup> For example a person leaves home for work

<sup>1</sup> *Trip Generation* 5th ed., Institute of Transportation Engineers, Washington, D.C., 1991.

**TABLE 20.1 Summary of Characteristics of the Functional System**

STREET CLASSIFICATION	SERVICE AREA	RELATIVE VOLUME	LOS*/OPERATING SPEED	ACCESS CONTROL
Principle Arterial	Through movements; Major activity centers	High	High	Full to Partial
Minor Arterial	Lesser activity centers	High to Medium	High to Moderate	Some limitation
Collector	Regional and some local land access	Medium to Low	Moderate to Low	None
Local	Neighborhood land access	Low	Low	None

\*Level of Service—a qualitative measure of the operating conditions

**TABLE 20.2 Weekday Vehicle Trip Generation for Selected Land Uses (given as trip-ends)**

Single-Family Detached	9.6/DU
Residential Planned Unit Development	7.4/DU
Residential Condominium/Townhouse	5.8/DU
Apartment	6.5/DU
Mobile Home Park	4.8/DU
General Office Building	
10,000 ft <sup>2</sup> Gross Floor Area	24.6
50,000 ft <sup>2</sup> Gross Floor Area	16.6
100,000 ft <sup>2</sup> Gross Floor Area	14.0
200,000 ft <sup>2</sup> Gross Floor Area	11.9
500,000 ft <sup>2</sup> Gross Floor Area	9.5
Shopping Center (excluding Christmas season)	
100,000 ft <sup>2</sup>	70.1
500,000 ft <sup>2</sup>	38.7
1,000,000 ft <sup>2</sup>	32.1
Business Park	14.4/1000 ft <sup>2</sup> Gross Floor Area
Industrial Park per employee	3.3/employee
County Park	3.0/acre

(Source: ITE, TRIP Generation 1991)

and returns later in the day constitutes *two* trip-ends for that dwelling unit. This *Trip Generation* manual contains trip rates for nearly every type of land use. The reader should refer to this manual to estimate trip generation in more specific detail.

Depending on the nature of the project, its location relative to the transportation network, and other factors, a traffic analysis would include data such as current daily traffic volumes, existing peak hour turning volumes, estimated site traffic generation, directional distribution of site traffic, estimated trip generation for nonsite development, estimated total future traffic, and projected levels of service. Part of the street design, such as number of lanes, turn lanes, and intersection control would be based on this data. Figure 20.3 is an example from a traffic study showing impact of a new project on the traffic.

The traffic study typically projects the volume for four types of traffic:

- *Existing traffic*—volume of traffic vehicles that are using the road prior to improvements and modifications

- *Future background traffic*—increase in traffic expected to occur at the time of development (not including traffic generated by new development)
- *Site traffic*—traffic explicitly generated by new development project
- *Total future traffic*—sum of the future background traffic and site traffic

Traffic volume is a result of traffic demand. Land use, socioeconomic conditions and the amount and type of available transportation affect travel demand. Land use dictates the location and intensity of the activity, and socioeconomic factors determine the magnitude and extent of population activity. The two factors are integral components in the development of trip generation models.

Trip generation models predict how many trips each activity produces and the origin and destination of such trips. Forecasting travel demand with elaborate trip generation models or with tables, such as Table 20.2, depends on the level of detail required for the study. The travel demand analysis is a key element of transportation planning,

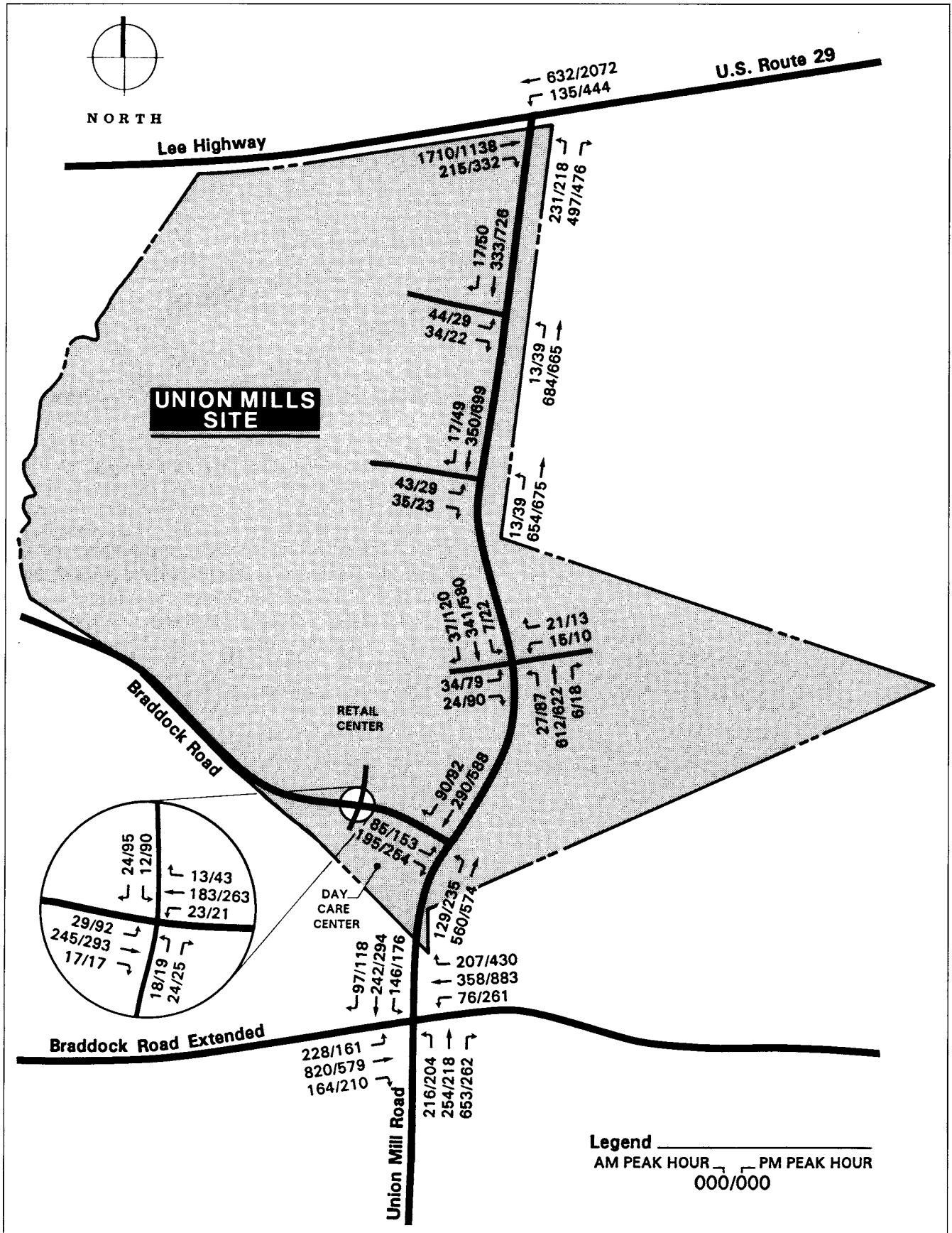


FIGURE 20.3 Peak hour traffic for a proposed development. (Courtesy of Barton-Aschman Assoc., Inc. Washington, D.C.)

whether for long-range wide-area comprehensive plans or for short-range plans for a traffic corridor or specific facility.

Transportation impact analysis for various types of land development projects above a certain size, when required, are normally performed by traffic engineering consultants. Traffic volumes are necessary to determine the impact a development project has on existing streets. New development generates more traffic and, consequently, may alter prevailing traffic patterns. Traffic volumes fluctuate daily and seasonally depending on the area served by the street. Major corridors serving commercial and business areas have peak volumes in morning and late afternoons. Retail areas have peak volumes on weekends and evenings, especially during holidays. Many local agencies require off-site construction of financial contributions to alleviate the impact of development on nearby collector and arterial streets.

Two parameters frequently referenced in the design of major roads are the average daily traffic (ADT) and design hourly volume (DHV). The ADT is the traffic volume counted over a number of 24-hr periods divided by the number of 24-hr periods. The ADT is a parameter used to measure and evaluate existing traffic flow and to plan for future roads. ADT volumes do not reflect the specific variation in volumes during peak hours. Design for the expected greatest peak volume would not be cost effective. Therefore a cost-effective design must be one based on a traffic volume that may be only occasionally exceeded.

The DHV is the projected hourly volume used for setting the geometric design. It provides a base to account for change in demand on a roadway over a projected design period. Data collected from rural arterial streets for a wide range of traffic volumes over a wide geographic area has shown that there are approximately 30 hours in a year where the road carries the highest hourly volume (HV). The traffic volume for the remaining hours of the year is nearly uniform relative to the 30 highest hours. Based on these results, the 30th highest hourly volume for the projected design year is determined as the DHV. That is, the projected hourly volume of traffic will be greater than the DHV only 29 hours in the year. In urban areas the 30th HV is selected as the DHV as a general rule. However, unique land use and other special situations such as major seasonal recreation areas may warrant deviations from this general rule. In many instances the DHV is taken as a percentage of the ADT. On urban highways the DHV is normally between 8 and 12% of the ADT.

### Driver Characteristics

The characteristics of vehicles and drivers using the roadway directly affect the geometric design. Driver behavior and response to various situations vary as conditions vary. Behavior and response for the same situation and condition vary for the same driver at different times due to physical condition and mental attitudes. Likewise, the performance of vehicles varies due to size, weight, power, mechanical condition, and the driver. Road design must account for

some minimum standard of driver behavior and vehicle performance.

Individuals vary, and therefore, as an added measure of safety, designers must design for below-average driver characteristics, with the presumption that a high majority of drivers will have these minimum threshold characteristics. Drivers' fundamental characteristics include perception, identification, emotion, and volition (PIEV).

Perception includes seeing and observing objects. Identification consists of comprehending an encountered object and its surrounding elements. Emotion involves judgment and the process of decision-making leading to responsive actions. Volition or reaction is the driver's will to react to or execute the actions decided during the emotion process. The total time elapsed through the PIEV process is referred to as the PIEV time, or more commonly the perception–reaction time.

PIEV time varies with the complexity of individual situations. Factors such as the use of drugs and alcohol, age, fatigue, and physical impairments cause increases in PIEV time. Studies conducted in laboratory situations determined PIEV time as lying in a range from 0.5 to 7.0 seconds. Highway agencies typically use PIEV times of 2.5 seconds for stopping distance and 2.0 seconds for determination of adequate sight distance at intersections. The latter is shortened because the driver anticipates a possible reaction at intersections. Similarly, familiarity with common signs and traffic markings reduces PIEV time because perception, identification, and emotion occur almost instantaneously.

### Vehicle Characteristics

Roads are designed to accommodate the characteristics of the vehicles using it. There is a wider range of vehicle types using arterial roads as compared to vehicle types using local streets. In addition, the frequency of occurrence of specific vehicle types is higher on arterial roads. For example, a large truck (e.g., moving van) is only occasionally found on residential streets but large trucks are common on arterial streets. Therefore, for economic reasons, streets higher in the functional hierarchy must be geometrically and structurally designed to accommodate larger vehicles. In comparison, the design requirements for local streets may not have to be as stringent. For example, lane widths for local streets may not be as wide as arterials, and curve radii may be shorter and gradients steeper. Local requirements for pavement structure and operating room for fire apparatus often provide more than adequate design for other similar vehicles.

The largest vehicle type that is most likely to use the roadway with regular frequency is selected as the design vehicle. The physical and operating characteristics of the design vehicle are then used to establish critical geometric and structural design features of the road. Local streets, like any roadway, must accommodate the predominant vehicles using this type of functional street. The predominant vehicle on local streets is the private passenger car, including

small-, medium-, and large-size cars, vans and pickup trucks. In high-density developments (e.g., townhouses, condominiums), the travelways are typically not publicly-owned and maintained. Consequently, design criteria for minimum widths and radii of curvature are even less than that prescribed for local public streets. However, the design engineer must consider the use of these streets and travelways by emergency vehicles, moving vans, refuse trucks, school buses, and other large vehicles. Table 20.3 shows the acceptable dimensions of design vehicle types and the designation for each vehicle type commonly used in traffic engineering (e.g. an intermediate semi-trailer is designated as a WB-40).

### Circulation and Access

The traffic circulation and the access often determine a land development's consumer appeal. In purely residential developments, the recommended practice is to discourage outside-generated through traffic by limiting the number of access points to the arterials or collectors or by designing the internal streets in short, narrow, curvilinear, discontinuous patterns. Although multiple access points might encourage through traffic, they do provide for less congestion because of more dispersion of the traffic from within the residential development. In mixed-use developments and commercial/retail projects, the access and circulation pattern is more complex. Circulation and access is important to facilitate the traffic flow through the commercial and retail sections. However, the commercial and retail traffic should not be directed through residential areas. Proper planning of the development necessitates that the residential section be located apart from the business areas to reduce the high traffic volume's impact. As previously mentioned, through traffic can be discouraged by street geometry and configuration, but this strategy is most effective if the alternative minor arterials have adequate capacity and provide a good level of service.

Street circulation is site oriented to a degree. Where and how the streets are oriented depend on the interrelationships of various aspects such as lot layout, topography, noise levels, accident potentials, and other factors. To accommodate access and circulation demands, street networks have evolved into patterns that fit certain situations and lend themselves to particular solutions. The *grid pattern* is a series of parallel streets intersecting at right angles creating rectangular blocks. The grid is best suited for use in high-density areas with widely distributed traffic flows. Grid systems provide easily recognizable orientation for users. Their simplicity in providing access is a definite advantage. The grid system simplifies surveying and maximizes the number of houses fronting on a street.

Grid street systems dictate the topography by forcing a specific grading design in order to obtain buildable lots. Therefore, they are best suited for generally flat areas. Imposing a grid system on rolling terrain in most cases requires extensive grading. Other drawbacks to this type of

system are the monotony of the layout and the lack of differentiation of function between individual streets within the system. The system also invites through traffic. These drawbacks can be overcome with imagination and by variations from the typical grid. Figure 20.4 illustrates the simple grid pattern and modifications such as using secondary loop roads to provide a more attractive environment.

**Curvilinear Patterns.** Many medium to large projects develop a curvilinear type of street pattern. Such a pattern discourages "cut-through" traffic, optimizes land use and minimizes cut and fill grading operations during construction by taking advantage of the existing topography for streets and, also, lot orientations and configurations. Such patterns create a sense of neighborhood on a small scale by collecting the lots into small groupings through the use of smaller elements of street patterns such as cul-de-sacs, loops courts, and short streets (see Figure 20.5). The combined use of the smaller street elements with curvilinear street patterns is evident in the aerial photograph of Figure 40.22. Such a pattern's drawback is the confusion created that may result when attempting to traverse the area. Extensive curvilinear patterns are not recommended for commercial and industrial developments.

Circulation and access may be considered from two different scales as they apply to the development's proposed public street system and as they apply to a specific land parcel. In both situations, circulation and access are constrained by the existing street network, adjacent property, and the intended type of land use for the proposed development.

Frequently, providing for adequate circulation and access requires improvements to existing infrastructure elements beyond the limits of the site, and these are often financed by the developer. Turn lanes added to the existing street or signalization of an intersection to improve capacity and safety are examples of such required offsite improvements. Necessary considerations for circulation and access include the following:

- Will the intersection of the proposed street adversely impact existing intersections?
- Is there adequate horizontal and vertical sight distance at the new access point?
- Will turn lanes be required?
- Will any intersections require signal control?
- Will any streets within the development be required to terminate at a particular point on the boundary to allow for access to adjacent properties for future development?

Considerations for access to a particular parcel such as entrances (driveways) into commercial/retail sites should address:

- An adequate driveway length (i.e., throat length) to allow for vehicle queuing into and out of the parcel

TABLE 20.3 Design Vehicle Dimensions

DESIGN VEHICLE TYPE	SYMBOL	DIMENSION (FT)												
		OVERALL			OVERHANG				WB <sub>1</sub>	WB <sub>2</sub>	S	T	WB <sub>3</sub>	WB <sub>4</sub>
		HEIGHT	WIDTH	LENGTH	FRONT	REAR								
Passenger car	P	4.25	7	19	3	5	11							
Single unit truck	SU	13.5	8.5	30	4	6	20							
Single unit bus	BUS	13.5	8.5	40	7	8	25							
Articulated bus	A-BUS	10.5	8.5	60	8.5	9.5	18			4 <sup>a</sup>	20 <sup>a</sup>			
Combination trucks														
Intermediate semitrailer	WB-40	13.5	8.5	50	4	6	13	27						
Large semitrailer	WB-50	13.5	8.5	55	3	2	20	30						
"Double Bottom" semi-trailer—full-trailer	WB-60	13.5	8.5	65	2	3	9.7	20		4 <sup>b</sup>	5.4 <sup>b</sup>	20.9		
Interstate Semitrailer	WB-62*	13.5	8.5	69	3	3	20	40–42						
Interstate Semitrailer	WB-67*	13.5	8.5	74	3	3	20	45–47						
Triple Semitrailer	WB-96	13.5	8.5	102	2.5	3.3	13.5	20.7	3.3 <sup>d</sup>	6 <sup>d</sup>	21.7	21.7		
Turnpike Double Semitrailer	WB-114	13.5	8.5	118	2	2	22	40	2 <sup>c</sup>	6 <sup>c</sup>	44			
Recreation vehicle														
Motor home	MH		8	30	4	6	20							
Car and camper trailer	P/T		8	49	3	10	11	18	5					
Car and boat trailer	P/B		8	42	3	8	11	15	5					
Motor home and boat trailer	MH/B		8	53	4	8	20	21	6					

(From *A Policy on Geometric Design of Highways and Streets*, Copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. used by permission.)

\* = Design vehicle with 48' trailer as adopted in 19872 STAA (Surface Transportation Assistance Act)

\*\* = Design vehicle with 53' trailer as grandfathered in 1982 STAA (Surface Transportation Assistance Act)

<sup>a</sup> = Combined dimension 24, split is estimated.

<sup>b</sup> = Combined dimension 9.4, split is estimated.

<sup>c</sup> = Combined dimension 8, split is estimated.

<sup>d</sup> = Combined dimension 9.3, split is estimated.

WB<sub>1</sub>, WB<sub>2</sub>, WB<sub>3</sub>, WB<sub>4</sub> are effective vehicle wheelbases.

S is the distance from the rear effective axle to the hitch point.

T is the distance from the hitch point to the lead effective axle of the following unit.

- Is the driveway entrance too close to other driveway entrances or to an intersection that causes conflicts in turning movements?
- If there is a median in the driveway, is it visible by the vehicles turning into the entrance?
- Are the profile and cross section of the driveway adequate and compatible with the roadway conditions?

Consider circulation within the parcel:

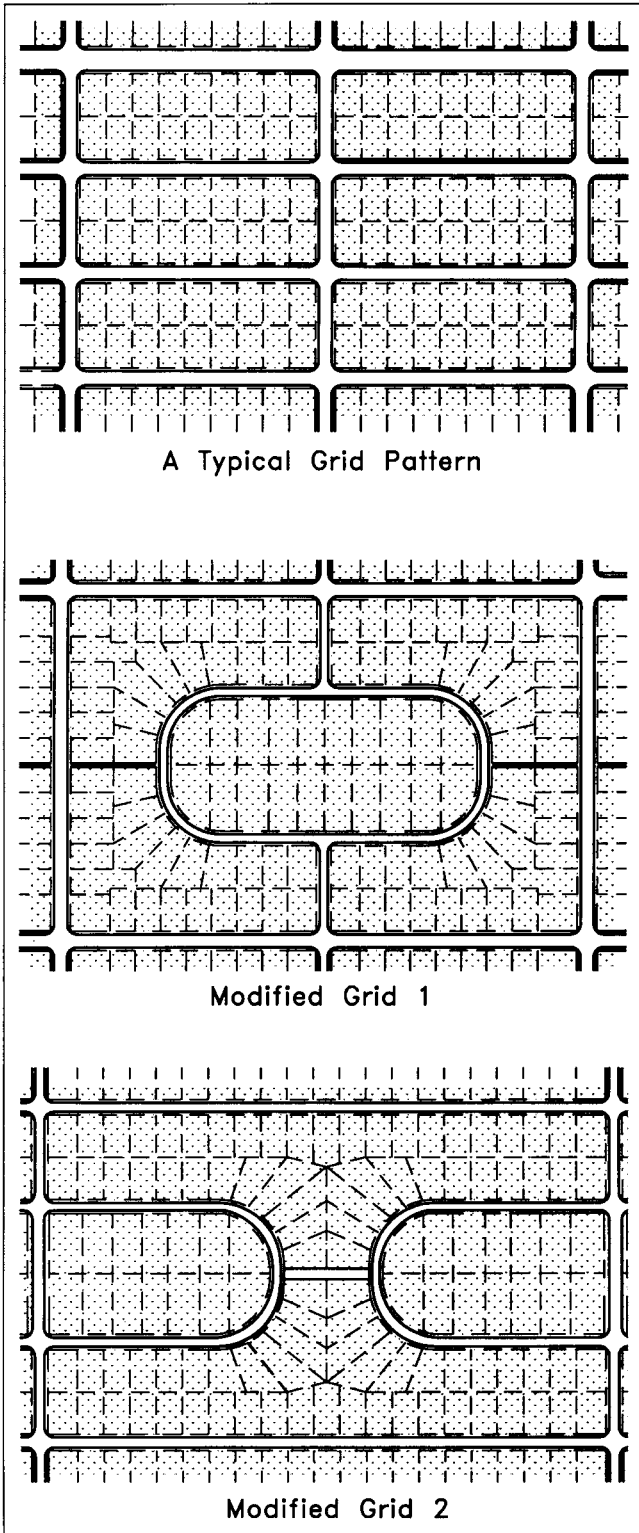
- Is a loop road or perimeter road appropriate?
- Does the parking area need to be partitioned or somehow sectioned to facilitate traffic flow?
- Are the landscaped islands adequately sized (e.g., radius and widths) and located for visibility and circulation?

## Pedestrians

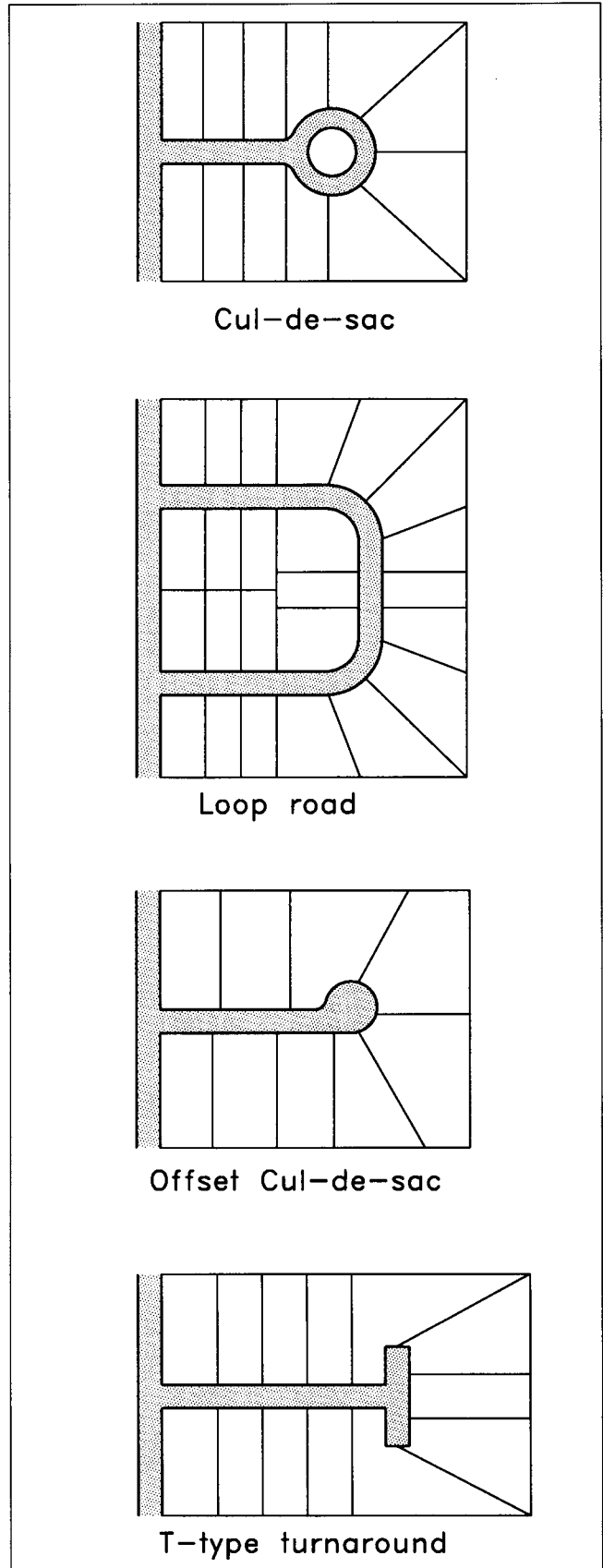
Pedestrian safety is a concern on any roadway where a combination of vehicular traffic and people occur. Pedestrian density substantially increases in the lower classifications of streets in both residential and commercial areas. In residential areas there is an added emphasis on child safety. Not to be overlooked is the safety concerns of pedestrians in parking areas. Parking areas are complex in that the driver, although proceeding slowly, must contend with numerous vehicle and pedestrian movements occurring nearly simultaneously. Pedestrians' movement along and randomly crossing travelways, vehicles entering and exiting parking spaces, as well as the movement and turns of the normal traffic, all contribute to parking area hazards.

Planning for the pedestrian is a high priority in the design of roadways at the local level. Pedestrian safety provisions included in roadway planning are sidewalks, crosswalks, curb cuts, and ramps for the physically disa-





**FIGURE 20.4** Grid pattern and variations. (DeChiara J, Koppelman L: *Time Saving Standards for Site Planning*, 1984, McGraw-Hill. Reproduced with permission of McGraw-Hill.)



**FIGURE 20.5** Smaller elements of street patterns. (DeChiara J, Koppelman L: *Time Saving Standards for Site Planning*, 1984, McGraw-Hill. Reproduced with permission of McGraw-Hill.)

bled. Another such provision is special walkways for pedestrian freeway crossings. Traffic control features aid both pedestrian traffic and vehicular traffic.

Sidewalks should provide a continuous path to service areas such as community centers, schools, parks, and shopping areas. Most ordinances require sidewalks on at least one side of the street. In some residential areas the ordinance may require a paved trail in lieu of or in addition to the conventional sidewalk. Conventional sidewalks typically are adjacent and parallel to streets and travelways, whereas paved trails meander along natural pedestrian circulation routes. The most obvious indication of a sidewalk or trail deviating from natural circulation is the worn earthen path. Besides being an eyesore, it may create problems for the private owners whose property is being trespassed.

Gradients for paved trails and sidewalks should be limited to a maximum of 8 to 10% for extended lengths. In hilly areas, steep gradients are acceptable for short sections. Steps are a consideration in lieu of steep gradients. Since many paved trails are used as bike paths, for pedestrian safety, steps and steep gradients should be adequately marked with signs and possibly markings on the pavement as a warning.

Another provision is the need for landing and staging areas at bus stops. Unfortunately, bus routes through new residential developments are rarely established prior to completion of the development. Providing landings and pe-

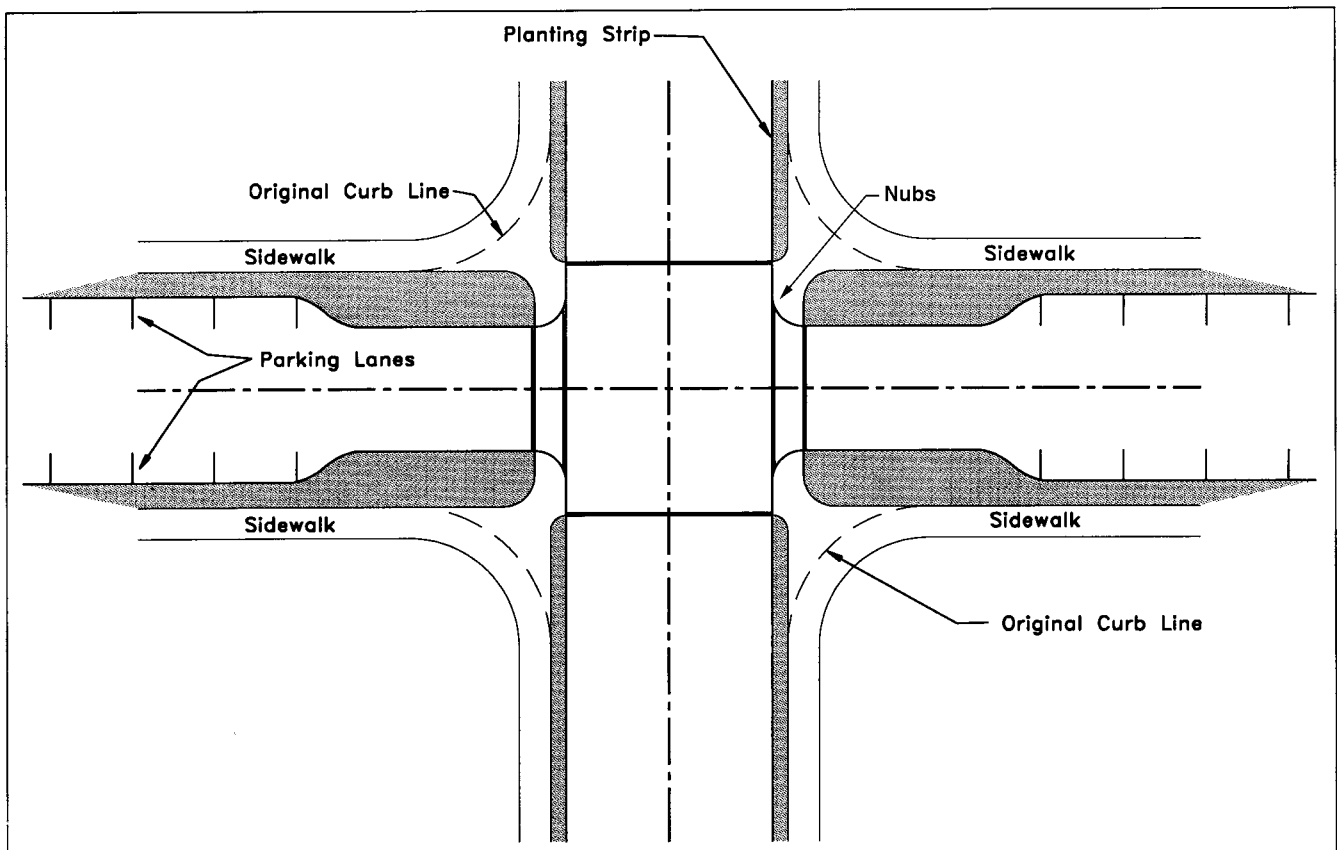
destrian circulation to these locations, when possible, adds to the appeal and safety of the land development project.

One of the most dangerous maneuvers for pedestrians is to cross a wide street at an unsignalized intersection. One option to increase pedestrian safety is to locate nubs near the ends of the street as shown in Figure 20.6. This not only reduces the pedestrian crosswalk but also creates an illusion of a narrower street to the driver, who instinctively reduces his/her speed.

### Intersections and Curb Cut Ramps

Curb cut ramps are a requirement by local ordinances at most intersections and crosswalk areas (see Figure 20.7). Most localities provide standards detailing their dimensions and intersection locations. Issues for consideration in the placement and design of curb cut ramps are:

- Curb cut ramps should not be located in sump areas where drainage inlets are located.
- Curb cut ramps are to be provided on traffic islands located within the crosswalk areas.
- Curb cut ramps should be located away from areas where parked cars will obstruct the path.
- Curb cut ramps should be located at best pedestrian sight distance location.



**FIGURE 20.6** Nubs.

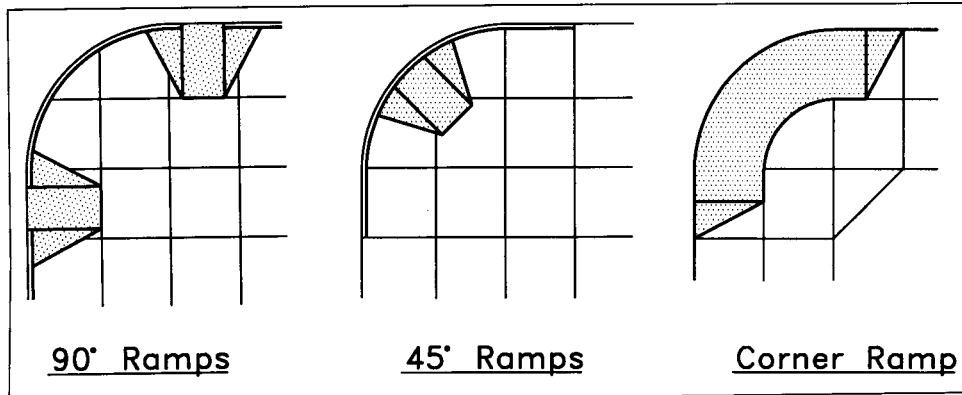


FIGURE 20.7 Curb cut ramp locations.

Typical curb cut ramp details and recommended dimensions are shown in Figure 20.8

**Stopping Site Distance**

Stopping sight distance (SSD), a major design consideration, is the minimum distance needed for a driver to react and safely cause the vehicle to stop before reaching an ob-

ject. Besides providing adequate stopping site distance on horizontal curves, SSD also must be considered on vertical curves and at intersections. These topics will be addressed later in this chapter.

The minimum stopping distance is the summation of the distances the vehicle travels while a driver is reacting to a situation and the distance the vehicle travels from the mo-

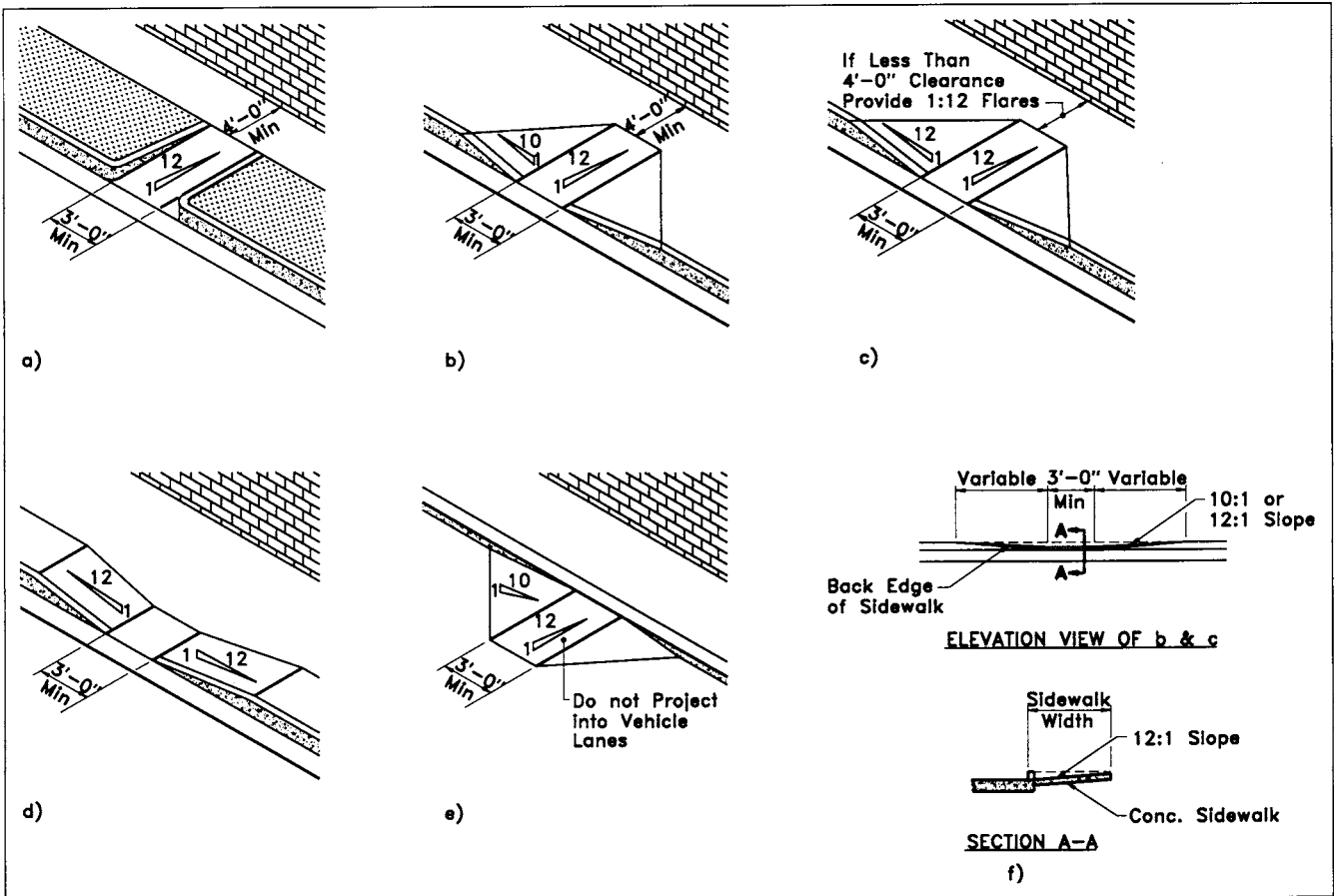


FIGURE 20.8 Curb ramps. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

ment its brakes are applied, until it is completely stopped. The first distance segment is a function of the reaction time of the driver. For most circumstances, the reaction time is assumed to be 2.5 seconds. However, this assumed increment increases in more complex situations. The second distance segment, the braking distance, is subject to the vehicle speed and the friction between the tires and pavement.

The SSD (i.e., minimum stopping distance) is given in Eq. (20.1).

$$SSD = 1.47t_{\text{sec}} V_{\text{mph}} + \frac{V_{\text{mph}}^2}{30(f \pm G)} \quad (20.1)$$

where SSD = stopping sight distance (ft),  $t$  = reaction time (seconds),  $V$  = initial travel speed (mph),  $f$  = coefficient of friction between tires and road, and  $G$  = longitudinal street slope (ft/ft). The first term on the right side of the equality represents the distance the car travels during the reaction time. The next term represents the braking distance. The  $\pm$  (plus/minus sign) in front of the  $G$  term accounts for the direction of the vehicle on a longitudinal slope. A + (plus sign) is used for a vehicle traveling uphill and the - (minus sign) is used for vehicles in the downhill direction. Uphill grades reduce the stopping distance, while the reverse is true for downhill grades. On two-way streets the controlling stopping distance is the downhill direction.

Table 20.4 summarizes the required stopping site distance on wet pavements. Due to the reduced friction between tires and wet pavement, the coefficient of friction for wet pavement is used as the governing parameter for setting adequate stopping site distances.

### CROSS-SECTIONAL ELEMENTS FOR LOCAL STREETS

Cross-sectional elements of a road include pavement, travel lanes, shoulders, medians, side slopes, curb and gutters, drainage ditches (median and side ditches), sidewalks, and right-of-way widths. The specific elements included in any road depend on the function and the particular locality. Design of such elements entails establishing the widths, depths, and cross slopes. These cross-sectional elements and their dimensions are identified on a detail drawing called the typical cross section.

It is possible for a project to require several typical cross sections. The cross-sectional elements may change to account for changes in lane additions, lane widths, super-elevation, and other factors. Each detail should identify the location of the typical section by specifying the beginning and end stations. Typical section details are used by the surveyor to set the road, by the contractor to construct and by the design engineer to plot cross-section drawings for computing earthwork quantities. Public review agencies use the typical section details to determine that the road design conforms to the local standards.

Many factors influence the cross-sectional design of local streets. These include the density of the development, avail-

ability of off-street parking, family demographics, availability of shopping, and public services. The type of local street is also a major consideration. For the lower hierarchy streets, many of the cross-section elements and their dimensions are usually included as part of the subdivision ordinance.

### Profile Grade Line

One important element identified on the typical section drawings is the *profile grade line* (PGL), an arbitrarily selected reference point showing the proposed street elevations. In the profile view, the PGL appears as a line; in the typical section, the PGL appears as a point (labeled as P in Figure 20.13). The PGL is selected for the convenience of design, stakeout, and construction. It can be the centerline of the road, edge of pavement, top of curb, or any other designated point as long as the horizontal and vertical relationship between the selected point and other elements of the typical section are known.

### Lanes

The local subdivision ordinance typically specifies the minimum number of lanes and their widths for a street, and this is a function of the anticipated traffic volume. Traffic volume is directly related to land use. Therefore, local design standards may categorize the streets according to the number of dwelling units it serves. In commercial or retail areas, traffic volume may be related to the number of square feet of the retail/commercial neighborhood. Therefore, street lanes and their widths could be indirectly related to the size of the building.

To provide a minimum level of service<sup>2</sup> on a local street, the design must incorporate one unobstructed moving lane. As the development density of the area increases, so must the number of travel lanes. The length of travel also impacts on the number of travel lanes needed to maintain a satisfactory level of service. Local streets are usually short, and some inconvenience to users of the street is acceptable. Additional lanes are required in those cases where on-street parking is allowed. However the width of the parking lane is narrower than the travel lane. Some general guidelines require parking lane widths on local streets to be 8 ft, while collector streets require an additional 1 ft of width in these lanes. Both collector and local streets should provide a minimum of 10-ft lane widths for moving traffic. Where only two lanes are proposed, a 22-ft minimum is the requirement of most jurisdictions. Considering these recommendations, lane widths on local streets vary from 22 to 24 ft (not including the parking lane). A typical range for collector street lanes with no parking is 22 to 26 ft.

<sup>2</sup> Level of service (LOS) is a qualitative measure of the operating conditions of the street. Oversimplified, it is a measure of the freedom of movement.

TABLE 20.4 Stopping Sight Distance (Wet Pavements)

DESIGN SPEED (MPH)	ASSUMED SPEED FOR CONDITION (MPH)	BRAKE REACTION		COEFFICIENT OF FRICTION F	BRAKING DISTANCE ON LEVEL (FT)	STOPPING SIGHT DISTANCE	
		TIME (SEC)	DISTANCE (FT)			COMPUTED (FT)	ROUNDED FOR DESIGN (FT)
20	20–20	2.5	73.3–73.3	0.40	33.3–33.3	106.7–106.7	125–125
25	24–25	2.5	88.0–91.7	0.38	50.5–54.8	138.5–146.5	150–150
30	28–30	2.5	102.7–110.0	0.35	74.7–85.7	177.3–195.7	200–200
35	32–35	2.5	117.3–128.3	0.34	100.4–120.1	217.7–248.4	225–250
40	36–40	2.5	132.0–146.7	0.32	135.0–166.7	267.0–313.3	275–325
45	40–45	2.5	146.7–165.0	0.31	172.0–217.7	318.7–382.7	325–400
50	44–50	2.5	161.3–183.3	0.30	215.1–277.8	376.4–461.1	400–475
55	48–55	2.5	176.0–201.7	0.30	256.0–336.1	432.0–537.8	450–550
60	52–60	2.5	190.7–220.0	0.29	310.8–413.8	501.5–633.8	525–650
65	55–65	2.5	201.7–283.3	0.29	347.7–485.6	549.4–724.0	550–725
70	58–70	2.5	212.7–256.7	0.28	400.5–583.3	613.1–840.0	625–850

(From *A Policy on Geometric Design of Highways and Streets*, Table III-1, p. 120, AASHTO, Washington, D.C., 1990).

### Cross Slope

A “normal crown” section street has a high point in the middle (usually at the centerline) with the pavement cross slopes extending toward the outer edges of the pavement. In most conditions the cross slopes are uniform across the total width of one direction of the pavement. However, for multilane roads, the cross slope remains uniform across a lane width, and any changes in the cross slope should occur at the edges of a travel lane.

For many of the minor streets (i.e., local and collector), cross slopes for lanes on a normal crown section street are typically  $\frac{1}{4}$  in. per foot (2.08%) sloped towards the outer edge of pavement. This cross slope is sufficient to direct stormwater runoff into the gutters or to the side ditches and yet does not cause any significant driver discomfort because of the slight tilt of the vehicle. Cross slopes change when superelevation requirements are incorporated into the design.

### Medians

Medians are used to increase the separation distance between opposing traffic and also to provide refuge for pedestrians. In urban areas, medians may be raised concrete curbs approximately 4 ft wide. Wider median strips are typically grass strips with a ditch. In rural areas and for high-speed roads, medians can be 30 ft and wider. Medians are typically not used on local streets and are only occasionally

used on collector streets. Their use is mostly in commercial areas where the traffic volumes are much higher and turning movements are more frequent and complex.

Design considerations include the spacing of median breaks. Median breaks allow traffic turning and crossover movements into and out of entrances, essentially providing more accessibility. Additionally, median breaks are necessary for most intersecting streets. Guidelines for spacing between median breaks are provided by the governing public agency. These guidelines may only prescribe minimum distances for ordinary conditions and may be modified by the reviewing agency to accommodate the specific conditions. Each jurisdiction has minimum spacing requirements. This spacing is subject to the traffic volume on the through street, traffic volume making the turning movement, whether the turning movements are signaled and the design speed of the through street. The design engineer must assess the impact of the median openings on the level of service (LOS) of the through street. A street with a higher number of median openings and crossovers can significantly reduce the LOS, especially during peak volumes of traffic.

The minimum spacing of median breaks can be viewed as the sum of the dimensions of median features that are required between two intersections. These elements include setback of the median noses to allow for proper turning radius at the intersections, length of each of the required left turn lanes (especially when the median is of such nar-

row width that the turn lanes must be back-to-back), and length of the median transition between the left turn lanes. Figure 20.9 illustrates the typical median geometry between two intersections and how it can dictate the minimum spacing between intersections and median breaks. In some cases, where a median width is approximately 28 ft or greater, the required left turn lanes may be configured such that they overlap each other for some distance, thus allowing a reduction in the overall intersection spacing.

In any event, the minimum spacing dictated by the median features discussed above must be compared to the local standard for intersection to median break spacing. The greater of the two should then be used to dictate design.

On commercial and retail projects there will be a desire to have numerous median openings to allow for land access. However, local conditions and the reviewing agency may restrict the number of openings, and this can impact the layout design of the project. The project coordinator needs to verify with the review agencies the location of the median openings early in the planning stage (see Figure 20.9).

Another situation occasionally overlooked is when a site has frontage on an existing road with a median. In developing the site, a median break is anticipated in order for the opposite side traffic to access the site, only to discover that the reviewing agency will not allow a median break at that location. Again, the project coordinator needs to verify such requirements with the reviewing agency early in the planning stages.

### Curb and Gutter

In most urban areas, curb or combination curb and gutter streets are preferred. Their purpose is to facilitate drainage,

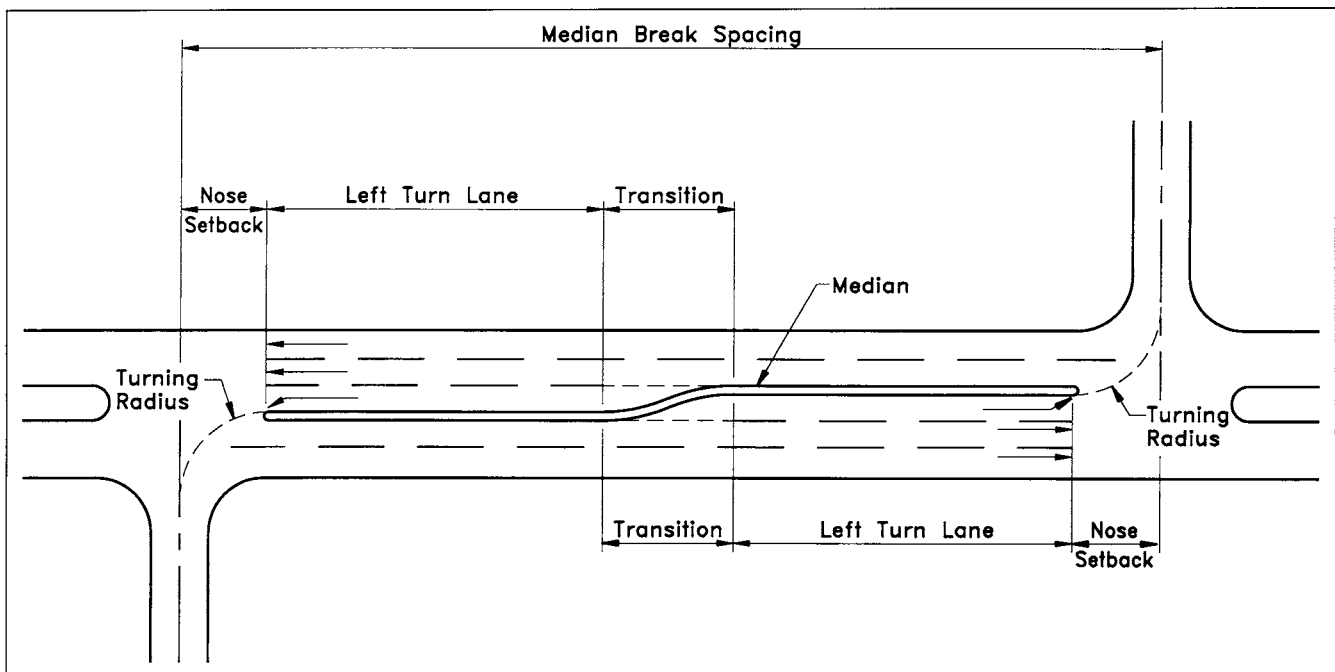
separate traffic lanes from pedestrian walkways, contribute to the aesthetic appearance, and reduce maintenance. Several different types of curbs are often utilized depending on the locality and design speeds. Figure 20.10 shows various types of curbs.

For low-design-speed roads, barrier curbs are typically used, while high-design-speed roads utilize the mountable curbs. These curbs are frequently used with traffic islands and along medians. Some localities permit the use of the combination curb and gutter where the gutter forces flow away from the face of the curb. These “spill” type gutters facilitate drainage by forcing the flow away from the curb to prevent ponding water in the gutter area. Such curbs are normally used only in parking areas where the pavement typically has a nonuniform cross slope throughout the parking area.

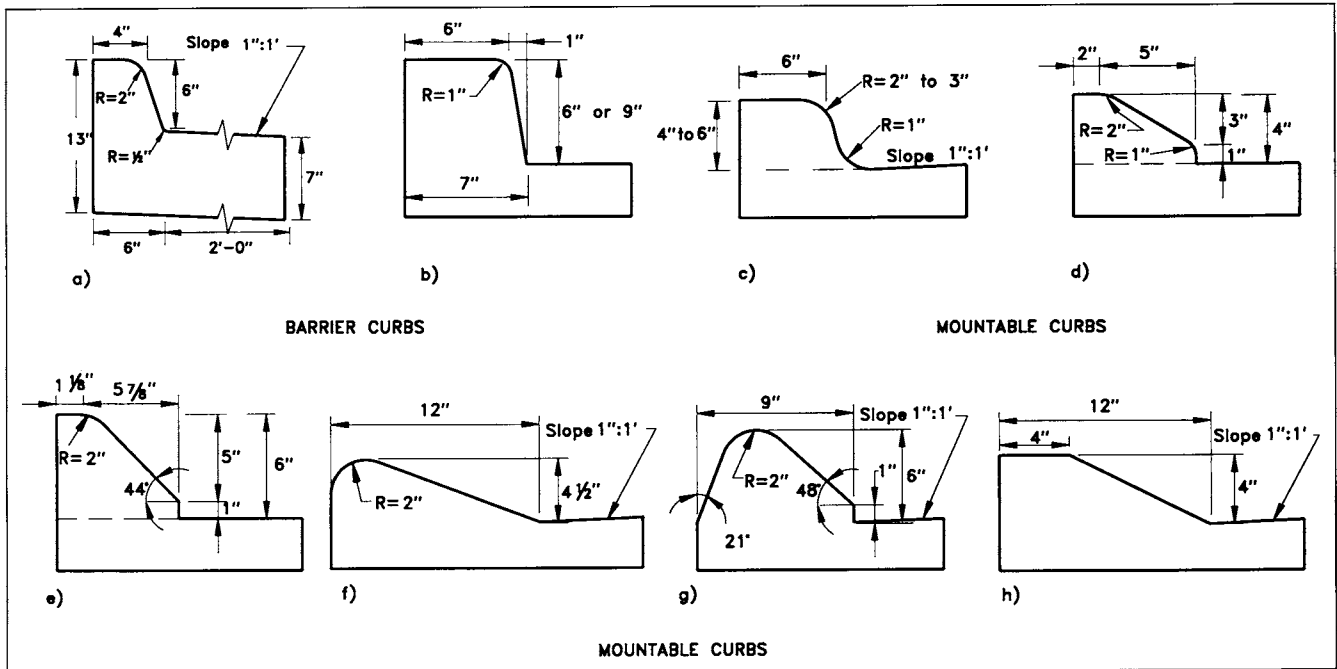
### Sidewalks and Utility Strips

Sidewalks are typically placed away from the curb near the right-of-way line. In residential developments, the area between the inside edge of the sidewalk and the curb is typically a grass strip used for placement of utility lines to avoid placing them beneath pavement. However, in commercial and retail areas, this strip is typically paved for maintenance reasons. A utility strip width of 2 to 4 ft is reasonable; however, in areas of high land values, this may not be practical. The ITE (1989)<sup>3</sup> recommends a minimum width of 5 to 6 ft to allow for tree planting.

<sup>3</sup> *Guidelines for Subdivision Residential Street Design*, ITE, Washington, D.C., 1989.



**FIGURE 20.9** Median break spacing.



**FIGURE 20.10** Common types of curb and gutter. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. (figures b through h) Used by permission.

Sidewalk widths of 4 ft are adequate in moderate-density residential areas. At locations of high pedestrian traffic, sidewalk widths of 6 to 10 ft should be considered. The sidewalk cross slope is inclined down towards the curb and should be less than  $\frac{3}{8}$  in. per foot of width. Sidewalk widths and requirements for utility strips are dictated by local standards. For economic reasons, sidewalks may only be required on one side of the street in low- to moderate-density residential areas. Frequently, no sidewalks are required around cul-de-sacs. In commercial areas, sidewalks may be required on both sides of the street.

### Shoulders, Side Slopes and Ditches

In some low-density residential areas and in most rural areas, shoulders rather than curbs are normally used. Shoulders are 6 ft or wider and typically gravel. High-speed highways will have wider shoulders and are generally paved with asphalt.

Cross slope of the shoulder is more steeped than the cross slope of the normal street section. Typical cross slopes are two to three times greater than normal pavement cross slope, on the order of  $\frac{5}{8}$  in.:1 ft. Where roads are superlevated, shoulder cross slopes are dictated by the vertical angle between the outside edge of pavement and the beginning of the shoulder—that is, the algebraic difference between the pavement cross slope and the shoulder cross slope.

Side slopes are used for connecting the road section into natural ground. The slope gradient ( $H$ ) depends on the angle of repose of the soil. In many cases the constructed

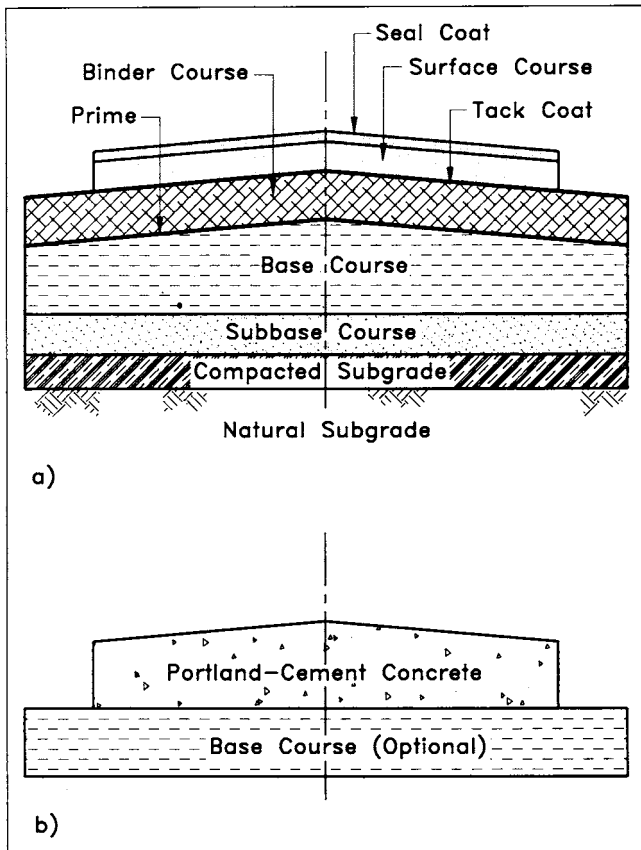
slopes are at a 2H:1V ratio. However, roads with higher design speeds ( $V$ ) may require flatter slopes. The type of mowing equipment also factors into determining the maximum slope gradients. Drainage ditches at the toe of the slope control surface water from the pavement in excavated areas or through an adjacent property along embankment areas.

### Pavement

Pavement thickness is subject to prevailing soil conditions, climate, vehicle design loadings, and type of pavement. There are two basic types of pavement: flexible and rigid (see Figure 20.11).

Flexible pavement consists of two or more of the following layers:

- *Subgrade*—the natural soil material where the upper layers rest
- *Subbase*—an optional layer used to augment the strength of the subgrade. It consists of a compacted granular material that may be treated with admixtures to achieve certain strength characteristics required in specific conditions. Additionally, the subbase acts to reduce frost action or as a filter to prevent intrusion of fine-grained roadbed soils into the base course.
- *Base course*—constructed on top of the subbase or subgrade. It consists of granular material such as crushed stone or gravel. The specifications for the base course are more rigorous than that of the subbase in



**FIGURE 20.11** Typical pavement sections: (a) asphalt (flexible); (b) concrete (rigid). (Yoder: *Principles of Pavement Design*, 2/E. Reprinted by permission of John Wiley & Sons.)

terms of strength, hardness, gradation, and aggregate types.

■ **Surface course**—top course of the pavement section consisting of a mixture of mineral aggregates and asphaltic materials. The purpose of the surface course is to provide a smooth riding surface, resist the wear and tear from traffic, provide skid resistance to vehicles, and prevent excessive water from penetrating into the base course.

Depending on the site conditions and the performance of the subgrade, the reasonable range of thickness of the subbase is 6 to 8 in. Base course thickness ranges from 3 to 6 in. and surface course is 1 to 2 in. for lower-category type roads.

Rigid pavements typically consist of a pavement slab of Portland cement concrete and a subbase layer. However, the subbase layer is omitted when the subgrade material is granular. Depending on the amount of steel reinforcement, rigid pavement can be classed as plain concrete, simply reinforced concrete and continuously reinforced concrete. Subbase depths under reasonable conditions may range from 4 to 8 in., and the concrete thickness can range from 6 to 12 in.

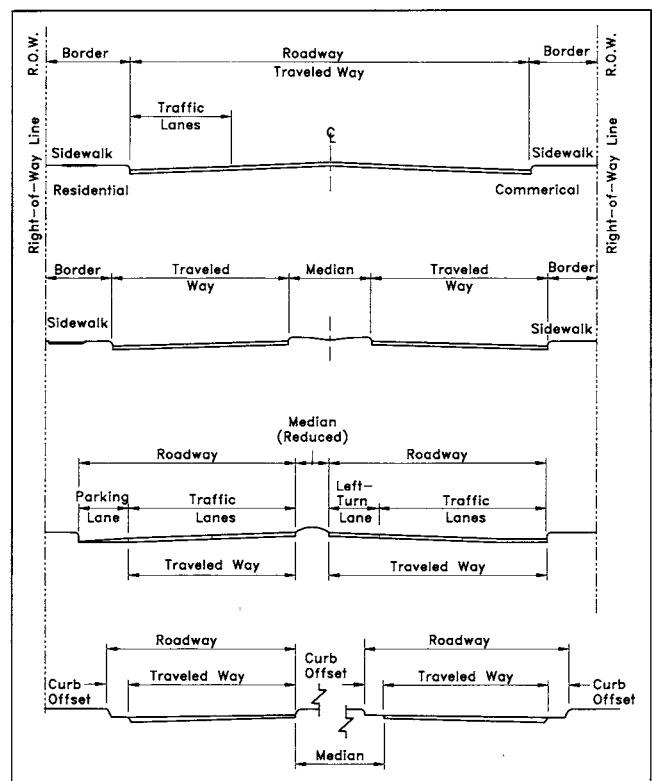
Some local criteria may provide a range of alternative pavement designs for local and residential streets that can be used directly or modified, if necessary, to accommodate specific site conditions.

**Right-of-way Widths on Local Streets**

Right-of-way widths on local streets need be only wide enough to accommodate the pavement and other facilities and uses provided by the land development project or as required by the approving governmental agency. Items for consideration when making a determination on the proper width include water mains, storm and sanitary sewers, power lines, other utilities, roadway signs, and sidewalks. In climates susceptible to heavy snow fall, provisions for storage of plowed snow could affect right-of-way widths. Local controls and requirements of planning boards and other government bodies often dictate predetermined widths without regard to functional needs.

**Typical Section Drawing**

The cross-sectional elements are part of the street design. Their defined interrelationship is based on the functional category of the street, the design standards of the local jurisdiction and the specific requirements of the site. This information should be depicted on the typical section diagram and included as part of the construction drawings. The numerous variations of a typical section are based on



**FIGURE 20.12** Cross-section elements—nomenclature for urban streets. (Used by permission of CH2M Hill Evanston, Ill.)



the amount and type of traffic and vary from project to project. Several typical sections may be necessary for a segment of a road of the same project. Several variations of a typical section are shown in Figure 20.12. Figure 20.13 is an example of a typical section drawing.

**HORIZONTAL ALIGNMENT**

The street layout is constrained by topography, points where the proposed streets can or must connect to existing street networks, and where proposed streets of the development must extend to provide access to adjacent land. The optimal layout of the development must produce maximum lot yield (buildable lots) with consideration for aesthetics and market attraction. All of these constraints factor into the horizontal alignment of the proposed streets. Additionally, required improvements to any existing streets may affect the proposed street layout.

**Circular Curves.** The alignment of a street or roadway in the horizontal plane is critical to the subdivision of land. For arterial streets and highways, this alignment has an affect on design speed and sight distance. For a local street,

speed is not a significant controlling design factor in setting the alignment. Rather, the alignment is determined by the designer's ability to take advantage of the existing topography, while maintaining a safe functional street. The horizontal alignment is comprised of two basic types of geometric sections: straight tangents and circular curves. The circular curve used on local streets is a simple curve. This is a section of a circle having a constant radius as opposed to spiral curves, which are curves of nonuniform radius. Figure 20.14 shows the basic elements of the circular curve that are important to street design.

Two tangent sections of different directions are joined together by a constant radius arc at two points. The point where the tangent ends and the curve begins is labeled the point of curvature (PC). The point where the curve ends and the next tangent begins is labeled the point of tangency (PT). The tangents intersect at the point of intersection (PI). The deflection angle ( $\Delta$ ) is the angle formed between the two tangents at the PI. The angle formed by a tangent and a line from the PC or PT to the radius point is always  $90^\circ$ . Any angle other than  $90^\circ$  produces a flaw in the geometry

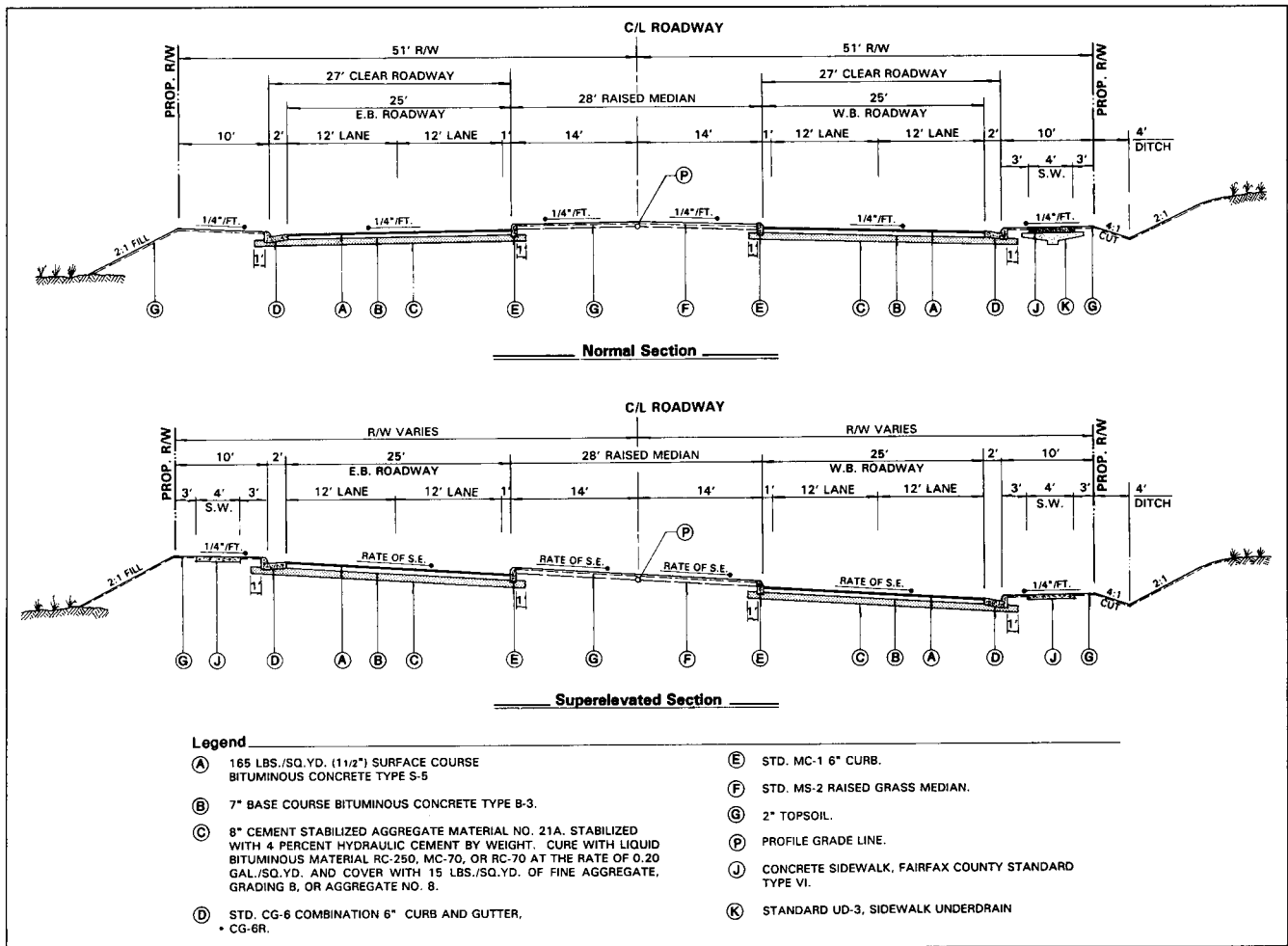
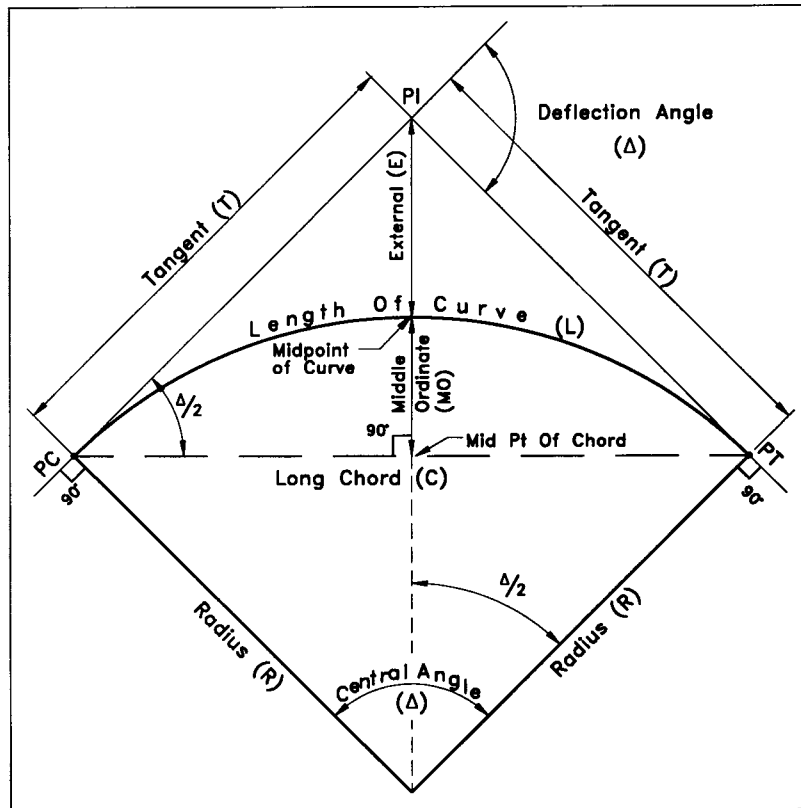


FIGURE 20.13 Typical section.



**FIGURE 20.14** Elements of a circular curve.

and is not a true representation of the circle-tangent relationship.

Two of the seven basic components of the circle have been identified (radius and deflection angle). The remaining components are:

Length of curve ( $L$ ) is the arc length from PC to PT

$$L = \Delta_{(\text{in Radians})} \times R \quad (20.2)$$

Tangent distance ( $T$ ) is the distance from the PI to the PC or PT (both distances are equal)

$$T = R \tan \left( \frac{\Delta}{2} \right) \quad (20.3)$$

Long chord ( $C$ ) is the distance measured along a straight line from the PC to PT

$$C = 2R \sin \left( \frac{\Delta}{2} \right) = 2T \cos \left( \frac{\Delta}{2} \right) \quad (20.4)$$

External distance ( $E$ ) is the distance from the PI to the midpoint on the curve

$$E = R \left( \frac{1}{\cos \frac{\Delta}{2}} - 1 \right) = T \tan \left( \frac{\Delta}{4} \right) \quad (20.5)$$

Middle ordinate ( $MO$ ) is the distance from the midpoint on the curve to the midpoint on the long chord

$$MO = R \left( 1 - \cos \left( \frac{\Delta}{2} \right) \right) \quad (20.6)$$

The length of the arc of a simple curve is proportional to the length of a full circle of the same radius as the central angle subtended by the arc is proportional to 360°. Simply stated,

$$\frac{\widehat{AB}}{2\pi R} = \frac{D}{360^\circ} \quad (20.7)$$

The degree of curvature, like the radius, is a measure of the sharpness of curvature. The degree of curve by arc definition is the amount of central angle ( $\Delta$ ) subtended by 100 ft of arc (see Figure 20.15). Letting the arc distance  $\widehat{AB} = 100$  ft in Eq. (20.7) defines the degree of curve,  $D$ , as

$$D = \frac{5729.58}{R} \quad (20.8)$$

As example, a curve with a 1637-ft radius is equivalent to a  $3\frac{1}{2}^\circ$  curve (i.e.,  $5729.58 \div 1637$ ). A smaller 572.96-

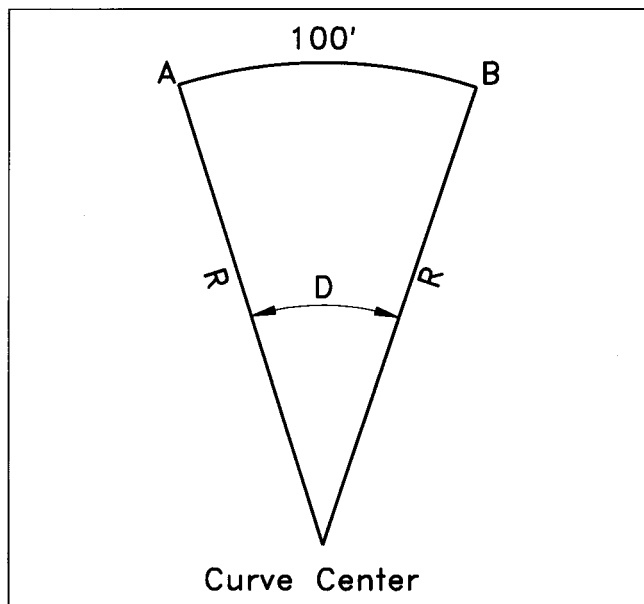


FIGURE 20.15 Degree of curve by arc definition.

ft radius requires a 10° angle to create a 100-ft arc length (see Figure 20.16). Notice that according to Equation (16.8) and the example, as the radius decreases the degree of curve increases. This definition for degree of curvature serves for streets and highways, while a definition based on a 100-ft chord length defines degree of curve for railroad curves.

**Compound Curves**

Connecting simple curves in series with the radius points on the same side of a common tangent forms curves known as compound curves. Each curve has a different radius and connects at a common point of tangency referred to as the point of compound curvature (PCC). As shown in Figure 20.17, the two radius points and the PCC are collinear. In a compound curve, there are three pairs of tangents: the tangents,  $t_1$  and  $t_2$ , associated with each individual curve, and the long tangent,  $T$ , which includes both curves. Unlike the simple curve, the long tangents may not be equal. The following equations are used in computing elements of the compound curve with respect to the nomenclature of Figure 20.17.

$$\Delta = \Delta_L + \Delta_S \tag{20.9}$$

$$T_L = \frac{R_S - R_L \cos \Delta + (R_L - R_S) \cos \Delta_S}{\sin \Delta}$$

$$T_S = \frac{R_L - R_S \cos \Delta - (R_L - R_S) \cos \Delta_L}{\sin \Delta}$$

$$\tan \left( \frac{\Delta_L}{2} \right) = \frac{T_S \sin \Delta - R_S(1 - \cos \Delta)}{T_L + T_S \cos \Delta - R_S \sin \Delta}$$

$$\tan \left( \frac{\Delta_S}{2} \right) = \frac{R_L(1 - \cos \Delta) - T_L \sin \Delta}{R_L \sin \Delta - T_L \cos \Delta - T_S}$$

Compound curves are typically used near at-grade intersections and to accommodate the desired street alignment, especially in difficult topographic areas and in

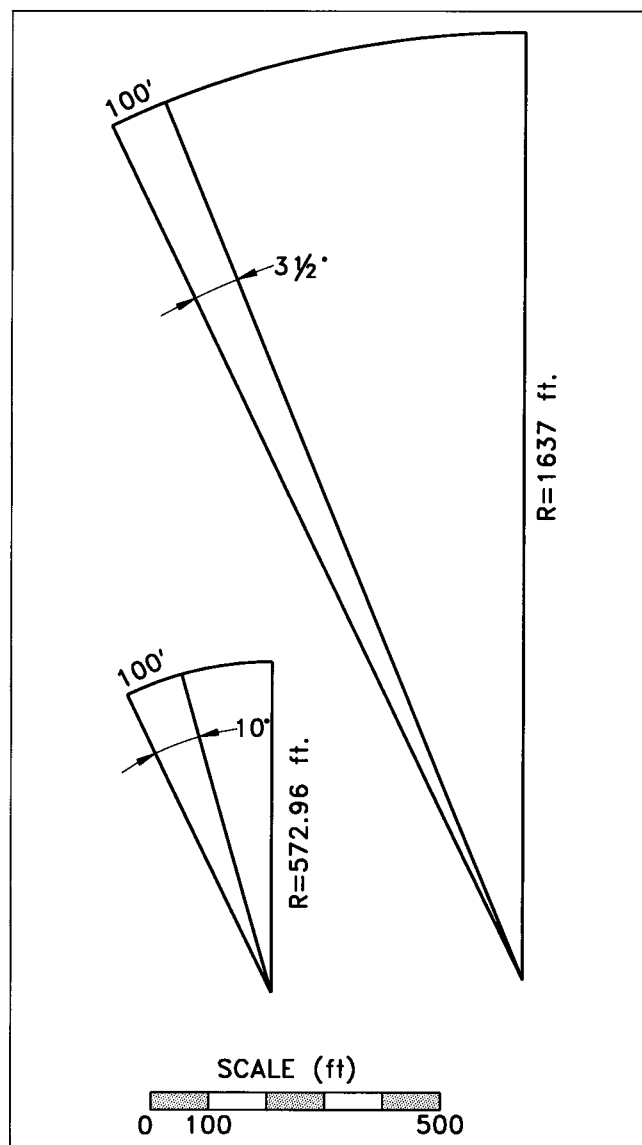


FIGURE 20.16 Comparison of degree of curvature with radius of curvature.

high-density developments where the road alignment may be constrained by other physical features. At low speeds, encountered on local streets, the effects of a sudden change in curvature at the PCC may not be as pronounced as on high-speed highways. To decrease rider discomfort through a compound curve, the radii of the two curves should not differ considerably. AASHTO recommends that the ratio of the flatter radius to the sharper radius be less than 2:1 at intersections. On high-speed highways the ratio is reduced to 1.5:1.

**Reverse Curves**

Two consecutive curves having their radius points on opposite sides of the common tangent is known as a reverse curve. These curves join at a common point of tangency called the point of reverse curve (PRC). To be geometrically correct, the two radius points and the PRC must be collinear.

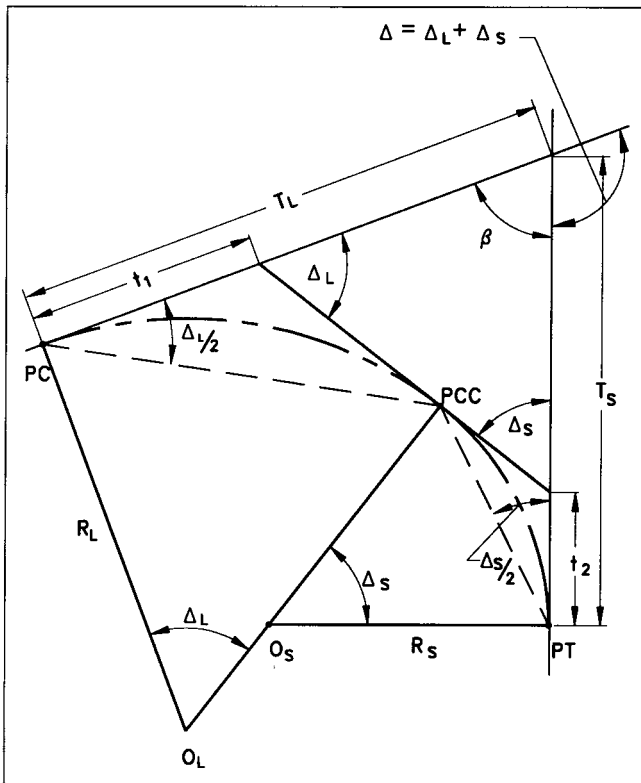


FIGURE 20.17 Elements of a compound curve.

ear. Reverse curves are not recommended on high-speed highways or roads where superelevation is required. In such instances it is desirable to insert a minimum tangent length of 100 ft between curves to allow for transition of superelevation between the curves. This is also a safer design for the driver to negotiate the vehicle through the directional change. When installed with the tangent section between curves, the result is two simple curves used to change the direction of travel. Figure 20.18 shows a schematic of a reverse curve.

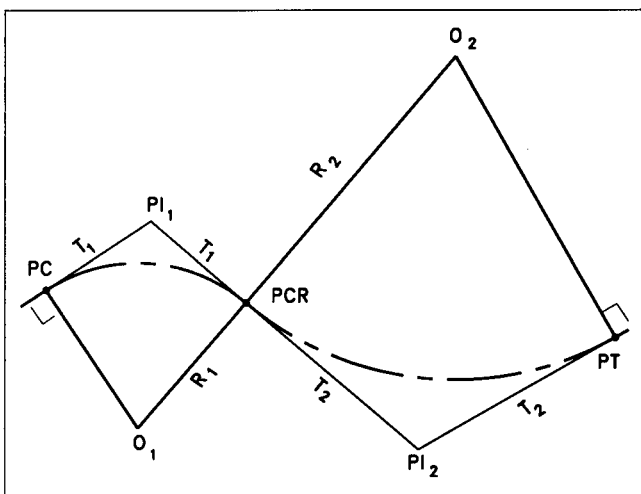


FIGURE 20.18 Elements of a reverse curve.

### Superelevation

A force is imposed on an object whenever it is subjected to a change in momentum. This change in momentum is a result of a change in the magnitude or direction of the velocity. For a vehicle traveling along a curve, the magnitude of the velocity is the speed and the direction of the velocity is a line tangent to the curve in the direction of motion. Assuming a vehicle maintains a constant speed through a curve, the velocity changes direction by virtue of the circular path. The result is a force that tends to push the vehicle radially outward. This radially outward force, the centrifugal force, must be countered by the weight of the vehicle and side friction at the tires for dynamic equilibrium to exist. Superelevation is the raising of the outside edge of pavement on a circular curve to help counterbalance the centrifugal force.

Figure 20.19 depicts the forces acting on the vehicle. Writing the equations of equilibrium with respect to the incline,

$$\frac{Wv^2}{gR} \cos \alpha = W \sin \alpha + Wf_s \cos \alpha \quad (20.10)$$

where  $W$  = weight of the vehicle,  $v$  = velocity of the vehicle

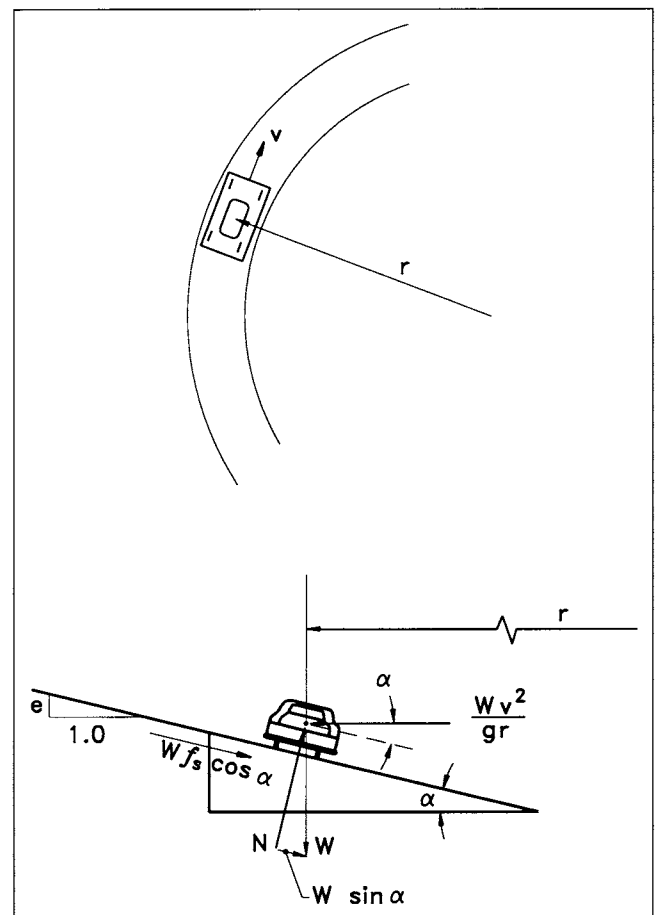


FIGURE 20.19 Forces acting on a vehicle traveling along a curved path.

(fps),  $R$  = radius of curvature,  $g$  = acceleration constant,  $f_s$  = coefficient of side friction,  $\alpha$  = transverse angle of inclination of the road, which can be simplified to

$$\frac{V_{\text{mph}}^2}{15R} = e + f_s \quad (20.11)$$

by dividing through by  $W \cos \alpha$  and recognizing that  $\tan \alpha = e$  (the rate of superelevation in feet per foot of width)  $V$  is in miles per hour and  $g = 32.2 \text{ ft/sec}^2$ . The left side of the equation represents the radial force due to change in velocity direction, and the right side represents the counteracting effects of the weight of the vehicle and the side friction developed between the tires and pavement.

The coefficient of side friction,  $f_s$ , depends on the conditions of the roadway surface, tires, and vehicle speed. Although a constant for a given set of conditions and vehicle,  $f_s$  is not a constant for all conditions nor for all vehicles using the road. Ideally, as a vehicle travels at a constant speed on an appropriately superelevated curve, only the horizontal weight component counters the centrifugal force and no steering effort is required. Inevitably, steering is required as the vehicle speed varies from the theoretical velocity and consequently side friction develops. When the centrifugal force is less than the sum of the horizontal weight component and  $f_s$ , the vehicle will tend to slide down slope. Likewise, on a road with inadequate superelevation, a vehicle at high speeds will tend to move toward the outside of the curve and potentially off the road.

If, as with many local streets, normal crown section is maintained throughout the curve, the cross slope of the outer pavement section has an adverse or negative cross

slope relative to the direction of the curve. For the condition of negative superelevation, there is a minimum radius of curvature where the combined effects of the centrifugal force and the adverse slope are greater than what the force of the side friction can counter, and superelevation is required. Or stated another way, there is a maximum radius of curvature that is suitable for the negative superelevation and the design speed. AASHTO recommends the values given in Table 20.5.

Superelevation is a function of the tires and pavement, weight of the vehicle, and speed of the vehicle. Each of these factors varies from vehicle to vehicle and from day to day. Therefore, there cannot be one single superelevation that will cover the entire range of variables. Maximum superelevation rates are set to account for the uncertainties in these variables. In urban areas where superelevation is used, the recommended upper limit of the rate of superelevation is 0.04 ft/ft to 0.06 ft/ft. Solving for  $R$  in Eq. (20.11) gives the minimum radius or maximum degree of curvature for a given design speed determined from some maximum rate of superelevation and the maximum side friction that may be developed given the range of climatic conditions and range of vehicle speeds. These limiting values are listed in Table 20.6.

Superelevation is rarely a design requirement for minor roads such as local streets and collectors. The reason is that superelevation, in its attempt to improve riding comfort, promotes speeding and hence is a safety liability on residential streets. At design speeds of 25 mph, the reduction in riding comfort for non-superelevated pavement is not significant. However, others might argue that superelevation

**TABLE 20.5 Maximum Curvature for Normal Crown Section**

DESIGN SPEED (MPH)	AVERAGE RUNNING SPEED (MPH)	MAXIMUM CURVE (DEGREES)	MINIMUM CURVE RADIUS (FT)	RESULTING SIDE FRICTION FACTOR, $F$ , WITH ADVERSE CROWN	
				AT DESIGN SPEED	AT RUNNING SPEED
20	20	3°23'	1,700	.031	.031
30	28	1°43'	3,340	.033	.031
40	36	1°02'	5,550	.034	.031
50	44	0°41'	8,320	.035	.031
55	48	0°35'	9,930	.035	.031
60	52	0°29'	11,690	.035	.030
65	55	0°26'	13,140	.036	.030
70	58	0°23'	14,690	.037	.030

(From *A Policy on Geometric Design of Highways and Streets*, Table III-13, p. 172, AASHTO, Washington, D.C., 1990.)

**TABLE 20.6 Maximum Degree of Curve and Minimum Radius Determined for Limiting Values of  $e$  and  $f$ , Rural Highways and High-speed Urban Streets**

DESIGN SPEED (MPH)	MAXIMUM $E$	MAXIMUM $F$	Total ( $E + F$ )	MAXIMUM DEGREE OF CURVE	ROUNDED MAXIMUM DEGREE OF CURVE	MAXIMUM <sup>a</sup> RADIUS (FT)
20	.04	.17	.21	44.97	45.0	127
30	.04	.16	.20	19.04	19.0	302
40	.04	.15	.19	10.17	10.0	573
50	.04	.14	.18	6.17	6.0	955
55	.04	.13	.17	4.83	4.75	1,186
60	.04	.12	.16	3.81	3.75	1,528
.....						
20	.06	.17	.23	49.25	49.25	116
30	.06	.16	.22	20.94	21.0	273
40	.06	.15	.21	11.24	11.25	509
50	.06	.14	.20	6.85	6.75	849
55	.06	.13	.19	5.40	5.5	1,061
60	.06	.12	.18	4.28	4.25	1,348
65	.06	.11	.17	3.45	3.5	1,637
70	.06	.10	.16	2.80	2.75	2,083
.....						
20	.08	.17	.25	53.54	53.5	107
30	.08	.16	.24	22.84	22.75	252
40	.08	.15	.23	12.31	12.25	468
50	.08	.14	.22	7.54	7.5	764
55	.08	.13	.21	5.97	6.0	960
60	.08	.12	.20	4.76	4.75	1,206
65	.08	.11	.19	3.85	3.75	1,528
70	.08	.10	.18	3.15	3.0	1,910
.....						
20	.10	.17	.27	57.82	58.0	99
30	.10	.16	.26	24.75	24.75	231
40	.10	.15	.25	13.38	13.25	432
50	.10	.14	.24	8.22	8.25	694
55	.10	.13	.23	6.53	6.5	877
60	.10	.12	.22	5.23	5.25	1,091
65	.10	.11	.21	4.26	4.25	1,348
70	.10	.10	.20	3.50	3.5	1,637
.....						
20	.12	.17	.29	62.10	62.0	92
30	.12	.16	.28	26.65	26.75	214
40	.12	.15	.27	14.46	14.5	395
50	.12	.14	.26	8.91	9.0	637
55	.12	.13	.25	7.10	7.0	807
60	.12	.12	.24	5.71	5.75	996
65	.12	.11	.23	4.66	4.75	1,206
70	.12	.10	.22	3.85	3.75	1,528

(From *A Policy on Geometric Design of Streets and Highways*, Table III-6, p. 154, AASHTO, Washington, D.C., 1990.)

NOTE: In recognition of safety considerations, use of  $e_{\max} = 0.04$  should be limited to urban conditions.

<sup>a</sup>Calculated using rounded maximum degree of curve.

is a necessity on low-speed streets for the safety of the vehicle and occupants. Rather than superelevate the road beyond normal section cross slopes, a compromise is to superelevate the road only to the extent where the outside cross slope equals the cross slope of the inside lanes at normal section. In practice, this is referred to as reverse crown. On some roads, such as major collectors and minor arterials, superelevation is a requirement, and the design must conform to the applicable criteria.

**Transitions Into and Out of Superelevated Sections**

Superelevation is attained through a smooth gradual process as the outer edge of pavement is gradually lifted up (while the inner edge maintains normal cross slope) until a uniform cross slope exists across the full width of pavement. Further superelevation is then accomplished by rotating the pavement about the centerline, the inner edge of pavement or the outer edge of pavement (see Figure 20.20). The length of road between the point where superelevation begins and full superelevation is attained is called the runoff length or transition.

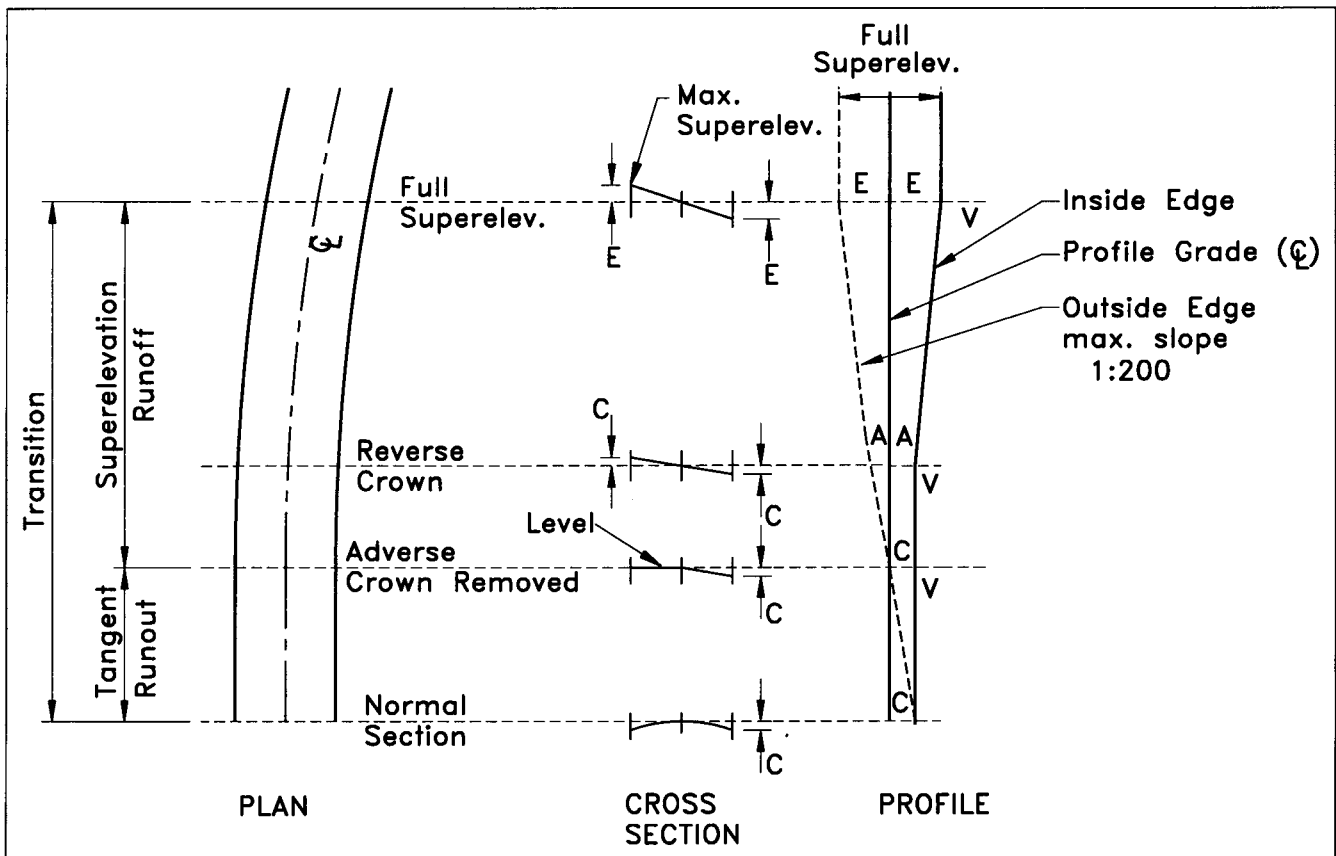
Transition sections are required where the cross slopes of the road change, such as from normal crown to full superelevation going into a curve or from full superelevation to normal crown coming out of a curve. Also, transitions exist in compound curves where the superelevation in-

creases or decreases from one curve to the subsequent curve.

A transition is divided into two segments: tangent runoff and superelevation runoff. Tangent runoff is the length of road where the pavement section changes from normal crown to adverse crown removed or vice versa. (Adverse crown removed is where the outer section of the pavement of the curve is horizontal—i.e., the cross slope is 0%.) Superelevation runoff is the length of the road where the pavement section changes from adverse crown removed to full superelevation and vice versa. Figure 20.20 shows a street transitioning from normal crown to full superelevation by rotation about the centerline.

The length of tangent runoff and superelevation runoff depends on the initial cross slope of the pavement, cross slope at full superelevation, pavement widths and design speed. Table 20.7 is the recommendation by AASHTO. Many local and state highway agencies have developed minimum superelevation runoff lengths of about 100 to 200 ft regardless of the pavement widths and superelevation rate.

Another point of consideration is the location of the transition. It may be divided equally on either side of the PC (or PT) or offset such that most of the transition occurs on the tangent section and full superelevation is attained a short distance into the curve. Design standards of the reviewing agency should specify the transition location. One



**FIGURE 20.20** Transition to full superelevation by rotation about the centerline.

**TABLE 20.7 Length Required for Superelevation Runoff—Two-Lane Pavements**

SUPERELEVATION RATE	L—LENGTH OF RUNOFF (FT) FOR DESIGN SPEED (MPH) OF:							
	20	30	40	50	55	60	65	70
12-ft lanes								
.02	50	100	125	150	160	175	190	200
.04	60	100	125	150	160	175	190	200
.06	95	110	125	150	160	175	190	200
.08	125	145	170	190	205	215	230	240
.10	160	180	210	240	255	270	290	300
.12	195	215	250	290	305	320	350	360
10-ft lanes								
.02	50	100	125	150	160	175	190	200
.04	50	100	125	150	160	175	190	200
.06	80	100	125	150	160	175	190	200
.08	105	120	140	160	170	180	190	200
.10	130	150	175	200	215	225	240	250
.12	160	180	210	240	255	270	290	300

(From *A Policy on Geometric Design of Streets and Highways*, Table III-15, p. 178, AASHTO, Washington, D.C., 1990.)

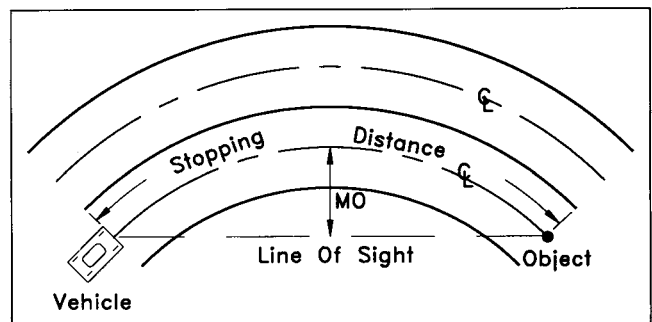
of the reasons for avoiding short sharp circular curves, broken back curves, and short compound curves is the problem of transitioning into and out of superelevation.

It must be recognized that this is not the only way to attain superelevation. Depending on the cross-section geometry of the road (e.g., the presence of medians), the rotation point may vary. The reader is referred to AASHTO manuals, local design manuals, and other engineering texts for detailed discussion.

**Stopping Sight Distance (SSD) on Horizontal Curves**

In residential areas, children play in or near the street and pedestrians tend to cross the streets at their convenience rather than at designated crosswalks and intersections. Such common situations emphasize the need for adequate SSD. Assessing adequate stopping site distance is further complicated by on street parking and trees planted along the side and median of the road. Often adequate stopping site distance is provided with the initial design and the planting of trees and shrubs is placed later. As the trees and shrubs mature, stopping site distance is significantly reduced. Other factors impacting site distance include barriers to the line of sight such as ground slopes, existing buildings, and vertical grades in combination with horizontal curves.

Sight distance for a horizontal curve is the chord length of the centerline of the inside lane of the pavement with the stopping distance measured along the arc length of the curve. This is the actual distance the vehicle travels during the reaction and stopping time. Figure 20.21 illustrates the location of the sight distance line and the stopping distance. The relationship between the middle ordinate distance,



**FIGURE 20.21** Sight distance along a horizontal curve. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.



stopping distance and the curvature is obtained from Eqs. (20.4) and (20.7).

$$MO = R \left( 1 - \cos \left( \frac{28.65}{R} SSD \right) \right) \quad (20.12)$$

where  $MO$  = middle ordinate (ft),  $S$  = stopping distance (ft), and  $R$  = the radius (ft). The reader must recognize that the figure shows how to determine adequate site distance in the horizontal plane. Vertical alignment of the street is also a factor, and this method is not applicable if the horizontal curve is near a vertical curve.

Equation (20.13) shows how to determine the middle ordinate for a local street having a design speed of 25 miles per hour (mph) and no superelevation.

From Eq. (20.11), find the minimum radius for 25 mph using a side friction<sup>4</sup> value of 0.252 (Table 20.8) and  $e = 0$  for no superelevation.

$$R_{\min} = \frac{25^2}{15(0.252)} = 165 \text{ ft} \quad (20.13)$$

(Note, however, that this is a computed value and local criteria may dictate an absolute minimum value different than this.)

The stopping sight distance on level ground is determined by using Eq. (20.1) or from Table 20.4:

$$SSD = (1.47)(2.5)(25) + \frac{25^2}{30(0.38)} = 145 \text{ ft} \quad (20.14)$$

The calculated value of 147 ft is rounded to 150 ft to correspond with Table 20.4. The middle ordinate for use in locating the line of sight comes from the solution of Equation 20.15. Assume that the minimum radius of 165 ft is for the inside lane of a 22-ft-wide local street. The middle ordinate is found to be 16 ft:

$$MO = 165 \left( 1 - \cos \frac{28.65(147)}{165} \right) = 16.1 \text{ ft} \quad (20.15)$$

Referring back to Figure 20.21, the minimum distance

**TABLE 20.8 Side Friction Factors for Selected Design Speeds**

DESIGN SPEED	SIDE FRICTION FACTOR
20 mph	0.300
25 mph	0.252
30 mph	0.221
35 mph	0.197

<sup>4</sup>Side friction is the force between the tires and pavement due to the centripetal force on a vehicle moving through a circular curve.

from the centerline of the travel lane to any object that might impede the line of sight is 16 ft in order to provide a 147-ft stopping sight distance.

**VERTICAL ALIGNMENT**

**Vertical Curves**

Street alignment in the vertical plane is accomplished by connecting straight tangent sections with parabolic curves rather than circular curves. This permits the street alignment to better adapt to the topography for a smooth, safe, and aesthetically pleasing design. Typically, street design will utilize equal tangent vertical curves, called symmetrical vertical curves. Unequal tangents—i.e., unsymmetrical vertical curves—may be used to fit the alignment to uncommon site constraints.

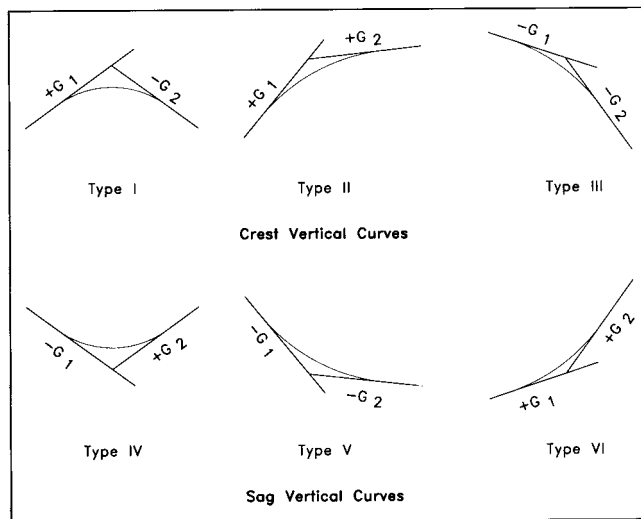
Figure 20.22 shows six types of vertical curves possible by combining positive and negative gradients. Figure 20.22 (top) shows crest vertical curves, and Figure 20.22 (bottom) shows bottom sag vertical curves. A type I curve has a high point and a type IV curve has a low point.

A vertical curve consists of two straight tangent segments at slope gradients, designated as  $g_1$  and  $g_2$ , and the connecting vertical curve (see Figure 20.23). The curve is tangent to the straight segments at the point of vertical curve (PVC) and the point of vertical tangent (PVT). The tangents intersect at the point of vertical intersection (PVI). The length of the vertical curve is the horizontal distance between the PVC and PVT. It is not measured along the curve itself. For equal tangent vertical curves, the horizontal distances from the PVC to PVI and PVI to PVT are equal.

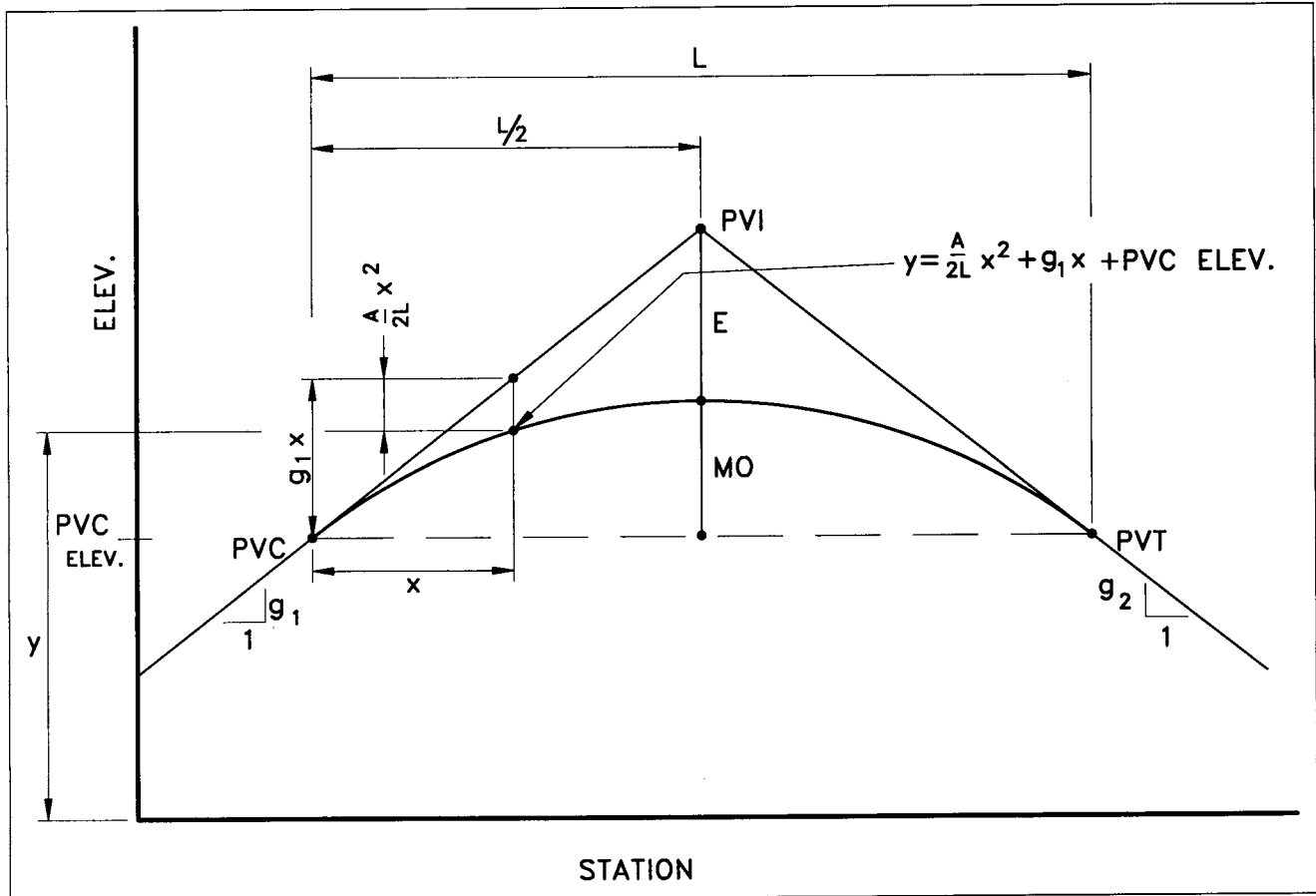
The equation for a vertical curve is based on the general equation of a parabola, given as

$$y = ax^2 + bx + c \quad (20.16)$$

where  $a$ ,  $b$ , and  $c$  are constants characteristic of the vertical



**FIGURE 20.22** Various types of vertical curves. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.



**FIGURE 20.23** Schematic diagram for symmetrical vertical curve.

curve and depend on the length of curve, the slope gradients, and elevations. As shown in Figure 20.23,  $y$  is the elevation and  $x$  is the distance from the PVC to a point on the curve.

Using the parabolic Eq. (20.16), the general equation for any vertical curve is

$$y = \frac{1}{2} \left( \frac{g_2 - g_1}{L} \right) x^2 + g_1 x + \text{PVC}_{\text{Elev}} \quad (20.17)$$

where  $y$  is the elevation of the point on the vertical curve a horizontal distance  $x$  away from the PVC. Frequently in highway design manuals and textbooks, the algebraic difference of the gradients,  $g_2 - g_1$ , is designated as  $A$ .

Figure 20.23 shows the application of the mathematical terms. The first term, the tangent offset  $= Ax^2/2L$ , represents the vertical distance from the point on the curve to the tangent represented by  $g_1$ . The  $g_1 x$  term is the vertical distance between the point on the tangent and the PVC, and the summation  $g_1 x + \text{PVC}$ , gives the tangent elevation.

The center of the parabola is midway between the vertex—i.e., PVI—and the long chord. The middle ordinate, the vertical distance from the long chord to the curves equal to the external, is the vertical distance from the curve to the PVI and is given as

$$E = MO = \frac{AL}{8} \quad (20.18)$$

The relationship between the tangent offset,  $d$ , at any point on the vertical curve and the external is given as

$$d = \frac{1}{2} \left( \frac{A}{L} (x^2) \right) = E \left( \frac{x}{L/2} \right)^2 \quad (20.19)$$

If  $x$  is the distance measured from the PVC,  $d$  is the vertical distance measured from the connecting tangent. Likewise, if  $x$  is measured from the PVC,  $d$  is the distance measured from the tangent connecting the PVT. Note that the offsets from the two tangents are symmetrical with respect to the PVI.

Using the derivative of  $x$  with respect to  $y$  in Eq. (20.17), the distance from the PVC to the high point/low point of a crest/sag curve is

$$x = -\frac{g_1}{A} \quad (20.20)$$

Substitution of this value into Eq. (20.17) sets the elevation at the high or low point. For type I and IV curves, Eq. (20.20) shows that

If  $g_1 = g_2$ , the high/low point location is in the middle of the curve.

If  $g_1 \neq g_2$ , the high/low point is located on the side of the PVI with the flatter gradient.

**EXAMPLE 1:** A crest vertical curve ( $L = 500$  ft) has tangent gradients of  $+3.5$  and  $02.0\%$ . The PVI station is  $11 + 50$  with elevation  $260.00$ . Find the station and elevation of the PVC and PVT, the station of the high point, and the elevations at stations  $10 + 25$  and  $12 + 75$ .

**SOLUTION:** The station of the PVC and PVT are

$$\begin{aligned} \text{PVC Sta} &= \text{PVI Sta} - L/2 = (11 + 50) \\ &\quad - \frac{1}{2}(500) = 9 + 00 \end{aligned} \quad (20.21)$$

$$\begin{aligned} \text{PVT Sta} &= \text{PVI Sta} + L/2 = (11 + 50) \\ &\quad + \frac{1}{2}(500) = 14 + 00 \end{aligned} \quad (20.22)$$

The elevations of the PVC and PVT are

$$\begin{aligned} \text{PVC Elev} &= \text{PVI Elev} - g_1(L/2) \quad (20.23) \\ &= 260 - (0.035)(250) \\ &= 251.25 \end{aligned}$$

$$\begin{aligned} \text{PVT Elev} &= \text{PVI Elev} - g_2(L/2) \quad (20.24) \\ &= 260 - (.02)(250) \\ &= 255.00 \end{aligned}$$

From Eq. (20.20), the distance from the PVC to the high point is

$$x_{HP} = \frac{(0.035)(500)}{.055} = 318.18 \text{ ft} \quad (20.25)$$

The station of the high point is

$$(9 + 00) + (3 + 18.18) = 12 + 18.18 \quad (20.26)$$

Note that the high point falls to the right of the PVI—i.e., the side with the flatter gradient.

Using Eq. (20.17), the elevation at station  $10 + 25$  is (recognizing that the distance from PVC to  $10 + 25$  is  $125$  ft)

$$\begin{aligned} y &= \frac{1}{2} \left( \frac{(-.02) - (.035)}{500} \right) (125)^2 \\ &\quad + (.035)(125) + 251.25 \quad (20.27) \\ &= -0.86 + 4.38 + 251.25 = 254.77 \end{aligned}$$

Similarly the elevation at station  $12 + 75$  is

$$\begin{aligned} y &= \frac{1}{2} \left( \frac{(-0.02) - (0.035)}{500} \right) (375)^2 \quad (20.28) \\ &\quad + (.035)(375) + 251.25 \\ &= -7.73 + 13.12 + 251.25 \\ &= 256.65 \end{aligned}$$

Alternatively, the elevations at these stations can be found by computing the tangent elevations of  $g_1$  and  $g_2$  and subtracting the corresponding tangent offsets using Eq. (20.19).

$$\begin{aligned} \text{Tangent Elev at } 10 + 25 & \quad (20.29) \\ &= \text{PVC Elev} + g_1x \\ &= 251.25 + (0.035)(125) \\ &= 255.63 \end{aligned}$$

$$\begin{aligned} \text{Tangent Offset} &= \frac{1}{2} \left( \frac{(0.02) - (0.035)}{500} \right) (125)^2 \\ &= -0.86 \text{ ft} \end{aligned}$$

$$\begin{aligned} \text{Elev on Curve} &= \text{Tangent Elev} + \text{Tangent Offset} \\ &= 255.63 + (-0.86) = 254.77 \text{ ft} \end{aligned}$$

$$\begin{aligned} \text{Tangent Elev at } 12 + 75 & \quad (20.30) \\ &= \text{PVT Elev} + g_2x \\ &= 255.00 + (0.02)(125) \\ &= 257.50 \end{aligned}$$

$$\begin{aligned} \text{Tangent Offset} &= \frac{1}{2} \left( \frac{(0.02) - (0.035)}{500} \right) (125)^2 \\ &= -0.86 \text{ ft} \end{aligned}$$

$$\begin{aligned} \text{Elev on Curve} &= 257.50 + (-0.86) \\ &= 256.64 \end{aligned}$$

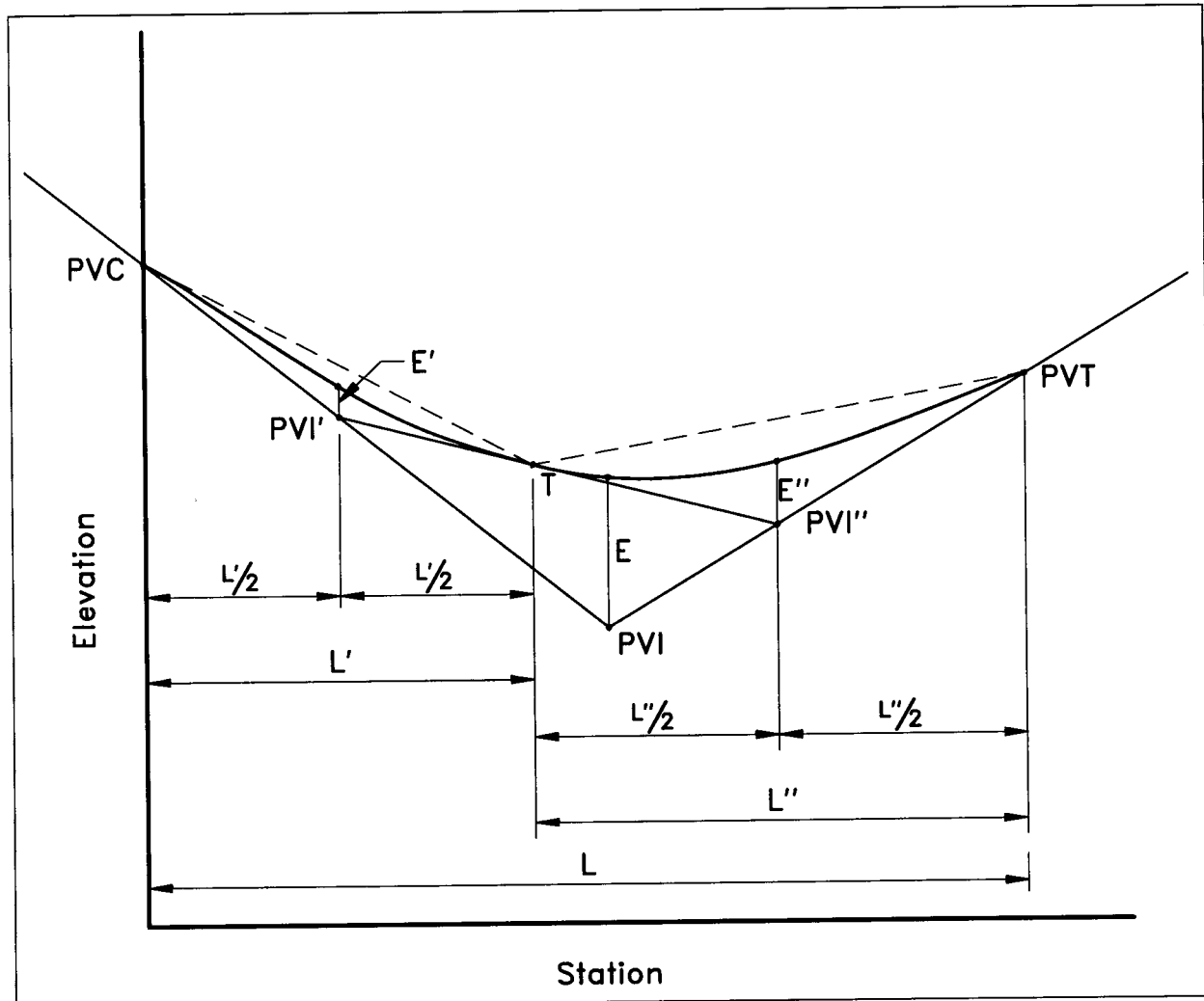
Note that in the preceding calculations, stations  $10 + 25$  and  $12 + 75$  are  $125$  ft on either side of the PVI; hence, the corresponding tangent offsets are equal.

One last note regarding vertical curves. In Figure 20.24, a line drawn tangent to the curve at  $T$  intersects the initial tangents, represented by  $g_1$  and  $g_2$ , at  $\text{PVI}'$  and  $\text{PVI}''$ . Note that  $\text{PVI}'$  is the midpoint between the PVC and  $T$  and  $\text{PVI}''$  is the midpoint between  $T$  and the PVT. Generalizing, two tangents drawn from any two points on a parabolic curve intersect midway between the two points as measured along the horizontal. The gradient of such a line tangent to the curve is given as

$$g' = (g_2 - g_2) \left( \frac{x}{L} \right) + g_1 \quad (20.31)$$

where  $g'$  is the slope of the tangent line a distance  $x$  from the PVC.

Occasionally a situation exists where a portion of an existing vertical curve needs to be modified to accommodate special field conditions. The following example shows how to preserve part of the existing vertical curve using the preceding discussion on intersecting tangents.



**FIGURE 20.24** Tangents drawn from any two points on the vertical curve intersect at the midpoint.

**EXAMPLE 2:** Construction has begun on a road that has a 600-ft vertical curve with PVI at station 25 + 00, elevation = 230.00,  $g_1 = +5\%$ ,  $g_2 = -3\%$ . This curve is based on connecting to a proposed road (design nearly complete but not approved) at elevation 215.00 at station 30 + 00. Based on a change in the design of the connecting road due to newly discovered field conditions, the grade of the connecting road has to be raised 3 ft. What can be done to the design of the approved road to meet the new criteria? Recognize that a major revision to the approved road will delay construction and require time to change the grading, drainage, and other plan sheets (see Figure 20.25).

**SOLUTION:** To preserve as much of the design of the approved road as possible, thus minimizing the impact of the revision, draw a line from the new elevation of 218.00 at station 30 + 00 tangent to the 600-ft vertical curve. This will be the new tangent of the revised vertical curve. By performing this step,

part of the original 600-ft vertical curve is preserved. The location (i.e., station) and elevation of the point where this line is tangent to the curve is unknown. Set the unknown distance from the PVC as  $x$  and the unknown elevation as  $y$ .

The slope of this line relative to the vertical curve is given by Eq. (20.31). Writing the slope of the tangent line using the coordinates of the point at station 30 + 00 and the unknown point is

$$g' = \frac{y_1 - y}{x_1 - x} \quad (20.32)$$

where  $x_1$  is the distance from the PVC to station 30 + 00 and  $y_1$  is the elevation at station 30 + 00. Setting Eq. (20.31) equal to Eq. (20.32) and substituting Eq. (21.17) for  $y$  produces an equation with only the unknown variable  $x$ . The equation simplifies to the quadratic form of

$$ax^2 + bx + c = 0 \quad (20.33)$$

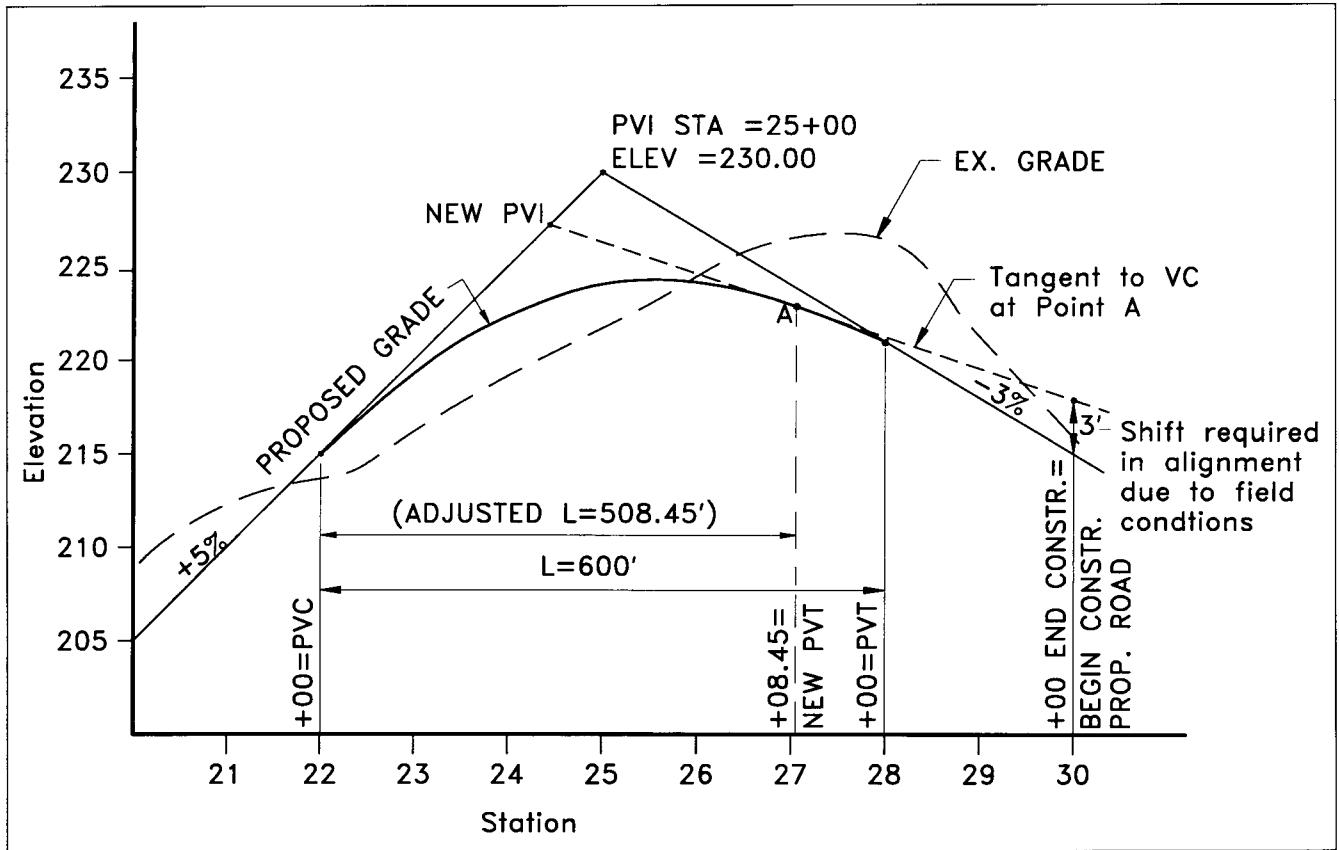


FIGURE 20.25 Example for maintaining part of an existing vertical curve.

where

$$a = \left( \frac{g_2 - g_1}{2L} \right)$$

$$b = \left( (g_2 - g_1) \frac{x_1}{L} \right)$$

$$c = (-PVC_{Elev} + y_1 - g_1 x_1)$$

Using the quadratic equation with appropriate substitutions for  $a$ ,  $b$ , and  $c$  gives two  $x$  values

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \tag{20.34}$$

$$x = 1092 \text{ ft or } 508.45 \text{ ft}$$

The correct value for  $x$  is 508.45 ft.

1. The slope of this tangent is found by substituting this value of  $x$  into Eq. (20.31), resulting in a slope of  $-1.78\%$ . The new PVI is located at the midpoint between the PVC and this point of tangency, which becomes the new PVT, or at station  $24 + 54.23$ .

2. This method shows that 508.45 ft of the original vertical curve remains unchanged, thus

minimizing the overall revisions that might be required.

**Sight Distance on Crest Vertical Curves**

Design of vertical curves includes the provision for safe stopping distances, rider comfort, and general appearance. However, in most if not all cases, the stopping sight distance is the controlling factor. The safe stopping distance requires a minimum length of curve fitted to the given tangent gradients and design speed.

Two cases exist with regard to the sight distance and the length of the vertical curve.

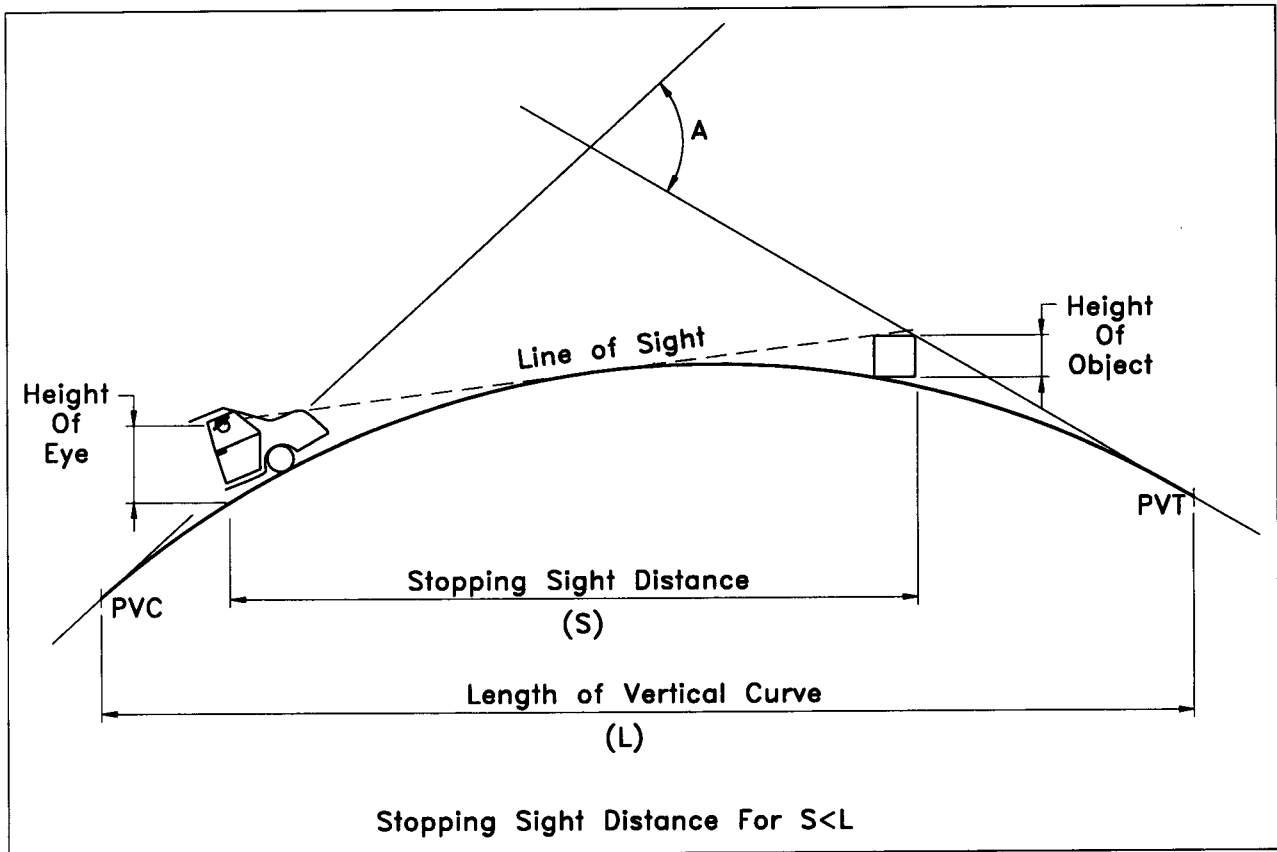
Case I exists where the required sight distance,  $S$ , is less than the length of the vertical curve,  $L$  as shown in Figure 20.26. The applicable equation is

$$L = \frac{AS^2}{200(\sqrt{H_1} + \sqrt{H_2})^2} \tag{20.35}$$

$$L = \frac{A \cdot S^2}{1329} \text{ (for } H_1 = 3.5 \text{ ft, } H_2 = 0.5 \text{ ft)}$$

where  $S < L$ .

Case II (shown in Figure 20.27) occurs when the required sight distance is greater than the length of the vertical curve, that is,  $S > L$ :



**FIGURE 20.26** Stopping sight distance when  $S < L$ .

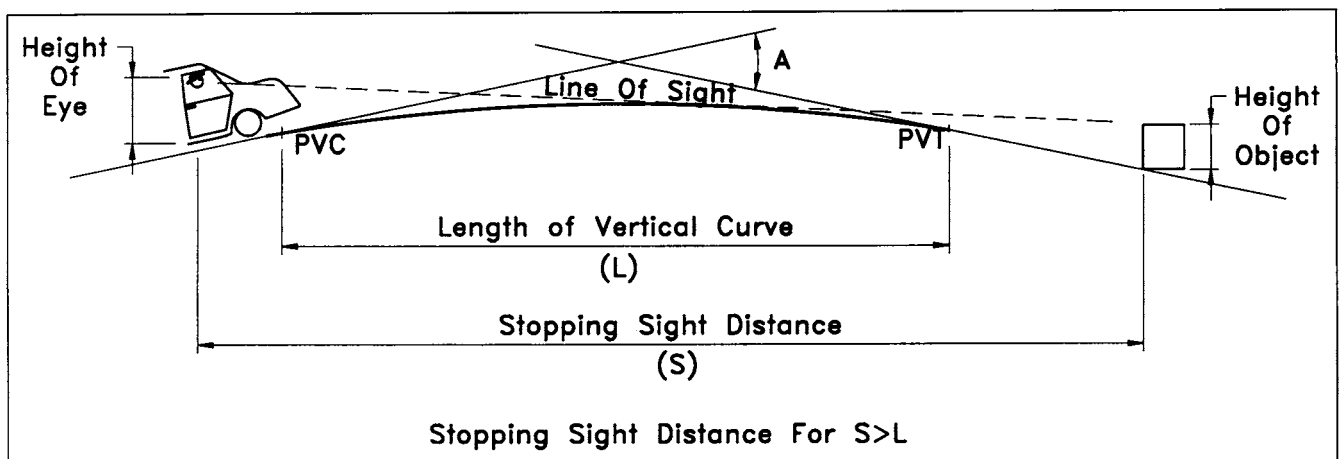
$$L = 2S - \frac{200 (\sqrt{H_1} + \sqrt{H_2})^2}{A} \quad (20.36)$$

$$L = 2S - \frac{1329}{A} \quad (\text{for } H_1 = 3.5 \text{ ft, } H_2 = 0.5 \text{ ft})$$

$H_2$  = height of object (AASHTO recommends  $H_1 = 3.5$  ft, and  $H_2 = 0.5$  ft).

To find a minimum curve given  $g_1$ ,  $g_2$ , and the design speed requires knowing the stopping sight distance obtained from either Eq. (20.1) or Table 20.4. Assume either the  $S < L$  or  $S > L$  case and use the appropriate equation (Eq. (20.35) or (20.36) according to the assumption) to find curve length,  $L$ . If the computed curve length is contrary

where  $L$  = length of the vertical curve,  $S$  = sight distance in feet,  $A$  = algebraic difference in grade where  $A = G_2 - G_1$  (grades are given in percent),  $H_1$  = height of eye, and



**FIGURE 20.27** Stopping sight distance when  $S > L$ .

to the assumed condition, use the other equation to recalculate the correct length.

**EXAMPLE 3:** Given  $g_1 = +3.5\%$  and  $g_2 = -2\%$  and a design speed of 45 mph, determine the minimum curve length.

**SOLUTION:** From Table 20.9, the upper value for the SSD is 400 ft. Assume that  $S < L$  (see Figure 20.26) and compute  $L$  from Eq. (20.35):

$$L = \frac{(3.5 - (-2.0))(400)^2}{1329} = 662 \text{ ft} \quad (20.37)$$

Since  $S (= 400 \text{ ft})$  is less than  $L (= 662 \text{ ft})$  the assumed case is correct.

Note that if Eq. (20.36) were used instead, then

$$L = (2)(400) - \frac{1329}{3.5 - (-2)} = 558 \text{ ft} \quad (20.38)$$

Here  $S (= 400)$  is less than  $L (= 558 \text{ ft})$  but the equation was for  $S > L$  (see Figure 20.27), which contradicts the result. Therefore, case I is the correct value.

A frequently used parameter for relating stopping sight distances to the length of a vertical curve is the ratio of the

length of curve to the algebraic difference in tangent gradients,  $K = L/A$ . Parameter  $K$  is a measure of curvature in that the ratio represents the horizontal distance required generating a 1% change in grade and  $A$  is the algebraic difference of the tangent gradients in percent (%). As shown in Figure 20.28, a higher  $K$  value is indicative of a flat vertical curve. For a given design speed,  $K$  is a constant and is based on sight distance criteria. Therefore, given either tangent gradients or a known curve length immediately establishes the other minimum vertical curve design parameters. In lieu of using the equations to determine the minimum length of vertical curve, AASHTO provides  $K$  values listed in Table 20.9 for various design speeds. These  $K$  values can then be used to determine the minimum length of vertical curve required for the given conditions. The  $K$  values in this table are derived from Eq. (20.35), which can be written as  $L = KA$  where  $K = S^2/1329$ .

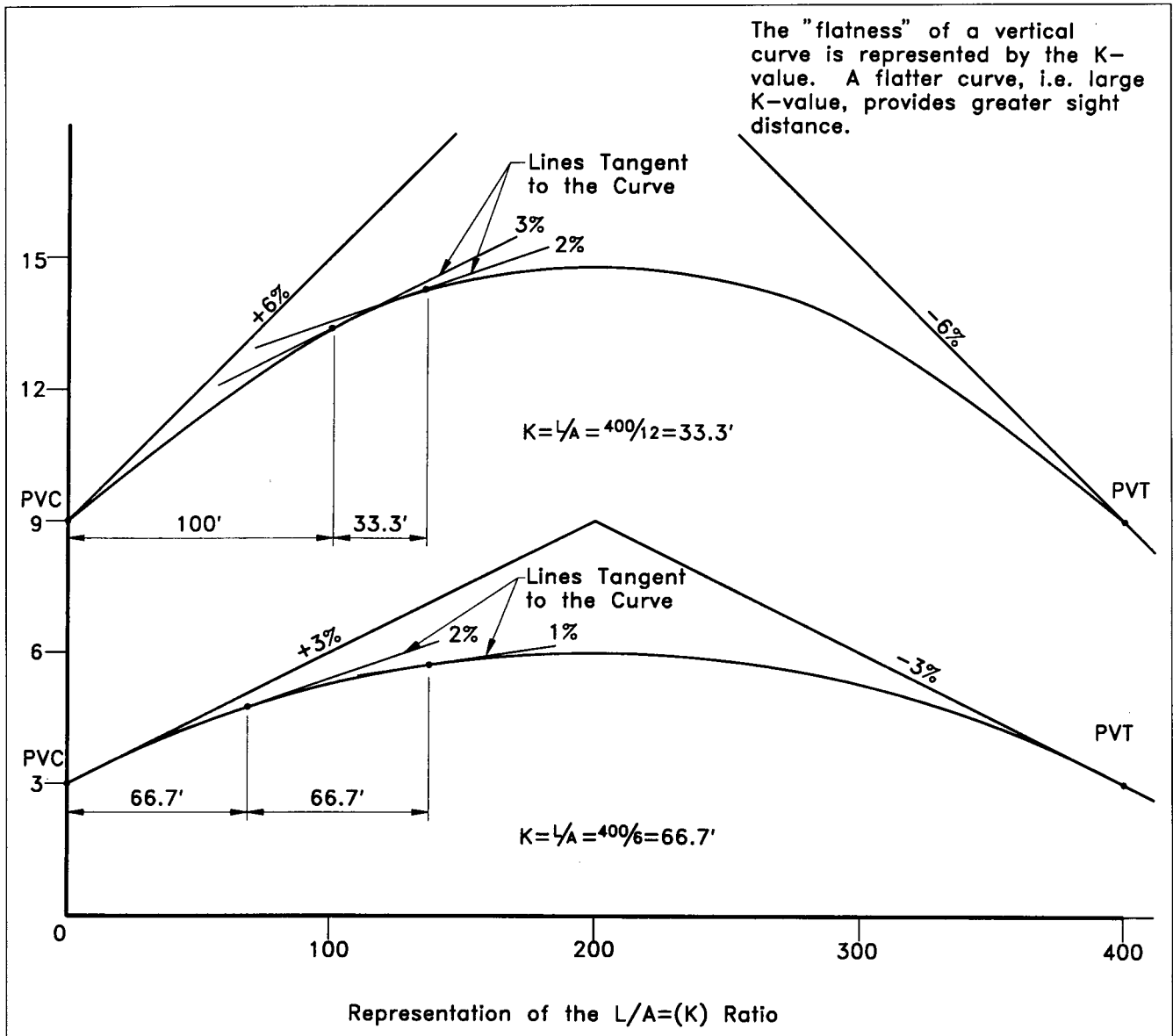
Most design standards maintain absolute minimum distances for the length of a vertical curve when  $S$  is greater than  $L$ . For local and collector streets, this minimum may typically be between 50 and 200 ft. Local ordinances or state highway criteria may require additional increases in stopping distance based on development density, local climatic conditions, and other experiences. The engineer should check the local design criteria to determine the minimum vertical curve requirements.

**TABLE 20.9 Design Controls for Crest Vertical Curves Based on Stopping Sight Distance**

DESIGN SPEED (MPH)	ASSUMED SPEED FOR CONDITION (MPH)	COEFFICIENT OF FRICTION $F$	STOPPING SIGHT DISTANCE ROUNDED FOR DESIGN (FT)	RATE OF VERTICAL CURVATURE, $K$ (LENGTH (FT) PER PERCENT OF $A$ )	
				COMPUTED <sup>a</sup>	ROUNDED FOR DESIGN
20	20–20	0.40	125–125	8.6–8.6	10–10
25	24–25	0.38	150–150	14.4–16.1	20–20
30	28–30	0.35	200–200	23.7–28.8	30–30
35	32–35	0.34	225–250	35.7–46.4	40–50
40	36–40	0.32	275–325	53.6–73.9	60–80
45	40–45	0.31	325–400	76.4–110.2	80–120
50	44–50	0.30	400–475	106.6–160.0	110–160
55	48–55	0.30	450–550	140.4–217.6	150–220
60	52–60	0.29	525–650	189.2–302.2	190–310
65	55–65	0.29	550–725	227.1–394.3	230–400
70	58–70	0.28	625–850	282.8–530.9	290–540

(From *A Policy on Geometric Design of Highways and Streets*, Table III-40, p. 244, AASHTO, Washington, D.C., 1990.)

<sup>a</sup>Using computed values of stopping sight distance.



**FIGURE 20.28** Representation of the  $L/A (= K)$  ratio.

**Sight Distance on Sag Vertical Curves**

Factors controlling the lengths of sag vertical curves include sight distance related to automobile headlights, drainage, and rider comfort. Equations for determining sag vertical curve lengths for minimum sight distances are based on the illuminated area from the average car headlight beam. The headlight beam is assumed to be 2 ft high and directed 1° upward from the longitudinal axis of the vehicle. Additional area illuminated by dispersion is ignored for establishing the site distance equations. Equation (20.36) applies to sag vertical curves. Table 20.10 contains the appropriate K values and stopping site distances for sag vertical curves.

$$L = \frac{A \cdot S^2}{400 + 3.5S} \tag{20.39}$$

where  $S < L$ .

$$L = 2S - \frac{400 + 3.5S}{A} \tag{20.40}$$

where  $S > L$ .

On sag curves, drainage is a more critical factor than for crest vertical curves. AASHTO guidelines suggest providing a minimum grade of 0.3% within 50 ft of the low point to ensure adequate drainage. This corresponds to a K value of 167. Sag vertical curves with K values of 167 and greater are an indication that the curve may be too flat in the sump area to properly drain. This can create safety and drainage problems especially on streets with curb or curb and gutter. The 167 value is not a minimum but rather a guide indicating that attention to grade lines is warranted.

**EXAMPLE 4:** Given tangent grades  $g_1 = -3\%$  and  $g_2 = +11\%$  for a local category street and design



**TABLE 20.10 Design Controls for Sag Vertical Curves Based on Stopping Sight Distance**

DESIGN SPEED (MPH)	ASSUMED SPEED FOR CONDITION (MPH)	COEFFICIENT OF FRICTION <i>F</i>	STOPPING SIGHT DISTANCE ROUNDED FOR DESIGN (FT)	RATE OF VERTICAL CURVATURE, <i>K</i> (LENGTH (FT) PER PERCENT OF <i>A</i> )	
				COMPUTED <sup>a</sup>	ROUNDED FOR DESIGN
20	20–20	0.40	125–125	14.7–14.7	20–20
25	24–25	0.38	150–150	21.7–23.5	30–30
30	28–30	0.35	200–200	30.8–35.3	40–40
35	32–35	0.34	225–250	40.8–48.6	50–50
40	36–40	0.32	275–325	53.4–65.6	60–70
45	40–45	0.31	325–400	67.0–84.2	70–90
50	44–50	0.30	400–475	82.5–105.6	90–110
55	48–55	0.30	450–550	97.6–126.7	100–130
60	52–60	0.29	525–650	116.7–153.4	120–160
65	55–65	0.29	550–725	129.9–178.6	130–180
70	58–70	0.28	625–850	147.7–211.3	150–220

(From *A Policy on Geometric Design of Highways and Streets*, Table III-42, p. 293, AASHTO, Washington, D.C., 1990.)

<sup>a</sup>Using computed values of stopping sight distance.

speed of 30 mph. The PVI station is  $9 + 27.6$  at elevation 659.8 ft.

1. Establish a vertical curve to meet AASHTO criteria
2. Plot the curve in profile
3. Locate the station of the low point.

1. From Table 20.10, the *K* value based on 30-mph design is 30. The algebraic difference in grades is  $(-3\%) - 11\% = 14\%$ . Using the relationship  $L = KA$ , the curve length is  $L = 30 \times 14 = 420$  ft. A vertical curve length of 420 ft sets the PVC and PVT station at  $7 + 17.60$  and  $11 + 37.60$ , respectively. Notice that using  $K = 30$  provides a stopping sight distance of 200 ft, which is above the computed range of 177.3 ft to 195.7 ft shown in Table 20.4.

2. With reference to Figure 20.29, the curve can be easily plotted by computing several elevations along the curve. The elevation of the PVC, determined from the PVI elevation and the tangent gradient, is

$$PVC_{Elev} = 210(0.03) + 659.8 = 666.1 \text{ ft} \quad (20.41)$$

Likewise the PVT elevation is

$$PVT_{Elev} = (210)(0.11) + 659.8 = 682.9 \text{ ft} \quad (20.42)$$

One method to plot the vertical curve is to compute the tangent offsets at selected stations and plot these points on the profile. A smooth line can then be drawn through the plotted points using an irregular curve. Using Eq. (20.19) and the fact that tangent offsets are symmetrical with respect to the PVI, a table of values is created (see Table 20.11). (The table is for illustrative purposes; the procedure can be performed quickly with a hand-held calculator.) The curve is plotted in Figure 20.29.

3. The lowpoint is determined using Eq. (20.20).

$$x = -\frac{(-.03)(420)}{0.14} = 90 \text{ ft} \quad (20.43)$$

Note that the distance from the PVC to the lowpoint is equivalent to  $(g_1\%) \times K$ . The station of the lowpoint is  $(7 + 17.6) + (0 + 90.0) = 8 + 07.6$ . Substituting the value of 90 ft into Eq. (20.17) gives the lowpoint elevation as

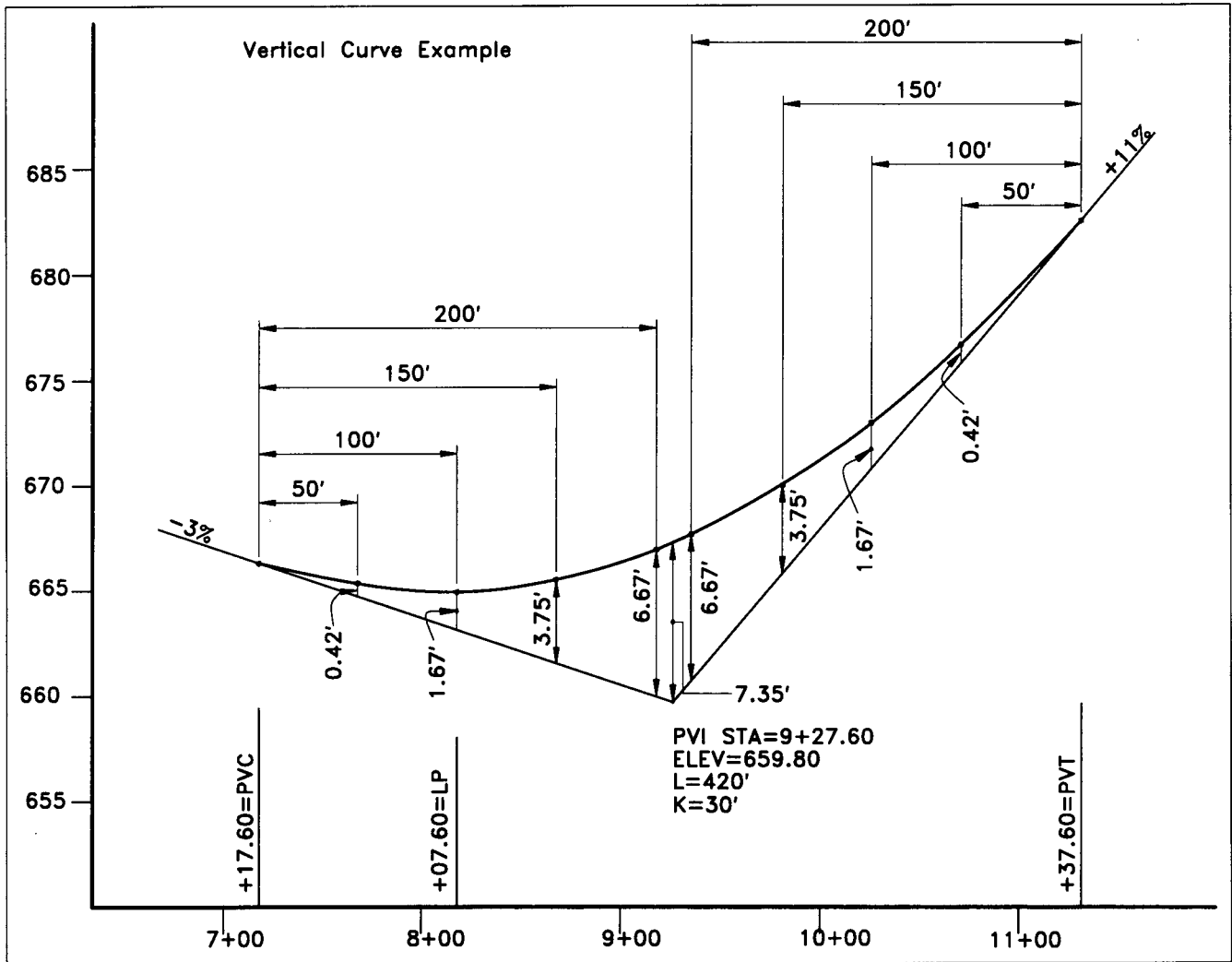


FIGURE 20.29 Vertical curve for Example 4.

$$y = \left( \frac{0.11 - (-0.03)}{2(420)} \right) (90^2) + (-0.03)(90) + 666.1 = 664.75 \text{ ft} \quad (20.44)$$

intersections are used for joining local, collector and many minor arterial streets, whereas the latter two types are more generally used on high-speed highways or arterials. Except

Another problem frequently encountered involving vertical curves is that of fitting the curve through a point with a prescribed elevation. For this type of problem, use Eq. (20.17) and let  $y$  equal the prescribed elevation. Three of the four remaining parameters  $g_1$ ,  $g_2$ ,  $L$ , or  $x$  in the equation must be known to obtain a solution.

**GEOMETRIC PROPERTIES OF AT-GRADE INTERSECTIONS**

Intersections are points where two or more streets or highways join or cross. The three basic types of intersections in use are at-grade, grade separated, and interchange. At-grade

**TABLE 20.11 Tangent Offsets**

DISTANCE FROM PVC OR PVT	TANGENT OFFSETS
50	0.42
100	1.67
150	3.75
200	6.67
210 (PVI)	7.35

for certain unusual situations, the number of legs at any one intersection should not exceed four.

Intersections affect the overall performance of the intersecting roads. A measure of the operational performance includes the speed of movement through the intersection. The movement speed determines the capacity and thus affects the suitability of a street to function as intended. In other words, the level of service of a street is greatly affected by the level of service of the intersection. Design features that impact the design of at-grade intersections include adequate corner sight distances, angle of approach, matching pavement grades, radius of curvature of the curb returns, need for auxiliary lanes or channelization, and intersection control.

Physical elements, traffic, human factors and economics encompass items for consideration in the design process of at-grade intersections. For local streets in residential areas, the major design considerations are adequate corner site distances, matching street grades, and minimum radius of the curb returns. The complexities of intersection design increase as the traffic and pedestrian volume increases and the potential rate of conflicts between pedestrian–vehicle and vehicle–vehicle increases.

The three basic types of at-grade intersection are the T-intersection, four-leg intersection, and the multileg intersection. Each of these types has numerous variations. The T-intersection and four-leg intersection are the most prevalent in the lower-category streets.

### T-Intersections

The T-intersection, or three-leg intersection, is as simple as the name implies. This intersection normally has an automatic right-of-way assignment; however, some situations may warrant the elimination of this automatic right-of-way assignment. One situation is to control speed through a particularly straight section of the road. The T-intersection has the least number of conflict points of any of the various types of intersections and therefore provides a high level of safety. Figure 20.30 illustrates some of the basic T-intersection variations.

### Four-Leg Intersections

The four-leg intersection, like the three-leg intersection, may or may not have an automatic right-of-way assignment. Depending on the design volume of traffic at the intersection, control can be as simple as yield/stop at the lesser-category street to signal control with auxiliary lanes and islands to channel the turning movements. Variations of the latter situation typically occur at intersections of major roads, such as collectors with minor arterials. Such intersections frequently are necessary at the fringe of large development projects in high-density areas. Figure 20.31 illustrates some of the basic configurations of the four-leg intersection.

Complete solutions and variations for various circumstances are numerous. For special situations, the designer might find helpful the National Cooperative Highway Re-

search Program publication 279, which addresses various intersection applications.

### Angle of Approach

Right-angle intersections provide a better view of traffic and provide better turning movements than roads intersecting at acute angles. A recommended practice is to limit the angle of approach of the intersecting road to 60° (relative to the through street). In those situations involving intersections of legs at acute angles, the engineer should consider slight changes to provide the desired 90° alignment.

Figure 20.32 shows how an approach street with an acute angle of approach can be modified to obtain a 90° intersection. On local roads, such an alignment is acceptable. On higher-category streets, this type of alignment should be used cautiously. Although this type of alignment may improve visibility and turning movements, there is a disadvantage to this alignment; it obscures traffic-control devices, and vehicles approaching the intersection through the short radius curve tend to smooth out the curve by encroaching into the opposite lane, creating a traffic hazard for vehicles turning from the main road.

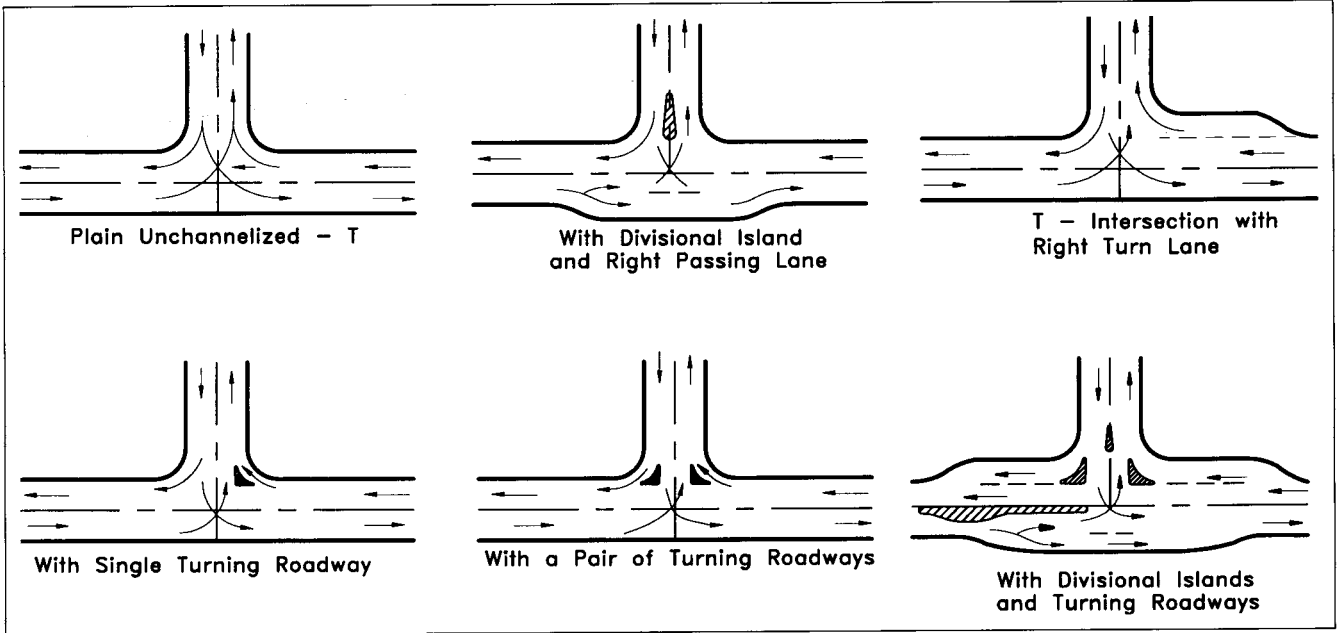
Within a relatively large project, adjusting lot lines and street configurations can attain proper alignment of intersections. For those projects that must access near or at an existing intersection, local agencies may require extensive modifications of the intersection to accommodate the additional volume of traffic. This can be costly and should be identified early in the project planning.

### Intersection Sight Distance

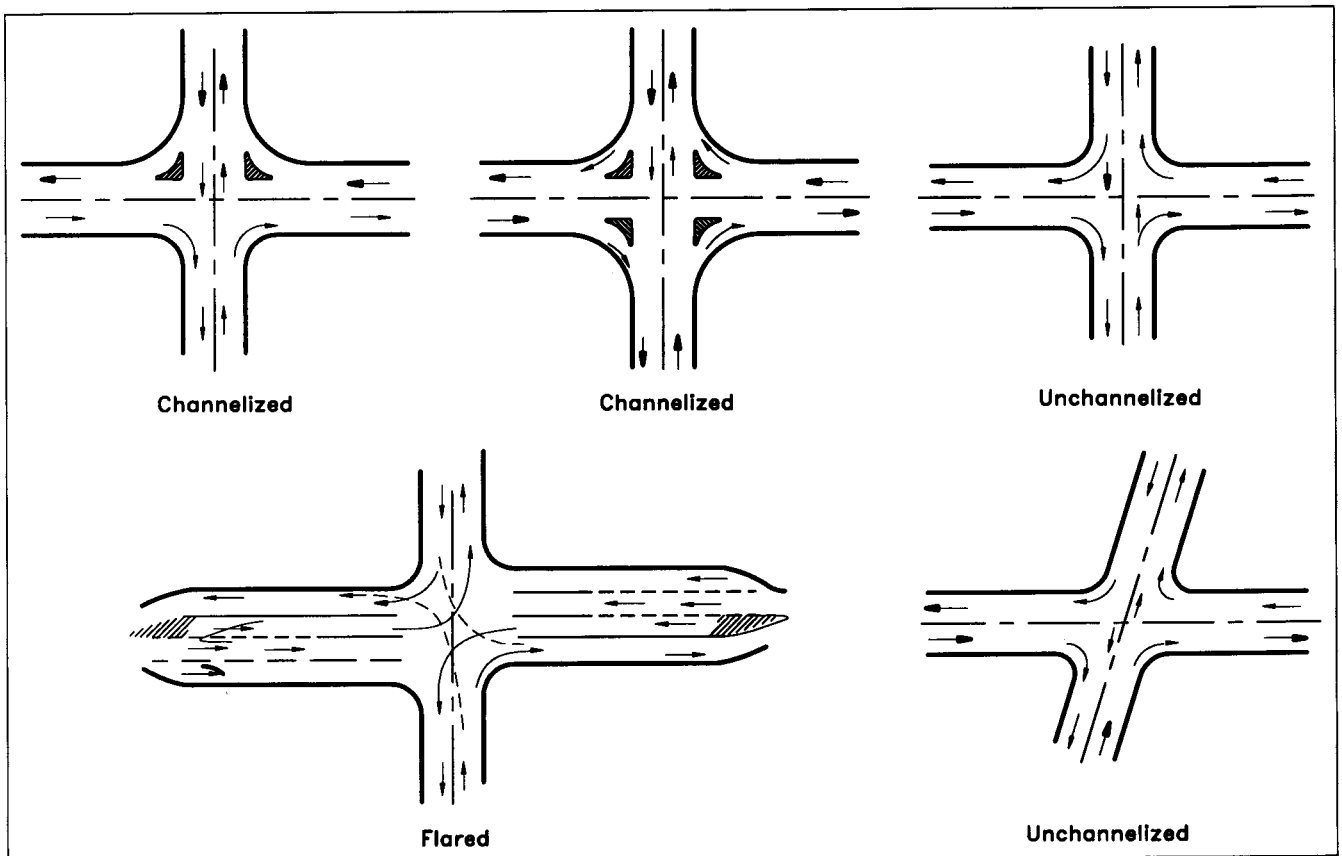
Intersection sight distance for an at-grade intersection is subject to the type of traffic control at the intersection. For most land development projects, the most prevalent type of traffic control is the stop control. The vehicle on the minor street is required to stop completely, ensure that movement through the intersection can be safely done without colliding with oncoming traffic, and then proceed.

The driver on the minor road must have an unobstructed view for a sufficient distance in order to ensure that movement through the intersection can be safely accomplished. This sufficient sight distance depends on the speed of the oncoming vehicles and some nominal perception–reaction time representative of most drivers as well as the physical conditions of the intersection. Figure 20.33 shows the three types of movements for the stopped vehicle and the corresponding sight triangles.

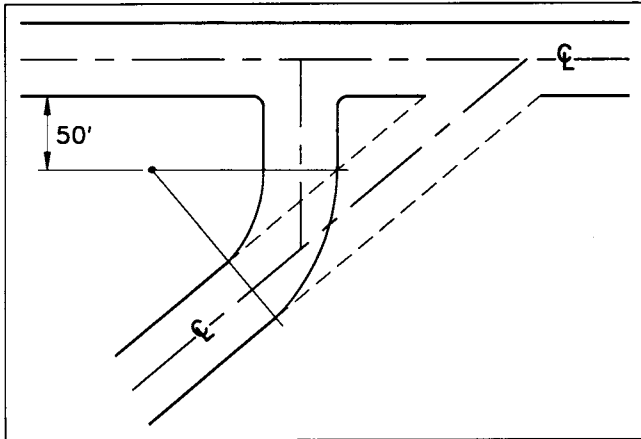
1. The vehicle can proceed through the intersection continuing along the minor street (four-legged intersections only). This requires that the vehicle clear traffic approaching from either direction of the major street and also vehicles turning left from the opposite direction on the minor street
2. The stopped vehicle can turn left onto the major street, which again requires clearing traffic from both directions



**FIGURE 20.30** Types of T intersections. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.



**FIGURE 20.31** General types of four-leg intersections. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.



**FIGURE 20.32** Realignment of intersecting road to obtain a 90° configuration.

3. The stopped vehicle can turn right, which requires clearing traffic coming from the driver's left.

The travel time for a vehicle to travel distances  $d_1$  and  $d_2$  in the figure at the design speed is the same time as it takes the stopped vehicle to clear the intersection.

The 20-ft setback shown on the minor street is the assumed location of the driver's eye. According to AASHTO (1990), the 20-ft location is measured from the edge of pavement or curb line and is 3.5 ft high. The unobstructed sight line must be able to see an object 4.25 ft high.

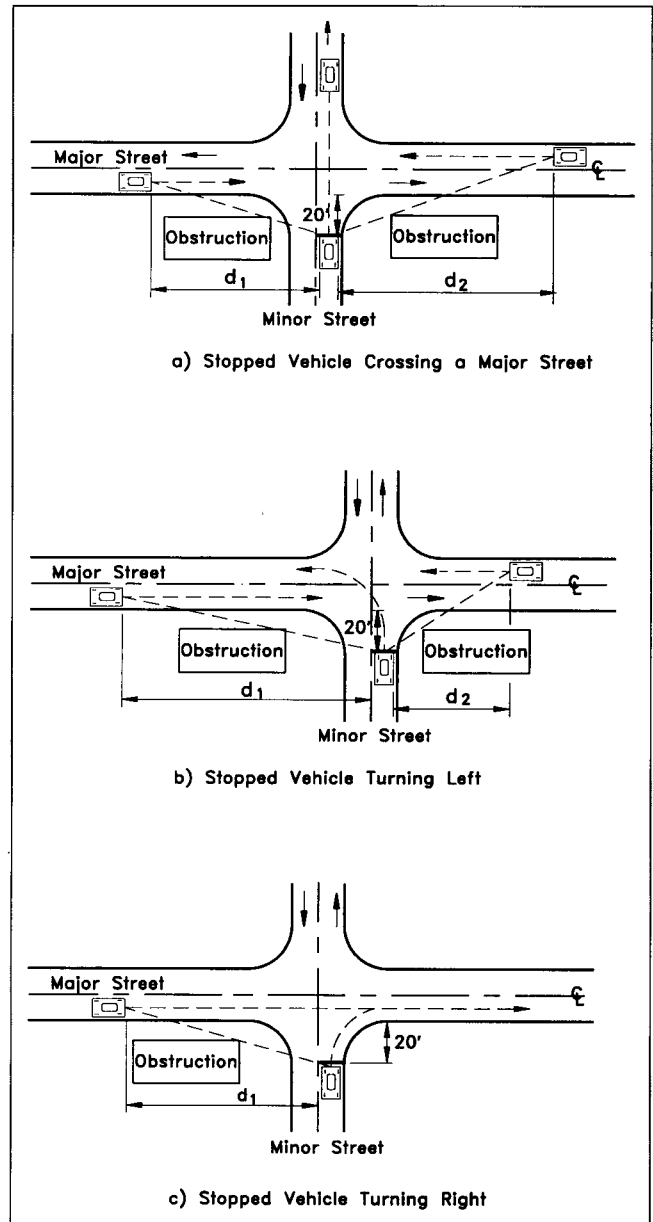
AASHTO (1990) provides criteria for calculating the time needed for several different types of vehicles to make a crossing maneuver, and the reader is encouraged to refer to this manual or local highway design criteria to establish  $d_1$  and  $d_2$ . Without the benefit of a specific method, a reasonable rule-of-thumb applicable to local-local and local-collector intersections is to allow 10 feet of distance for every 1 mph of design speed. For example, an intersection where the design speed of the major street is 35 mph requires a sight distance of 350 ft.

Signal control intersections have signals for traffic control. Intersections of this type exist most often on streets at the collector or arterial level.

The preceding discussions on sight distance for at-grade intersections assume that grades on approach legs are level. When this is not the case, there is a need for adjustments to values affected by grades such as stopping sight distance. Also, it is reasonable to assume the height of eye at 3.5 ft and height of object is 4.25 ft. Therefore, nothing in the line of sight, such as fences and vegetation, should be higher than this value.

### Spacing

The type of control at the intersection influences spacing of intersections. For example, several consecutive intersections with signal control where synchronization is necessary will be spaced farther than stop/yield control intersections in a



**FIGURE 20.33** Site triangle for three types of movements through an intersection. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

residential area. On local streets in residential areas, recommended minimum spacing is 125 ft. On collector streets, spacing should be increased to 250 ft.

In commercial areas, there are frequently numerous entrances to the office and retail areas accessing the main street. Although these are not intersections as defined earlier, the volume of traffic and the numerous turning movements require these entrances to be treated as intersections. The residential counterparts, that is, driveways, are not considered as intersections because of the low volume and few consecutive turning movements. Recommended spacing of

commercial entrances is also around 200 ft, depending on the traffic on the serving road and the anticipated traffic through each entrance. Spacing may be reduced if the turning movements into and out of the entrances are limited to the direction of flow of traffic; that is, right-in and right-out movements.

Intersection spacing is always a matter for consideration in maintaining safe streets and avoiding traffic queuing. Excessive queuing is not a critical design element on low volume streets. In most cases, intersection spacing of 125 ft eliminates traffic flow problems. Such spacing also prevents corner cutting from one street to another.

### Curb Return Radius

The ability for a vehicle to turn safely and effectively depends on such factors as type of vehicle, pavement width, curb return radius, speed through the turn, longitudinal and transverse pavement slope and skew angle of the intersection. Ideally, as the vehicle makes a turning movement, it remains within the limits of the designated lane limits. On local streets, for which the design vehicle is predominantly small and the design speeds and traffic volumes are low, whether the vehicle stays within the confines of the designated travel lane is of minor consequence. The radius of curvature of the curb return affects the width of the entrance of the street. Since pedestrian safety is paramount in residential areas and speeding is discouraged, short radius curb returns are more desirable.

In commercial and industrial developments, pavement widths and curb returns are matched to accommodate larger vehicles and reduce (or eliminate) the need for the vehicle to encroach into other lanes during the turning movement. However, as the curb return radius increases, the entrance width increases and, consequently, lane designations through the intersection are obscured. The need for channelization by islands or special pavement markings to delineate the travel lanes then becomes a consideration.

According to the ASCE/NAHB/ULI's *Residential Street Design*, the recommended ranges for curb return radii are 15 ft to 20 ft at local–local street intersections, 25 ft to 30 ft for local–collector intersections, and 25 ft to 30 ft for collector–collector intersections.

However, most jurisdictions prescribe minimum radius of curvature for the curb returns. The engineer should use discretion in setting these radii. The curvature is based on the largest type of vehicle that will frequently use the street, pedestrian safety, and the cost for maintenance of the right of way and the loss of usable land for development.

When determining the curb return radii, consideration must be given for the largest vehicles that will use the street. Although large vehicles such as moving vans, motor homes, and buses are vehicles that occasionally traverse local streets and would not be selected as the design vehicle for local streets, consideration should be given to allow for their limited use. Accomplish this by checking larger vehicle requirements regarding turning and clearance. The use of

transparent vehicle templates made to the same scale of the site plan assist in checking these provisions. An example of one template is shown in Figure 20.34. Other templates for the vehicles listed in Table 20.12 are provided in the AASHTO manual.

Dimensions for the appropriate design vehicle and the minimum turning radii for 15 different vehicle types are listed in Table 20.12.

### Profile Grade Line (PGL) at Intersections

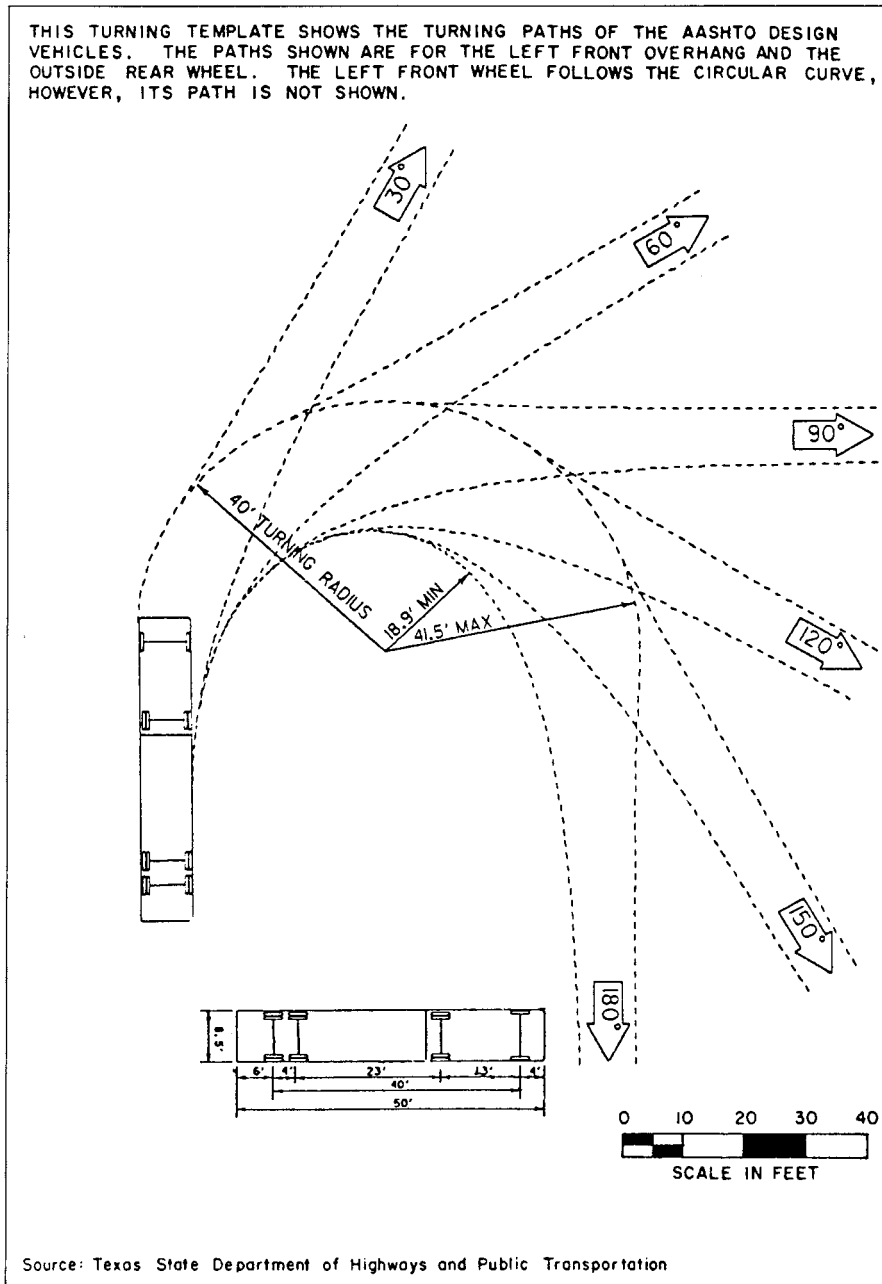
Proper alignment of street profiles at intersections provides a smooth transition through the intersection. The profile requirements of the intersecting side street vary significantly depending on the type of intersection control, that is, whether the intersection is signalized or has a stop sign, and, to a lesser extent, the design speed of the side street and whether it is a three-leg or four-leg type of intersection. If signalized, a portion of the traffic will arrive on the green indication and continue through the intersection without significantly slowing down. In this case, the design speed through the intersection will impact the profile design. With stop sign conditions, the design speed is negligible for the PGL design.

Ideally, the longitudinal approach gradient of the intersecting street should be the same as the cross slope of the through street. However this is rarely accomplished due to a mismatch of the main roadway cross slope with the site and design constraints that may impact the approach gradient of the side street. Some means is necessary to provide smooth transition between the PGL of the side street with the cross slope of the through street. For practical considerations, there are three basic methods in establishing the profile grade line for the intersecting street, as illustrated in Figure 20.35. The design standards for the review agency dictate which method, if any, and the acceptable values permitted.

Figure 20.35(a) shows the transition as just a grade break, that is, the algebraic difference of the gradients. This method is typically used for stop conditions and extremely slow speeds through the intersection. This also includes the speeds of the vehicles turning from the through street onto the side street. An algebraic difference of about 4 to 5% may be permitted. Higher values of the grade break have been used; however, the bouncing effects of the car over the grade break could cause riding discomfort and cause the underside of some vehicles to scrape the pavement.

In Figure 20.35(b) a short vertical curve of 50 to 100 ft provides a smoother transition when the grade break becomes too large. For slow speeds through the intersection, the vertical curve site distance as dictated by Eq. (20.35) and (20.36) is negligible.

Figure 20.35(c) provides the best possible way to connect the two PGLs for the combined conditions of higher design speeds of the side street, signalized control and for gradients with large algebraic differences. A section of nearly level roadway at the side street provides a staging



**FIGURE 20.34** Minimum turning path for WB-40 design vehicle. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

area for stopped vehicles, improves sight distance, allows the stopped vehicles to begin their movement without drifting backwards or from having to hold the brake pedal too stiffly to keep the vehicle from rolling during the red indication. Additionally, this method, with appropriate vertical curves, provides a smoother ride through a four-leg intersection. Although not specifically addressed in most design manuals, the length of the staging section should be based on intersection conditions, good engineering judgment, and possibly stopping distance for the design speed.

Finally, note that as a minimum, the tangent point of the vertical curve should be located outside the edge of pavement (EP) of the through street.

Matching the PGL alignment becomes more difficult when a superelevated through street must be matched to a side street with a longitudinal slope in the opposite direction. Factors such as sight distance, design speed, tangent gradients, intersection control, and other items must be considered to determine a safe and cost effective design. Frequently, a short vertical curve connecting the two op-

**TABLE 20.12 Minimum Turning Radii of Design Vehicles**

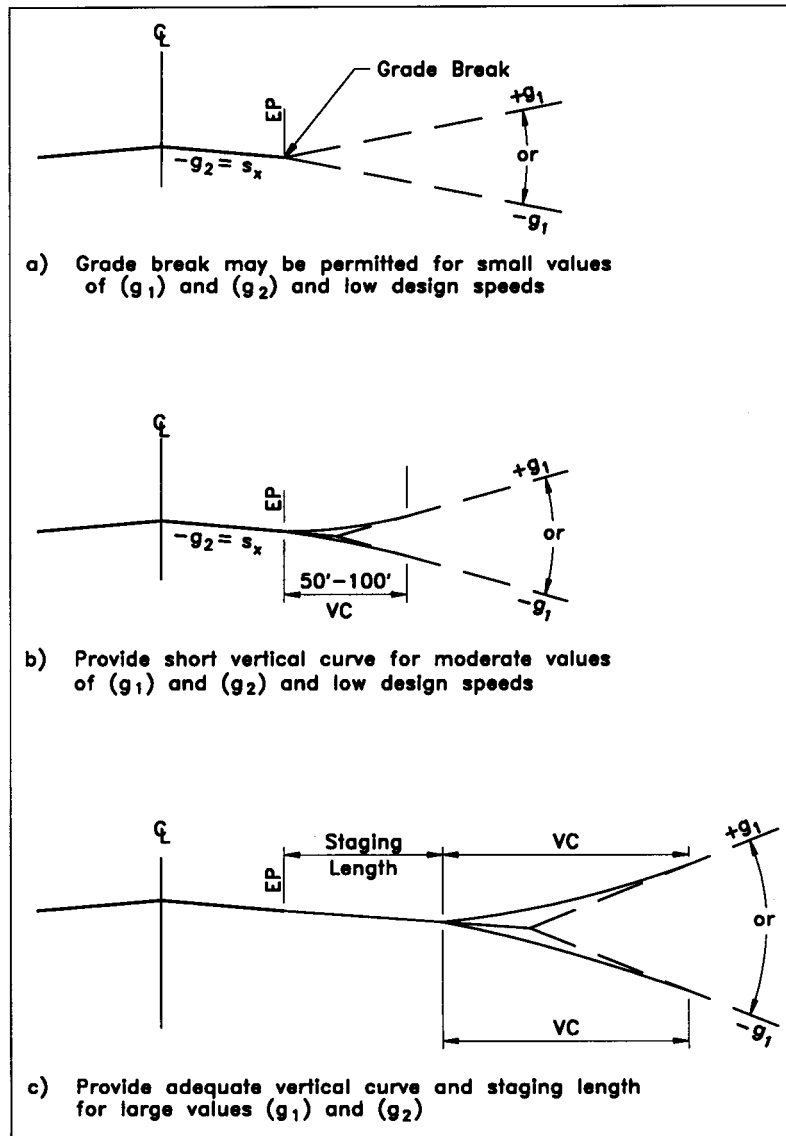
DESIGN VEHICLE TYPE	PASSENGER CAR	SINGLE UNIT TRUCK	SINGLE UNIT BUS	ARTICULATED BUS	SEMITRAILER INTERMEDIATE	SEMITRAILER COMBINATION LARGE	SEMITRAILER FULL TRAILER COMBINATION	INTERSTATE SEMI-TRAILER	INTERSTATE SEMI-TRAILER	TRIPLE SEMI-TRAILER	TURPIKE DOUBLE SEMI-TRAILER	MOTOR HOME	PASSENGER CAR WITH TRAVEL TRAILER	PASSENGER CAR WITH BOAT AND TRAILER	MOTOR HOME AND BOAT TRAILER
Symbol	P	SU	BUS	A-BUS	WB-40	WB-50	WB-60	WB-62*	WB-67**	WB-96	WB-114	MH	P/T	P/B	MH/B
Minimum design turning radius (ft)	24	42	42	38	40	45	45	45	45	50	60	40	24	24	50
Minimum inside radius (ft)	13.8	27.8	24.4	14.0	18.9	19.2	22.2	9.1	00	20.7	17	26.0	2.0	6.5	35

(From *A Policy on Geometric Design of Highways and Streets*, Table II-2, p. 22, AASHTO, Washington, D.C., 1990.)

\* Design vehicle with 48-ft trailer as adopted in 1982 STAA (Surface Transportation Assistance Act)

\*\* Design vehicle with 53-ft trailer as grandfathered in 1982 STAA (Surface Transportation Assistance Act)





**FIGURE 20.35** PGL for connecting streets at intersections.

posing slopes does not allow for sufficient sight distance or does not provide a suitable landing area. Use of a longer vertical curve may require extensive grading.

The situation becomes more complex for skewed intersections or as the intersecting street is on a curve that connects to the through street that is in superelevation or is in transition. For this reason,  $90^\circ$  intersections are strongly recommended to reduce the design headaches associated with the more complex situations. Furthermore, review agencies are less receptive to such irregular designs.

Another frequently occurring problem in intersection design is matching the curb returns—especially when the radius is very large and the arc length is long. It is not uncommon for the contractor to have to warp the pavement at the intersections to match pavements and slopes. For the most part, a good alignment design minimizes the amount

of warping done in the field to accommodate matching pavements.

It is good design practice in such situations to plot the profiles of the curb returns. This helps for setting stakeout grade for construction, ensures positive drainage around the return, and permits a smooth continuous curb return. Selecting a mathematical vertical curve to the curb return profile is desirable, but for the cases where a true mathematical fit cannot be accomplished, the designer draws a smooth curve using a parabolic template. This is commonly known as a spline curve. The curb return elevations are scaled directly from the smoothly drafted spline curve.

### Capacity at Intersections

For those projects where the development is large enough to impact significantly the traffic volumes on the existing

streets, or where the project includes intersections of high-volume roads such as collector–collector or collector–minor arterial, the land design team should employ the services of a traffic consultant to perform a traffic analysis. Included in this analysis will be, in most cases, a capacity analysis. By definition, a highway facility's capacity is the maximum flow rate (in vehicles per hour) at which vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a 15-minute time period under prevailing roadway, traffic, and control conditions. The capacity of major intersections is a major design consideration. A capacity analysis will lead to ways to improve under-capacity intersections to acceptable levels of service.

Level of service (LOS) is a qualitative description of the operational traffic conditions of a highway facility. Factors used in a LOS description include such items as speed, travel time, freedom to maneuver, traffic interruptions, comfort, convenience, and safety. The six classifications of level of service are:

1. LOS A: Free flow of traffic; users are affected by the presence of others; an excellent general level of comfort and convenience to the users
2. LOS B: Stable traffic flow but the presence of other users begins to be noticeable and affects individual behavior; slight decline in the level of comfort and convenience relative to LOS A
3. LOS C: Still stable traffic flow; however, the operation of individual users begins to become significantly affected by interactions with other users; noticeable decline in the level of comfort and convenience
4. LOS D: High-density stable flow with restrictions on operations by the users marked by a poor level of comfort and convenience
5. LOS E: Operating conditions at or near capacity; comfort and convenience levels are very poor and driver and pedestrian frustration is generally high
6. LOS F: Forced or breakdown flow resulting in unstable flow; the facility is over capacity and queues begin to form

Typically, the traffic consultant submits a traffic engineering report to the project engineer. In it are recommendations on how to improve intersection capacity through such means as signal control, turn lanes, channelization and other site-specific suggestions.

### Channelization

Wide street entrances at intersections, such as those generated by large curb return radii or skewed intersecting streets, create large areas of pavement that confuse drivers and promote irregular vehicle movements. To mitigate these

conditional hazards, painted pavement markings or raised islands are installed to properly direct the traffic flow. Raised islands also serve as pedestrian refuge points, separate opposing traffic, and serve as attractive aesthetic focal areas for the development when landscaped. In the latter case, however, the landscaping must not impede site distances nor obstruct the driver's view of pedestrians.

The size, configuration, and method of channelization is very site specific. It depends on the volume of traffic, the type of intersection control, the geometry of the intersecting streets, and most importantly the requirements and standards of the approving public agencies.

The islands must be large enough to be visible by drivers for each turning movement affected by the island. AASHTO recommends that the island area not be less than 50 ft<sup>2</sup> as an absolute minimum and preferably 100 ft<sup>2</sup> in size. Triangular islands should be 12 ft long on a side not including the rounding at the corners. Raised curb islands are set 2 ft to 4 ft from the edge of pavement of the through street on low-speed roads. The setback increases with increasing design speed on the through street (see Figure 20.36).

### Crest and Sag Vertical Curves with Respect to Intersections

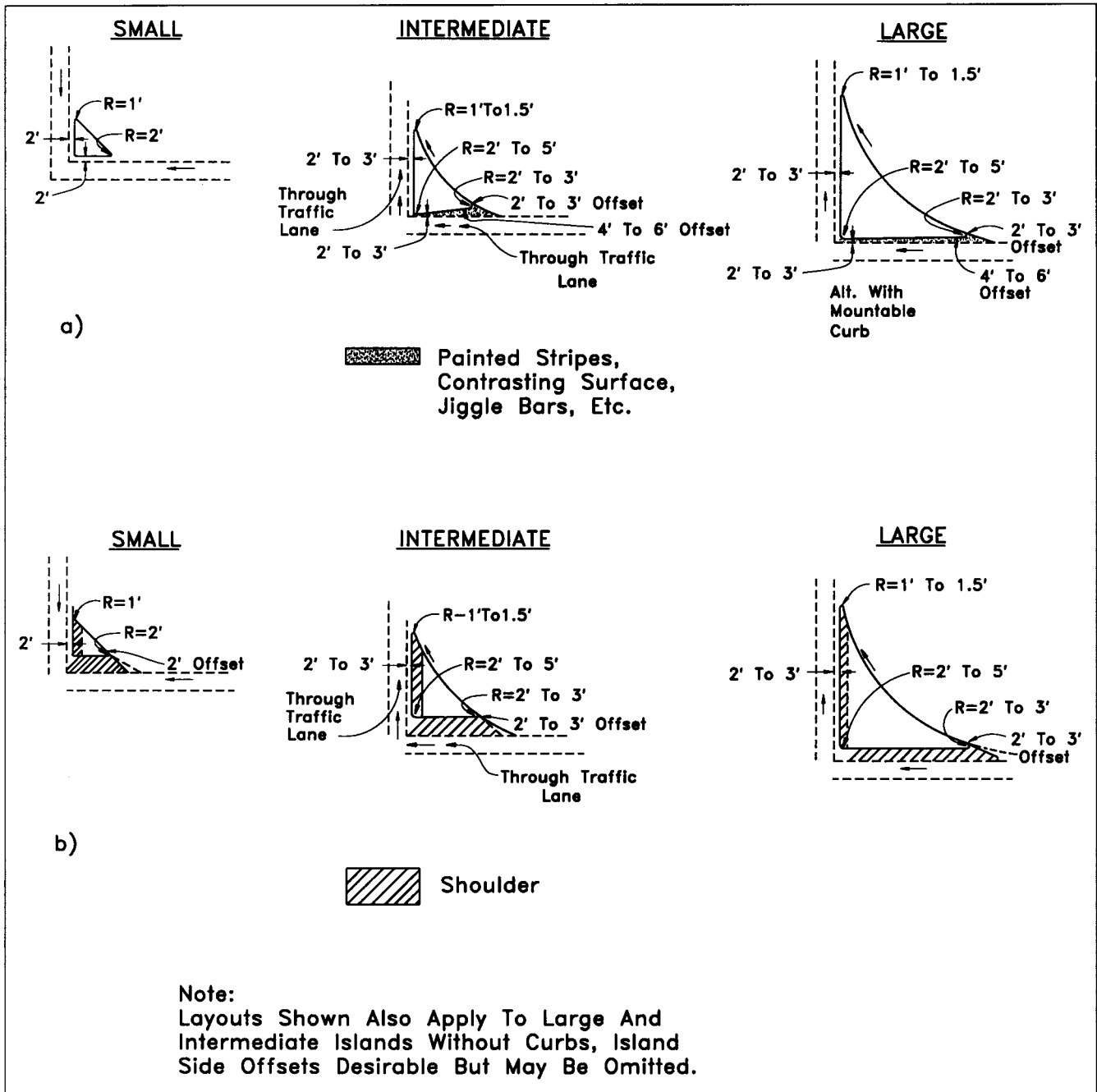
Intersection location on vertical curves must meet site distance requirements. This is more critical on crest curves than on sag curves due to the more restricted limits inherent in crest curves. Intersection sight distance is generally ten times the design speed (or greater). On short vertical curves with relatively steep tangent gradients, the optimum location is at or very near the top of the crest. As the length of the crest curve increases and the gradients flatten, the intersecting street can be located farther away from the crest.

On sag curves, site distance is a secondary problem as compared to drainage. Because of the potential drainage problems, the intersecting street should not be located at or very near the sump area of a sag curve. The location of the intersecting street depends on the length of the vertical curve and the tangent gradients.

### Street Layout

Considering the numerous issues that are factored into street design, the street layout, like any design process, is iterative. Not all of the issues can be addressed nor are all solutions apparent on the first try. Following is a general procedure for street layout.

1. Obtain an appropriate scale of the plan view of the project showing topography and all existing features. The scale and contour interval depend on size of the project and the availability of existing topographic maps. For large projects, e.g., greater than 100 acres, 1" = 100' and 5-ft contour intervals are acceptable for preliminary layouts. The scale of the plan view in-



**FIGURE 20.36** Details of triangular islands design: (a) curbed islands, no shoulders; (b) with shoulders. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

creases as the size of the project decreases. (It is very difficult to use a worksheet at 100 scale if the site is only a few acres.)

2. On the drawing worksheet, identify the site constraints and locate the points of access to the existing road network taking into consideration sight distances and proximity to existing intersections. Local ordinances may require that sections of the proposed road

be extended to the boundaries of the site to provide future access to adjacent parcels.

3. Set the horizontal alignment of the street network of the site by drawing straight lines. These lines represent the centerline tangents of the proposed roads. To minimize grading, the tangents should run closer to parallel with the existing contours rather than perpendicular.

4. After the tangents have been established and comply with the physical constraints of the site, horizontal curves are drawn. The radius of the curves must comply with the given criteria regarding curvature, design speed, design vehicle, and street category.

5. Prepare a profile view of the existing topography of the PGL for the road and include all vertical constraints such as culvert and utility crossings. With a profile of the existing ground elevation, draw a series of straight lines, which represent the proposed longitudinal grade of the street. The maximum and minimum gradients selected should comply with local requirements.

6. Select vertical curves to meet site distance and drainage constraints. Set the vertical curves on the profiles using appropriate curve length with the required vertical curve sight distance. Label the stations of the PVCs, PVTs, and high and low points. Show the length of the vertical curve and the *K*-value (or SSD). See Figure 20.39, for example.

General guidelines for establishing vertical alignment include:

- Avoid long lengths of steep grades.
- Place steeper grades at the beginning of an ascent followed by flatter grades at the peak.
- Reduce gradients at intersections.

7. Determine and depict the right-of-way widths of the streets as per local regulations. Using the zoning requirements (e.g., minimum lot size and widths), establish the lot lines. As the lot lines are set, the horizontal alignment may have to be adjusted to make the lot lines work.

8. Perform an earthwork computation. Adjust the street alignment to balance the earthwork.

9. Repeat as necessary to optimize the lot layout, the earthwork balances, and other constraints.

Lastly, the designer must coordinate the horizontal and vertical alignments. Often they are designed independently without consideration of the combined effects. When all specific criteria such as minimum radius of curvature, sight distance, optimizing land use, intersection alignment and configuration, and balancing earthworks, have been accommodated, the combined effects of the horizontal and vertical alignments need to be checked for compatibility.

One method to help visualize the complete picture is to develop the grading plan. In addition to showing how the horizontal and vertical curves are aligned, the grading plan shows the location of the deep excavation and embankments and where and how the grading encroaches on the

lots and adjacent properties. Another way to compare the horizontal with the vertical alignment is to set the profile sheet on a table with the plan sheet such that the stationing of one sheet is aligned with the stationing with the other view. This is most effective as the plan view scale matches the horizontal scale of the profile view. A common practice is to show the plan and profile view on one drawing sheet (see Figure 20.39).

Considerations for ensuring that the horizontal alignments are compatible with the vertical alignment are:

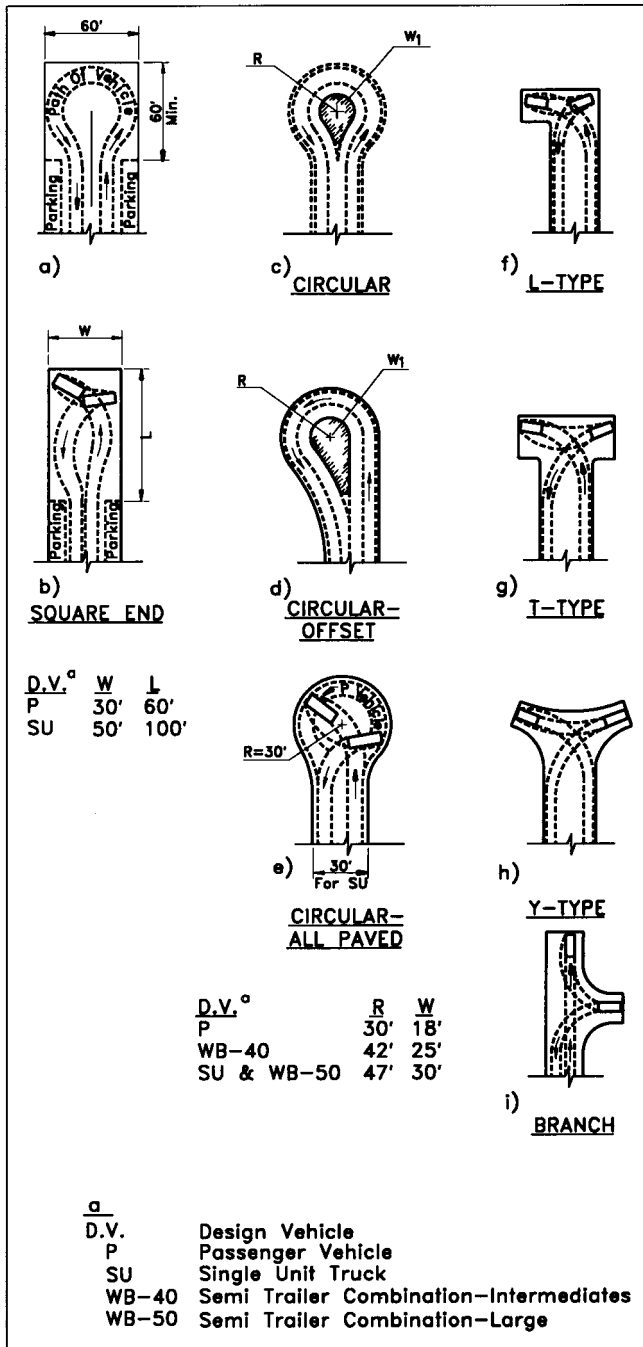
1. Flatten profiles at intersections to allow for better visibility, starting, stopping, and turning movements.
2. Avoid series of short vertical curves that give a rollercoaster effect regardless of the horizontal alignment.
3. Avoid a sag vertical curve at the beginning or end of a sharp horizontal curve.
4. Avoid short vertical curves through compound curves.
5. Avoid a sharp horizontal curve at the bottom of a long grade—especially a steep grade!
6. Avoid crest curve on a short tangent between two curves.
7. Avoid sharp horizontal curve near the top of a vertical curve.

In general, roads aligned perpendicular to the contours will cause buildings to have one side extend further out of the ground. This type of alignment can also create steep grades and extensive earthwork. Roads aligned parallel to the contours can cause buildings to be lower than the road if the road is located at the outer edges of the slope. This type of alignment typically requires less earthwork and does not create steep slopes.

Each site is unique and all of the above guidelines, in all likelihood, cannot always be implemented. Compromises will have to be made, and it is the design team's responsibility to devise a layout that is safe, aesthetically attractive, economically viable, and optimizes land use.

### Cul-de-sac Design

The cul-de-sac is a popular street type for use on local streets. Cul-de-sac streets are open at one end and provide a turning area at the closed end. Although the circular cul-de-sac and its variations may be the most widely used, other various types of turning areas include circular, T, and Y, among others. Some of these are depicted in Figure 20.37. The T and Y type require more inconvenient backing movements by the user but in low-volume situations conserve land and allow increased flexibility in the land planning process. The type chosen for particular situations depends on the primary type of vehicle being serviced by the turning



**FIGURE 20.37** Types of cul-de-sacs. From *A Policy on Geometric Design of Highways and Streets*, copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

area, traffic volume and most importantly the standards accepted by the approving agency.

Because it is open only at one end, cul-de-sacs are limited to an average daily traffic volume of about 200 vehicles per day. If it is assumed that a single-family residence generates ten trips per day, then 20 residences will generate a 200 ADT. Lot widths range from about 65 ft for small lots to 100 ft for moderately sized lots. Based on these assump-

tions, the maximum length of a cul-de-sac street is generally 650 ft to 1000 ft.

The recommended radius for an all-paved cul-de-sac is 30 ft to 40 ft. Most passenger vehicles require a minimum radius of 30 ft to turn around. Small trucks and fire equipment can maneuver a turnaround with one or two backing movements on 40-ft radius cul-de-sacs. However, this presumes no vehicles are parked within the turnaround area.

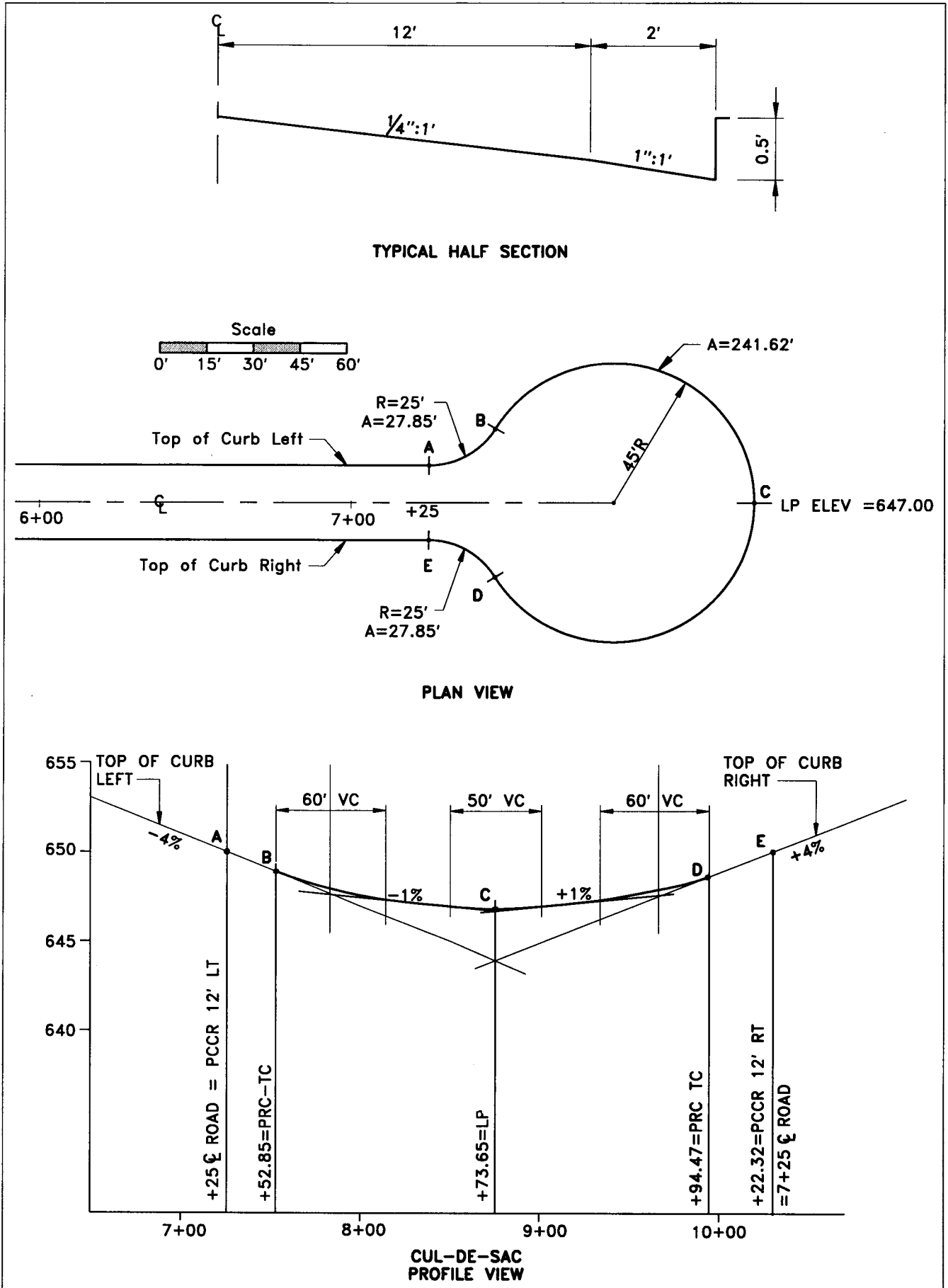
For practical reasons, cul-de-sacs should be relatively level. Pavement grades across the cul-de-sac should ensure adequate drainage over the pavement and yet allow a vehicle to access adjacent land. A vehicle should be able to turn around without negotiating steep grades. Maximum pavement gradients across the cul-de-sac of 3 to 4% are recommended. The street grade entering the cul-de-sac should be between 3 to 5%. To ensure proper design, the edge of pavement or top of curb around the perimeter of the cul-de-sac should be profiled. Spot elevations around the cul-de-sac based on the profile aid the contractor in construction. An example illustrates one method for designing a cul-de-sac.

**EXAMPLE 5:** Consider a circular cul-de-sac with a 45-ft radius. The cul-de-sac and typical section of the road are shown in Figure 20.38. Also for this problem, assume site constraints dictate the elevation at a position where the centerline of the road extended to the curb of the turnaround area must be 647.0 ft (shown as point C in the plan view of Figure 20.38).

1. To facilitate design and to insure an acceptable gradient throughout the turnaround area of the circular cul-de-sac, the top of curb is used as the PGL. That is, the design is established by setting the profile of the top of curb around the cul-de-sac by “unwrapping” (i.e., straightening) the top-of-curb profile.

2. The profile of the top of curb begins at the PC of the top of curb left side, identified as point A. The stationing of point A is equal to the stationing at the centerline of the road at the point on the centerline that is perpendicular from point A. From geometry, the stations at points B, C, D, and E are computed.

3. Point E is directly across the street from A and represents the top of curb right side. The elevation of the top of curb at this point must equal the elevation at top of curb left side from symmetry. The geometry and profile of the street set these two elevations. The profile gradient away from point E (= +4%) must equal the profile gradient into point A, which is the same profile gradient established for the street. The



**FIGURE 20.38** Cul-de-sac typical section, plan and profile.

stations, elevations, and gradients at these points are plotted on the profile.

4. Between points A and E, a profile of the curb is established such that station  $8 + 73.65$  (point C) is at elevation 647.0 (see Figure 20.38). Using a 50-ft symmetrical vertical curve with the PVI at that station also provides the low point at that station. For this example, the tangent grades were selected as 1%. (Recall that a symmetrical vertical curve with  $|g_1| = |g_2|$  has the lowpoint at the PVI.) Since the elevation on the curve is known, the PVI elevation corresponding to a symmetrical 50-ft vertical curve and the selected 1% gradients is obtained using Eq. (20.45), the distance from the tangent to the point on the vertical curve. The PVI elevation is  $647.0 - 0.13 = 646.88$ . Correspondingly the PVC and PVT elevation is 647.13 at station  $8 + 48.65$  and  $8 + 98.65$ , respectively.

$$\frac{AL}{8} = \frac{(.02)(50)}{8} = 0.125 \text{ ft} \quad (20.45)$$

5. Two other vertical curves of 60-ft lengths are inserted to provide a smooth connection between the 4% and 1% gradients.

This example is included to illustrate how a circular cul-de-sac could be designed. The lengths of the vertical curves and tangent gradients were selected to provide a relatively flat area around most of the perimeter of the cul-de-sac. This would allow for smoother driveway connections. Site conditions and drainage considerations primarily dictate the lengths, location, and tangent gradients of curves.

### Typical Street Plan and Profile

Engineering drawings serve two main purposes: to assure the design review agencies that the design conforms to all codes, standards, and ordinances; and to provide the contractor with clear, unambiguous detail of how and where to construct the road.

The plan and profile of the road drawings (see Figure 20.39) should be appropriately scaled. The PGL reference line (usually the centerline of the street) shows the stationing at 100-ft increments. This same stationing is used both in the plan and profile views. Station equalities at intersections are delineated in both views along with the stations where the proposed roads match to the existing roads or end at boundary limits. Many times this same reference line and stationing are used to locate proposed utilities, storm sewers, and other construction-related features.

In the plan view, basic horizontal curve information includes: stationing of the PC and PT points, radius, arc length, and delta angle. Tangent length, chord length and chord bearing are helpful but not necessary. The vertical

curve information in the profile shows the tangent gradients, stationing of the PVC, PVT, and PVI, with elevations with respect to the PGL. If the road is superelevated, the plan view shows the stationing where a normal crown ends and the transition to superelevation begins and the stationing where the transition ends. The section of roadway that is in full superelevation is shown along with the rate of superelevation.

Elevations along the PGL are shown on the profile view every 50 ft along the longitudinal tangent. Through vertical curves the elevations are shown in shorter increments—every 25 ft in normal circumstances and less for short vertical curves. In addition, elevations are frequently shown for the left and right edge of pavement (EP) or for top of curb (TC). In complex situations, elevations might be shown every 10 ft and at locations in addition to the PGL (e.g., left and right EP or TC).

Many contractors construct the curb and gutter section before placing the pavement. It is helpful to provide the top of curb elevations and especially at all curb return points. The curb return points are often referenced to the centerline (C/L) stationing.

### Indicating Sight Distance on Plans and Profiles

Sight distance depends on the height of the driver's eye above the road surface, the height above the road surface of the visible object and height of obstructions in the line of sight. The height of the eye is assumed to be 3.5 ft for passenger type vehicles (8 ft for large trucks) and the height of the object is 6 in. for determining adequate sight distance on vertical curves. In land developments, the sightline is often restricted to the right-of-way line because landowners later may plant shrubbery to the limits of their property. Situations where sight lines cross limits outside the right of way can be resolved by including provisions restricting obstructions within the sightline area.

To assure reviewing agencies that adequate sight distance has been provided, the engineer might consider showing the lines of sight on horizontal curves, vertical curves, and at intersections. Local criteria may even require this detail on the plan and profile sheets. In most cases the stopping sight distances on vertical curves has been accounted if the ratio of the length of the curve to the algebraic difference of tangent gradients is greater than the  $K$  value. Likewise, criteria dictating an absolute minimum radius for horizontal curves put the line of sight within the pavement area and away from obstructions that would encroach the line of sight near the right of way. Nonetheless, a reasonable comfort level for the design can be attained with periodic spot checks for horizontal and vertical stopping sight distances.

Intersection sight distances should be checked always, both in plan view and profile view. In plan view, the location of the vehicle is a nominal distance back from the face of curb (or edge of pavement) of the through street. The required distance is measured along the centerline of the oncoming travel lane. The line of sight is the connection of

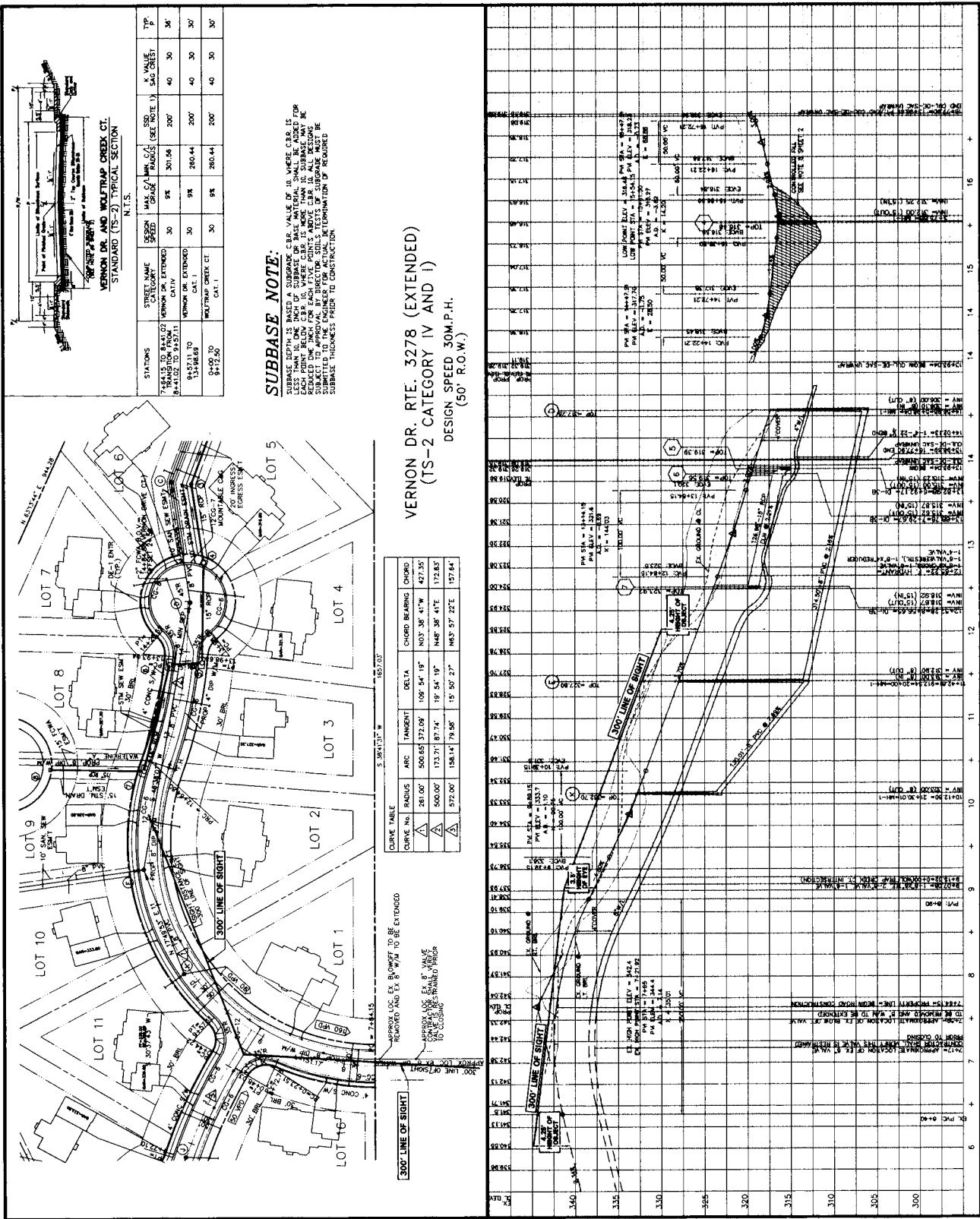


FIGURE 20.39 Typical street plan and profile; notice the labeling of the horizontal site distance at the street intersection.



**TABLE 20.13 Geometric Design Guides for Major Subdivision Streets<sup>1</sup>**

*PROJECTED TRAFFIC (ADT)	TERRAIN	MIN. DESIGN SPEED	MAXIMUM DEGREE OF CURVATURE	SUGG. MAX. % GRADE	MIN. STOPPING SIGHT DIST. <sup>2</sup>	MIN. INTERSECTION SIGHT DISTANCE <sup>3</sup>	WITHOUT CURB AND GUTTER			WITH CURB AND GUTTER		
							MINIMUM PAVEMENT WIDTH <sup>4</sup>	FILL w/Gr	FILL wo/Gr	WIDTH OF DITCH <sup>6</sup>	MINIMUM RIGHT OF WAY WIDTH <sup>7</sup>	MIN. DISTANCE FACE TO FACE OF CURBS <sup>8</sup>
251-400	Level	40	12°	7	275'	400'	20'	6'	4'	4'	36'	44'
	Rolling	30	22°	10	200'	300'	20'	6'	4'	4'	36'	44'
	Mountainous	20	110-ft rad	16	125'	200'	20'	6'	4'	4'	36'	44'
401-1000	Level	50	7°30'	6	400'	500'	22'	6'	4'	4'	30'	46'
	Rolling	40	12°	9	275'	400'	22'	6'	4'	4'	30'	46'
	Mountainous	30	22°	14	200'	300'	20'	6'	4'	4'	36'	44'
1001-2000	Level	50	7°30'	6	400'	500'	22'	9'	6'	6'	30'	46'
	Rolling	40	12°	9	275'	400'	22'	9'	6'	6'	30'	46'
	Mountainous	30	22°	14	200'	300'	20'	9'	6'	6'	36'	46'
2001-4000	Level	50	7°30'	6	400'	500'	24'	11'	8'	8'	40'	48'
	Rolling	40	12°	9	275'	400'	22'	11'	8'	8'	30'	46'
	Mountainous	30	22°	14	200'	300'	22'	11'	8'	8'	30'	46'
Over 4000	Level	50	7°30'	6	400'	500'	24'	11'	8'	8'	40'	48'
	Rolling	40	12°	9	275'	400'	24'	11'	8'	8'	40'	48'
	Mountainous	30	22°	14	200'	300'	24'	11'	8'	8'	40'	48'

<sup>1</sup>Roads functionally classified other than local are to be in accordance with the geometric design requirements in the current issue of the VDOT road and bridge standards for applicable road functional classification.

<sup>2</sup>Distances are based on a 3.5-ft height of eye and a 0.5-ft height of object.

<sup>3</sup>Distances are based on a 3.5-ft height of eye and a 4.25-ft height of object.

<sup>4</sup>A minimum level of service C is to be maintained. Additional lanes and/or channelized intersections may be required.

<sup>5</sup>Shoulder designs are to be in accordance with standard 65-12.

<sup>6</sup>Ditch slopes (from edge of shoulder to bottom of ditch) are to be 1-ft vertical for each 4-ft horizontal for 6-ft-wide ditches, and 1-ft-wide vertical for each 3-ft-wide horizontal for 4-ft wide ditches.

<sup>7</sup>A minimum 50-ft of right of way is required. Additional widening in increments of 10-ft as necessary to accommodate the roadway elements including cut and fill slopes and sidewalks where applicable, as required.

<sup>8</sup>Dimensions shown take into consideration that parking may be permitted.

<sup>9</sup>Where sidewalks are to be provided, additional right of way width to accommodate them is required.

Notes:

Each street should have continuity of design throughout, therefore, multiple or "step down" typical designs will not be acceptable except where a major traffic generator such as an intersection would delineate a clear line of declaration. Due to the normal density of development adjacent to residential subdivision streets, standard curve superelevation is not practical on local streets where the design speeds are 25 mph or less and no superelevation is required. Streets with design speeds exceeding 25 mph but no more than 40 mph are to be superelevated by an amount equal to the standard pavement comm. For all speeds exceeding 40 mph, superelevation and pavement widening should be provided in accordance with standard ic-4 (urban) of the department's road and bridge standards.

An adequate turnaround shall be provided at the end of dead end or cul-de-sac streets to allow safe maneuvering by service vehicles, school buses, and highway equipment.

For residential subdivisions where circular turnarounds are used, a minimum 30-ft radius is required, and for streets intended to service more than ten residences a 45-ft radius is recommended.

For nonresidential subdivisions where circular turnarounds are used, a minimum radius of 45 ft is required.

\*Based on traffic volume resulting from "complete development" (land).

(Source: Virginia Department of Transportation, Subdivision Street Requirements, 1989.)

**TABLE 20.14 Geometric Design Guides for Tertiary Subdivision Streets**

*PROJECTED TRAFFIC (ADT)	TERRAIN	MIN. DESIGN SPEED	MAXIMUM DEGREE OF CURVATURE	SUGG. MAX. % GRADE	MIN. STOPPING SIGHT DIST. <sup>2</sup>	MINIMUM INTERSECTION SIGHT DISTANCE <sup>3</sup>	MINIMUM PAVEMENT WIDTH <sup>4</sup>	WITHOUT CURB AND GUTTER			WITH CURB AND GUTTER			
								SHOULDER WIDTH <sup>5</sup>		WIDTH OF DITCH <sup>6</sup>	MINIMUM RIGHT OF WAY WIDTH	MIN. DISTANCE FACE TO FACE OF CURBS <sup>6</sup>	MINIMUM RIGHT OF WAY WIDTH <sup>8</sup>	
								FILL w/Gr	FILL wo/Gr					CUT
Up to 250	Level	30	22°	7	200'	300'	10'	6'	4'	4'	4'	40'	30' <sup>7</sup>	40'
	Rolling	30	22°	10	200'	300'	10'	6'	4'	4'	4'	40'	30' <sup>7</sup>	40'
	Mountainous	20	110-ft rad <sup>1</sup>	16	125'	200'	10'	6'	4'	4'	4'	40'	30' <sup>7</sup>	40'
251-400	Level	30	22°	7	200'	300'	20°	6'	4'	4'	4'	50'	30'	40'
	Rolling	30	22°	10	200'	300'	20'	6'	4'	4'	4'	50'	30'	40'
	Mountainous	20	100-ft rad <sup>1</sup>	16	125'	200'	20'	6'	4'	4'	4'	50'	30'	40'
401-1000	Level	30	22°	7	200'	300'	22'	6'	4'	4'	4'	50'	30'	46'
	Rolling	30	22°	10	200'	300'	22'	6'	4'	4'	4'	50'	30'	46'
	Mountainous	20	110-ft rad <sup>1</sup>	16	125'	200'	20'	6'	4'	4'	4'	50'	36'	46'

<sup>1</sup>When a 110-ft radius cannot be obtained in mountainous terrains, a minimum 90-ft radius may be used with a superelevation rate (e) of -0.02.

<sup>2</sup>Distances are based on a 3.5-ft height of eye and a 0.5' height of object.

<sup>3</sup>Distances are based on a 3.5-ft height of eye and a 4.25-ft height of object.

<sup>4</sup>Shoulder designs are to be in accordance with standard 65-12.

<sup>5</sup>Ditch slopes (from edge of shoulder to bottom of ditch) are to be 1-ft vertical for each 3-ft horizontal.

<sup>6</sup>Dimensions shown take into consideration that parking may be permitted except as provided for in footnote 7.

<sup>7</sup>If parking is prohibited, face to face curbs may be reduced to 24-ft on a minimum 30-ft right of way.

<sup>8</sup>Where sidewalks are to be provided, additional right of way to accommodate them will be required.

Notes:

Each street should have continuity of design throughout, therefore, multiple or "step down" typical designs will not be acceptable except where a major traffic generator such as an intersection provides a clear line of demarcation. Due to the nature of tertiary streets and their low design speed, curve superelevation is not required.

An adequate turnaround shall be provided at the end of dead end or cul-de-sac streets to allow safe maneuvering by service vehicles, school buses, and highway equipment. For residential subdivisions where circular turnarounds are used, a minimum 30-ft radius is required, and for streets intended to serve more than ten residences, a 45-ft radius is recommended.

For nonresidential subdivisions where circular turnarounds are used, a minimum radius of 45-ft is required.

\*Based on traffic volume resulting from "complete development" (land).

(Source: Virginia Department of Transportation, Subdivision Street Requirements, 1989.)

the two points. If the line of sight extends into private property, a sight easement may be required before the plans are approved.

To show adequate intersection sight distance in the profile view, the station of the intersection is plotted an elevation of 3.5 ft above the pavement elevation to represent the height of the eye. On either side of this station, another point is plotted at a distance equal to the required sight distance for intersections a height of 4.5 ft above the pavement. If these two points are unobstructed, then adequate sight distance for the intersection has been provided (see Figure 20.39).

### Sample Design Criteria for Local Streets

It has been emphasized throughout this chapter that standards and criteria dictating local street design are mainly controlled through local and state public agencies. An example for the design criteria for local streets as given in *Subdivision Street Requirements by Virginia Department of Highways* (1989) is presented in Tables 20.13 and 20.14.

### REFERENCES

American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, AASHTO, Washington, D.C., 1990.

Hickerson, Thomas F., *Route Location and Design*, 5th ed., McGraw Hill, New York, 1964.

Pline, James L. ed., *Traffic Engineering Handbook*, 4th ed., Institute of Transportation Engineers, Prentice Hall, Englewood Cliffs, N.J., 1992.

Garber, Nicholas J., and Lester A. Hoel, *Traffic Engineering and Highway Engineering*, West Publishing, St. Paul, Minn., 1988.

Stover, Vergil G., and Frank J. Koepke, *Transportation and Land Development*, Institute of Transportation Engineers, Prentice Hall, Englewood Cliffs, N.J., 1988.

Homburger, Wolfgang S. et al., *Residential Street Design and Traffic Control*, Institute of Transportation Engineers, Prentice Hall, Englewood Cliffs, N.J., 1989.

Institute of Transportation Engineers, *Recommended Guidelines for Subdivision Streets*, ITE, Washington, D.C., 1989.

American Society of Civil Engineers, National Association of Home Builders, and Urban Land Institute, *Residential Streets*, 2nd ed., ASCE, NAHB, ULI, 1990.

Highway Research, *Highway Capacity Manual*, Special Report no. 209, Highway Research, Washington, D.C., 1985.

Virginia Department of Transportation, *Subdivision Street Requirements*, Virginia Department of Transportation, Richmond, Va., 1989.

Yoder, E.J., and M.W. Witczak, *Principles of Pavement Design*, 2nd ed., John Wiley, New York, 1975.

## CHAPTER 21

# STORM DRAIN DESIGN

*William S. Springer, PE*

### INTRODUCTION

Drainage systems are divided into two categories: major and minor. The minor system, which consists of swales, small ditches, gutters, small pipes and the other various types of inlets and catch basins, collects and conveys runoff to a discharge area or impoundment. Components in the minor system are sized to carry runoff generated by the more frequent, short duration, storm events. The major drainage system includes natural streams, channels, ponds, lakes, retention and detention facilities, large pipes, and culverts (see Figure 21.1). Design criteria for the major system are based on significant amounts of rainfall produced by the less frequent, long duration, storms.

Storm drain design requires two basic types of analyses: the hydrologic aspect of estimating runoff and the hydraulic aspect of sizing the components. Although there are numerous techniques to estimate runoff, the one selected for a particular component's design depends on the following:

- What is being designed
- Type of input data available
- Type of output data required
- Cost effectiveness of the technique
- Required accuracy
- Size of watershed
- Accepted design standards of the approving agency or agencies

Generally, the design of minor system components only requires the determination of peak runoff discharges,

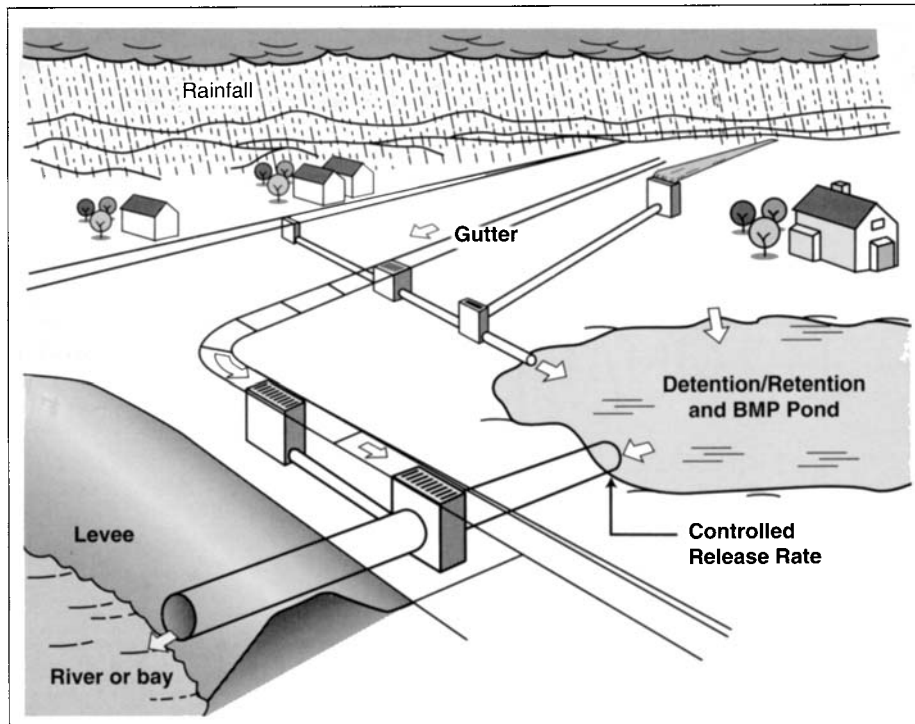
whereas major system components require not only the peak discharges but the time variation of runoff for an effective design. This chapter will focus on computing runoff for small drainage areas for the purpose of sizing minor system components, such as surface inlets and pipe systems. Analysis of minor systems includes hydraulic head loss through the system and hydraulic grade line computations. Also included in this chapter is a discussion on culvert design and analysis.

### LEGAL ASPECTS OF DRAINAGE

The laws regarding drainage can be viewed as a subset of a much larger body of law dealing with water resources. Drainage law primarily concerns the diversion and distribution of surface waters, and although it is under the umbrella of federal law, drainage law is largely governed by state and local law.

Since multiple sources of state and local law exist, the land development engineer should be familiar with the specific law governing the diversion and distribution of surface waters in the place where the land is located. Law libraries and local public libraries, as well as state and local agencies, can provide assistance in acquiring legal resources and information. Because of the variation in state and local law, only certain fundamental doctrines of drainage law are addressed in this handbook. For further assistance and guidance, the engineer may wish to consult an experienced attorney.

By introducing the reader to the fundamental concepts on which most states' drainage laws are based, the reader will be aware of some potential legal problems related to drainage law and be in a position to consult with legal counsel to address the issues involved.



**FIGURE 21.1** Typical urban drainage system. Reprinted with permission from Kibler, David F., ed., *Urban Stormwater Hydrology Monograph 7*. Washington, D.C.: Copyright 1982 by the American Geophysical Union.

Most engineers are familiar with the scientific classification of water as it exists in its various forms (e.g., groundwater and surface runoff). Unfortunately there is an inconsistency between the scientific classifications and the legal classifications. Moreover, the legal classifications of water and its forms vary according to an individual state's legal definition. To further complicate the matter, laws governing watercourses are different from those governing surface waters, the latter of which are altogether different from those governing groundwater. The engineer, when consulting legal counsel, needs to clarify the scientific classification from the legal classification before applying appropriate legal principles.

Most state and local courts recognize that the landowner also owns the diffused surface water, to the extent that the landowner has rights to capture, use, or sell it within the limits of applicable water laws. Whereas the layman perceives ownership of property as something that can essentially be bought and sold freely, the legal concept of property and ownership pertains to rights that are owned by the rightholder. A legal right is the assurance that an activity is protected by the legal system, entitling the rightholder to enforceable duties with other people. A set of legal rights, that a rightholder holds exclusively, is called property. Hence, in the legal sense, ownership of property is the owning of legal rights. For most property, the basic rights of ownership include the right to possess it, use it, dispose of it, and exclude everyone else from interfering with it for an indefinite period of time. Drainage laws and other as-

pects of water law in general provide for the allocation of certain rights; this may not extend to the basic rights associated with most other property.

A water right is property, although the rightholder may not have an exclusive right. Water rights, such as those associated with lakes and rivers, may be required to be shared with others depending on the water law. Some of these rights are common property owned by each state as a trustee for its citizens, and each citizen has a right to access it.

### Fundamental Water Doctrines for Diffused Surface Water

From a legal perspective, diffused surface waters are the uncollected flow from precipitation, melting snow, or spring waters that spread over the surface of the earth. This water appears as puddles, sheet, and overland flow and is considered surface water until it reaches well-defined water courses or standing bodies of water. The exact definition of diffused surface waters varies from state to state. There is some disagreement among states as to what extent marshes, swamps, and wetlands are surface waters.

Drainage law prescribes the rights of a landowner to repel surface water at the boundaries of his land and to expel the surface water once it has entered his land. The disposal of surface water becomes necessary when a landowner develops or makes improvements to the land that increases, diverts, or impedes natural flows. Repelling and expelling surface waters often becomes necessary since downstream property is affected by the actions of upstream landowners.

Such situations frequently occur in urban settings of land development projects. As a developer, you take on a legal liability to mitigate the impacts of runoff changes created by your project on the downstream landowner, especially since the downstream landowner has the recourse to hold the developer liable. This legal liability of the developer is the premise for many of the local stormwater management, adequate outfall, and erosion and sediment control guidelines required for permit and local approval of a project. Three basic rules, fundamental to most state laws, regarding drainage are: (1) common enemy rule (often mistakenly referred to as common-law rule); (2) natural flow (or civil law) rule; (3) reasonable use rule.

**Common Enemy Rule.** The common enemy rule regards diffused surface waters as an enemy common to all property owners. This rule states that the landowner is allowed to take whatever measures are necessary to repel or expel diffused surface water effectively regardless of the damage to his neighbors' lands. This rule is circular in its definition and as such creates a contradiction. Suppose the upper landowner uses some effective measures to expel waters onto the lower landowner in order to protect his land. The lower landowner, in order to protect his land, then erects a barrier or takes other effective measures. These measures may result in flooding the upper landowner. Under strict interpretation of the common enemy rule, both landowners have the right to perform (without liability) as described, and the conflict is not resolved.

**The Civil Law Rule.** Under the civil law rule, contrary to the common enemy rule, each piece of land has an easement for the natural drainage to flow across, and the downstream landowners must take the natural flows from the upper land. A landowner may not obstruct, divert, or collect the natural flow of diffused surface water such that it flows from his land in unusual quantities to the detriment of the downstream landowner. The downstream landowner has no obligation to accept surface water flow that has been increased "unnaturally" by the upper landowner. The key word of this rule is "natural," and each state adhering to this doctrine has its own interpretation. The strict interpretation of this rule severely limits what a landowner can do on his land without disrupting natural flows to downstream landowners. Some states interpret this doctrine such that no diversion is deemed to exist if surface waters are collected and discharged for a reasonable purpose and price into a stream that is their natural watercourse. Natural streams generally constitute the legally recognized channel for such purposes. Some states accept this concept regardless of any inadequacy of the existing natural stream. Hence, the liability rests with each downstream landowner to pass the "reasonably increased" flows through his property.

**Reasonable Use Rule.** The civil law rule and the common enemy rule constitute the two extremes of thought for surface water drainage law. Many courts have recognized this and consequently have modified these principles to include

a reasonable use rule. Under the reasonable use pretext, diffused surface waters may be obstructed and diverted if done without malice or negligence and the obstruction or diversion is incidental to ordinary use, improvement or protection of the land. One court stated the rule as follows:

In effecting a reasonable use of land for a legitimate purpose a landowner, acting in good faith, may drain his land of surface waters and cast them as a burden upon the land of another, although such drainage carries with it some waters which otherwise would never have gone that way but would have remained on the land until they were absorbed by the soil or evaporated in the air, if (a) there is a reasonable necessity for such a drainage; (b) reasonable care be taken to avoid unnecessary injury to the land receiving the burden; and (c) if the utility or benefit accruing to the land drained reasonably outweighs the gravity of the harm resulting to the land owner receiving the burden; and (d), if where practicable it is accomplished by reasonably improving and aiding the normal and natural system of drainage according to its reasonable carrying capacity, or if, in the absence of a practicable natural drain, a reasonable and feasible artificial drainage system is adopted.<sup>1</sup>

Another court has used identical ruling of part (c) in applying the reasonable use rule.<sup>2</sup> The interpretation of reasonableness is determined by circumstances of each case. In general, the courts look unfavorably upon those actions that cause damage if those damages could have reasonably been avoided.

The legal principles that govern the right to use the water flowing in a watercourse on one's land are different than those laws that govern the liability for repelling or expelling water from the land or even the liability for changing a watercourse through improvement or obstructions. The extent to which an owner is entitled to use the water then falls under either the riparian doctrine or the appropriation doctrine. Discussion of these doctrines and their application is beyond the scope of this book.

## RAINFALL AND RUNOFF

Every rainfall event is unique. Temporal and spatial precipitation varies seasonally as well as within a storm event due to the prevailing climatic conditions at the time of the storm. Just as every rainfall event is unique, the resulting runoff from a storm event is also unique. The temporal and spatial distribution of the precipitation affect the temporal and spatial distribution of runoff. Additionally, surface conditions such as the amount of vegetation, land use, type of soil, condition of the soil, topography, and other factors affect the volume and distribution of the runoff. When designing the individual smaller components of a storm drain system, the effects of the temporal and spatial distribution

<sup>1</sup> *Anderson v. Kelehan*, 226 Minn. 163, 32 N.W.2d 286 (1948).

<sup>2</sup> *Stouder v. Dashmer* (1951) 242 Iowa 1340, 49 N.W.2d 859; *Bush v. Rochester* (1934) 191 Minn. 591, 255 N.W. 256 (Chow p. 27-30).

of runoff have little impact. However, these effects must be considered when designing larger components, such as detention ponds and major culverts, or when determining the floodplain.

Hydrologic data are historical by nature. Unlike conventional experiments where data are collected through repetition, hydrologic data are collected through observation of an event (e.g., a measured amount of rainfall for a storm, or the flood water depth). The variables relating to hydrologic data, such as time and space variation of rainfall, abstractions, surface conditions, and numerous others that affect runoff are considered continuous variables—that is, quantitatively they can assume any real value. Because the combinations of values of all such variables are infinite, an exact repeat occurrence of an event, although not impossible, is very unlikely.

The infinite number of possible rainfall and runoff events presents an improbable task of ever obtaining all of the unique data potentially available in the hopes of predicting hydrologic events precisely and accurately. Therefore, statistical generalizations are used to represent design storm events. Statistics is a branch of mathematics that studies the collection and organization of data by numerical analysis. Oftentimes inferences are made statistically about larger data sets using the results of related smaller data sets. That part of mathematics used to predict the likelihood of occurrence of a random event is probability. Statistics and probability are often used in mathematical models to perform hydrologic analysis. The following brief explanation and example on frequency analysis should provide insight for the recurrence interval concept discussed later.

### Statistics and Frequency Analysis

For statistical analysis, hydrologic data is arranged according to the number of observations of a specific value or the number of observations within an interval range. A smoothed curve through these data points is a probability distribution function, representing the chance occurrence of a random variable. The most familiar example of a probability distribution function is the Gaussian distribution function, more commonly known as the normal distribution. A simplified discussion illustrates the concepts of such a distribution function and its application.

In statistics, the whole collection of objects or the entire realm of possible outcomes is termed the population or universe. In most analyses, this collection is infinite or so large that it cannot be realistically attained; subsequently, only a portion of the collection is analyzed. The analyzed portion is referred to as the sample space or sample population. The specific quantity that is to be analyzed is termed the variable, and a single measurement or observation is the variate. For example, stream flow, a random variable, can assume any non-negative values. A variate of

25.2 cfs is only one of an infinite number of values that could conceivably occur.<sup>3</sup>

Random variables are either discrete or continuous. Discrete variables are restricted to specific incremental values, and continuous variables can assume any value within the domain of the population. For a given sample population, a plot of the number of occurrences and the variate shows how the number of occurrences are distributed over the range of the variate. This is known as the frequency distribution diagram. The relative frequency is the number of occurrences,  $n$ , of a discrete variate divided by the total number of occurrences,  $N$ . This is also the probability of occurrence for that discrete variate.

**EXAMPLE 1:** Consider the list shown in Table 21.1 of mean annual stream flows for 30 years. For this discrete distribution (i.e., the number of years is finite), the discharges are grouped into an arbitrarily selected incremental range of 100 cubic feet per second (cfs), and the frequency of the groups and cumulative frequency of the groups is counted. The results are shown in Table 21.2.

The frequency distribution diagram is provided in Figure 21.2. As a point of illustration, the relative frequency for a discharge occurring between 200 and 300 cfs is  $4/30$ , or there is a 13% (i.e.,  $n/N$ ) chance for this discharge to occur based on these 30 years.

For many hydrologic events, the interest is in the chance occurrence of an event being equaled or exceeded. That is, what is the probability that stream flow will equal or exceed 200 cfs? This type of question is answered using a cumulative relative frequency diagram, as shown in Figure 21.3.

Note that the cumulated total of the relative frequencies must equal 1. In cumulating the relative probabilities, the probability for a discharge occurring less than a specific discharge is read directly on the ordinate of the cumulative distribution diagram. For this sample population, there is a 23% probability that the average annual stream flow for any given year will be less than 300 cfs. This is equivalent to the area of the histogram of the frequency distribution diagram between 0 and 300 cfs. Hence, the chance for the average annual stream flow to equal or exceed 300 cfs is 77% ( $= 100\% - 23\%$ ). As another example, the chance that the average annual discharge is between 200 and 300 cfs is the difference in the cumulative relative frequencies between the two variates (i.e., the area under the probability distribution diagram between the two values). Hence, there is a

<sup>3</sup> Even though a particular stream may realistically have an upper limit of the discharge, there exists an infinite set of values between 0 and the upper limit because it is a *continuous* variable.

**TABLE 21.1 Mean Annual Stream Discharge for 30-Year Record**

Yr	DISCHARGE (cfs)	Yr	DISCHARGE (cfs)	Yr	DISCHARGE (cfs)	Yr	DISCHARGE (cfs)
1	425	9	452	17	381	25	398
2	346	10	320	18	439	26	128
3	384	11	366	19	741	27	209
4	241	12	339	20	481	28	260
5	328	13	477	21	588	29	390
6	491	14	283	22	392	30	521
7	581	15	161	23	550		
8	398	16	72	24	608		

$(0.23 - 0.10) = 0.13$  probability that the flow is between 200 and 300 cfs.

The sample population for the preceding example is a relatively small data set. Nonetheless, it shows that the probability  $p(x)$  of a variate  $x$  is considered as the probability of a discrete value ( $x = 200$  cfs) falling in the range of  $x$  to  $x + \Delta$  (200 to 300 cfs if  $\Delta = 100$  cfs). As the data increases, the incremental range for grouping discharges decreases. Consequently, the number of groups increases and the graphs become smooth continuous curves. Therefore, in the limit as  $\Delta x$  becomes  $dx$ , the discrete probability distribution histogram becomes the probability density. The

mathematical equation describing the distribution curve is called the probability distribution function, shown as the smooth dashed-line curve in Figure 21.2.

The probability of an event then becomes an integral function with the upper and lower variate values as the upper and lower limits of the integral. Mathematically,

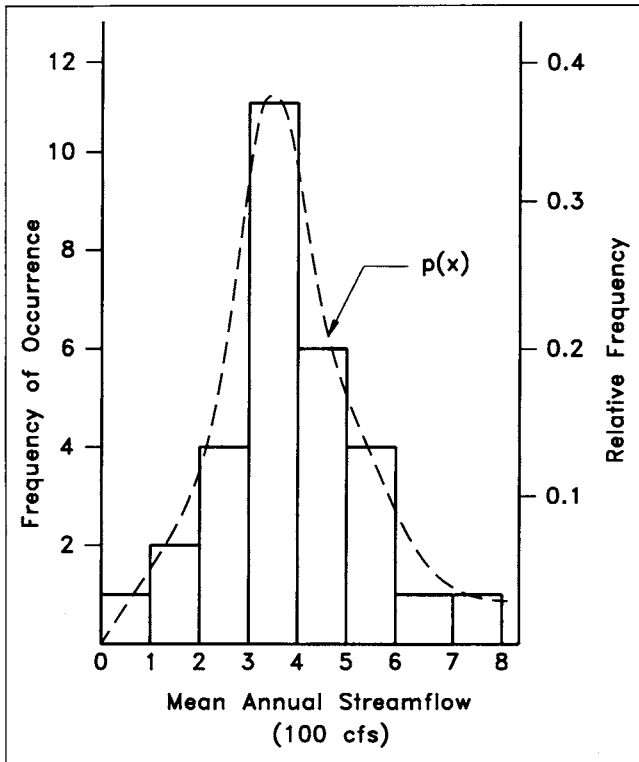
$$p(a \leq x \leq b) = \int_a^b p(x) dx \quad (21.1)$$

In other words, the probability for some value of the variate  $x$ , between  $a$  and  $b$ , is equal to the area under the probability density function between  $a$  and  $b$ .

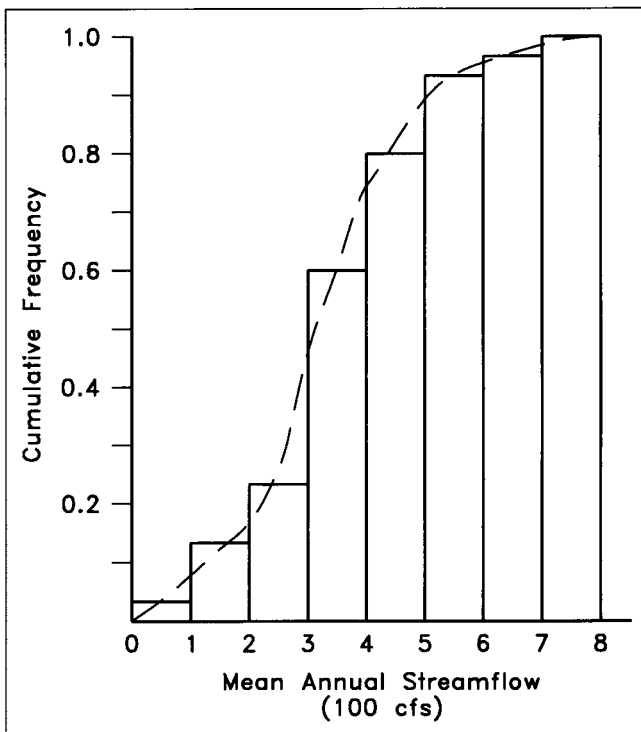
**TABLE 21.2 Frequency for 100-cfs Increments**

DISCHARGE INTERVAL	NUMBER OF OCCURRENCES	RELATIVE FREQUENCY ( $n/N$ )	CUMULATIVE RELATIVE FREQUENCY $\Sigma(n/N)$
$0 \leq q < 100$	1	0.033	0.033
$100 \leq q < 200$	2	0.067	0.100
$200 \leq q < 300$	4	0.133	0.233
$300 \leq q < 400$	11	0.367	0.600
$400 \leq q < 500$	6	0.200	0.800
$500 \leq q < 600$	4	0.133	0.933
$600 \leq q < 700$	1	0.033	0.966
$700 \leq q < 800$	1	0.033	1.00





**FIGURE 21.2** Frequency distribution histogram superimposed with continuous frequency distribution function,  $p(x)$ .



**FIGURE 21.3** Cumulative frequency histogram superimposed with continuous cumulative distribution function.

The normal distribution is one of the most frequently used distributions. However, other probability distributions are used for selected phenomena. For example, studies show that certain hydrologic random variables have specific types of probability distributions. These distributions are characterized by a specific equation with several parameters, whose values depend on the specific data set. The type of distribution selected for use depends on how well it fits the data. Certain hydrologic events tend to fit specific probability distributions more so than others.

Each distribution has specific parameters to describe the shape and locate the distribution curve. A data set requires these specific parameters to adequately describe it and for the data to be meaningful. Three parameters used in describing most distributions are the mean, variance, and skew; also known as the first, second, and third moments of the distribution. Mean:

$$\bar{x} = \frac{1}{n} \sum_{i=1}^n x_i \quad (21.2)$$

where  $\bar{x}$  represents the mean of the sample population and is the best estimate of the population mean,  $\mu$  and  $n$  is the number of data points. The mean is a measure of the distance from the origin to the centroid of the distribution. Variance:

$$\sigma^2 = \frac{1}{n} \sum_{i=1}^n (x_i - \mu)^2 \quad (21.3)$$

Since the population mean is not known, the best estimate of the variance for the sample is

$$s^2 = \frac{1}{n-1} \sum_{i=1}^n (x_i - \bar{x})^2 \quad (21.4)$$

The square root of the variance is the standard deviation. The coefficient of variation,  $C_v = s/x$ , and the variance are used to compare relative variability among distributions. The larger the standard deviation and the coefficient of variance, the larger the spread of the distribution, as shown in Figure 21.4. Skew:

$$\alpha = \frac{1}{n} \sum_{i=1}^n (x_i - \mu)^3 \quad (21.5)$$

Likewise the best estimate of the skewness,  $\alpha$ , for a sample population is:

$$\alpha = \frac{n}{(n-1)(n-2)} \sum_{i=1}^n (x_i - \bar{x})^3 \quad (21.6)$$

A skewed distribution is not symmetrical about the mean as shown in Figure 21.5.

The Gaussian distribution is more commonly used in the analysis of random errors rather than hydrologic data. Other distributions better suited to hydrologic events include the lognormal, extreme value, gamma, and log Pearson type III distributions. Please note that the application

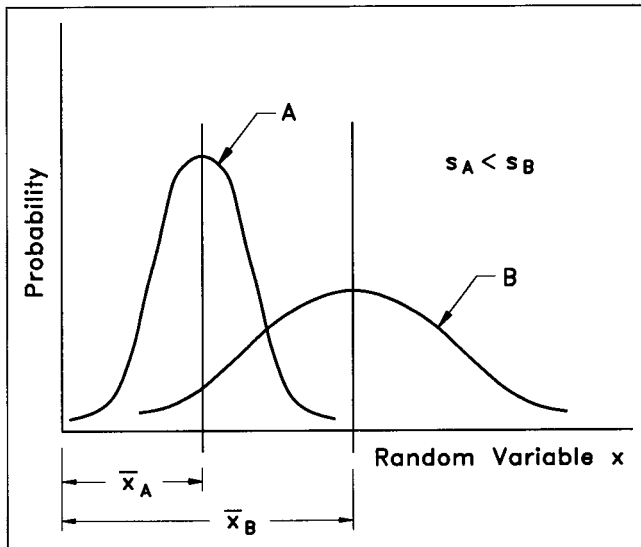


FIGURE 21.4 Standard deviation versus spread of distribution.

of probability theory is a tool used to *approximate* the natural process—it does not represent the phenomena exactly.

### Exceedence Probability and Recurrence Interval

Hydrologic events are predicted by stating their exceedence probability or recurrence interval. The exceedence probability represents the likelihood that an event of specified magnitude will be exceeded in a given time period. The typical time period is one year for most hydrologic events. The return period represents the *average* length of time that will pass between events having the same magnitude. For example, an annual flood discharge of 500 cfs that has an exceedence probability of 0.01 means that, *on the average*, there is a 1% chance that 500 cfs will be *exceeded* in any year. Specifically the recurrence interval is

$$T_r = \frac{1}{P} \times 100 \quad (21.7)$$

where  $T_r$  is the recurrence interval and  $P$  is the probability in percent. Hence, a 1% exceedence probability has a recurrence interval of 100 years.

The concept of exceedence probability and recurrence interval is often misinterpreted. If a 500 cfs has a recurrence interval of 100 years, this does not imply that 500 cfs will occur only once in 100 years. Likewise, if a particular event occurs today, then it will not occur for the next  $T_r$  years is not the proper interpretation of the recurrence interval. The recurrence interval represents the statistical average number of years between similar events given a very long period of record.

Another common misinterpretation is the failure to recognize what is meant by the exceedence probability concept. Given that a 500-cfs discharge has a 1 in 100 chance of being exceeded in any given year does not mean that the exact value of 500 cfs has a 1% probability of occurrence. In fact, the probability of exactly 500 cfs occurring is zero.

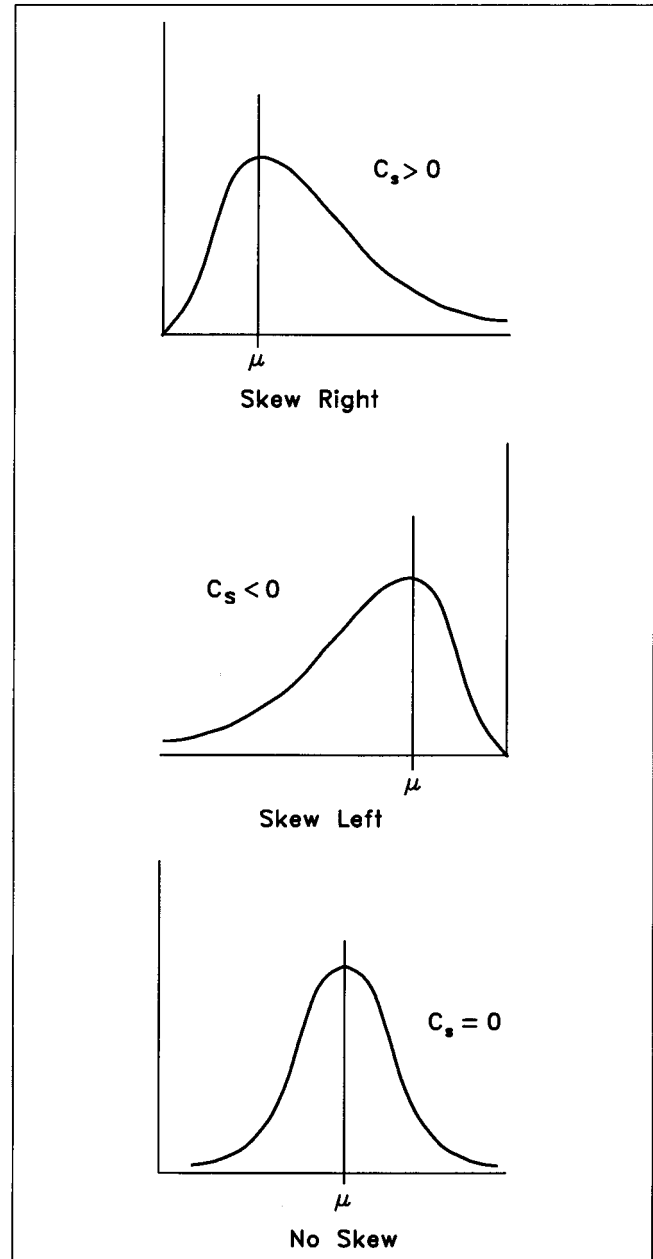


FIGURE 21.5 Skew coefficient types.

For a continuous random variable, like stream discharge, the probability of occurrence is represented by the area under the probability density function, between two distinct values of the variate. There is no area under the curve for just one point.

Occasionally, it is necessary to determine the probability of a specific event being exceeded within a specific time. The probability  $P$  of an event having a given return period  $T_r$  occurring at least once in  $N$  successive years is given as

$$P = 1 - \left(1 - \frac{1}{T_r}\right)^N \quad (21.8)$$

**EXAMPLE 2:** What is the probability that a 100-year storm will occur at least once in the next

25 years? Using Eq. (21.8) with  $T_r = 100$  and  $N = 25$ , the probability is 0.222 (i.e., there is a 22% chance that a 100-year storm will occur at least once in the next 25 years. In Eq. (21.8),  $P$  is known as risk.

A distinction exists between the probability of an event occurring at least once and exactly once in a given time period. Another form of the risk equation determines the probability of an event occurring a precise number of times in a given period. In this equation,

$$P = \frac{(N!) \left(\frac{1}{T_r}\right)^I \left(1 - \frac{1}{T_r}\right)^{N-I}}{I! (N-I)!} \quad (21.9)$$

Here,  $I$  is the exact number of times the event with  $T_r$  occurs in  $N$  successive years.

**EXAMPLE 3:** What is the probability (risk) that the 100-year storm will occur exactly two times in a 25-year period? Substituting  $N = 25$ ,  $T_r = 100$  and  $I = 2$  in Eq. (21.9) gives 0.024, or 2.4%.

### Design Storms

A design storm is the defined result of a statistically estimated rainfall–runoff event used in the design of hydraulic systems. Depending on the hydrologic technique selected, the design storm can be inferred from point precipitation depths (rainfall data), fabricated (synthetic) hydrographs, or isohyetal maps using predetermined spatial storm patterns. It is important to note that the design storm is not an actual storm of record. Rather, it is a fabricated storm compiled from average characteristics of previous storm events, and for convenience and standardization, most review agencies dictate the design storm(s) for use in the design process.

Every storm produces different peak discharges of runoff, has different times to the peak discharge, and consequently different volumes of runoff. Therefore, a specific design storm is characterized by at least two of the following three items:

- *Duration*—the length of time of the storm event
- *Depth*—the total amount of precipitation for the duration of the storm event
- *Frequency*—the average time between two events of similar duration and depth

Additional criteria, used in the hydrologic design process, derived from the foregoing are:

- *Intensity*—depth divided by duration
- *Volume of precipitation*—depth multiplied by areal coverage of the storm

It is important to recognize that the precipitation volume is not necessarily equal to the runoff volume. Runoff is the

amount of excess precipitation, that is, the amount of rainfall after all abstractions, including infiltration, evapotranspiration, and depression storage, have been accounted for.

A distinction should be recognized between intensity and depth–duration relationships. The same depth of rainfall can be produced by different combinations of intensities and durations. Conversely, the same intensity produces different depths of rainfall for various durations. The important concept in design is to specify two of the three parameters (intensity, duration, and frequency) for the design storm to be meaningful.

Table 21.3 provides general guidelines for recurrence interval storms for selected hydraulic systems typical of many local, state, and federal requirements. The duration is specified by the local public agencies.

### Intensity-Duration-Frequency Curves

The hydrologic procedure selected to establish the rainfall–runoff relationship determines what type of data is required to generate the design storm. Simple types of computational procedures, such as the rational method, require basic intensity-duration-frequency (IDF) curves, whereas more sophisticated hydrologic approaches require hyetographs (time variation of precipitation) or hydrographs (time variation of runoff) as input. Data specific to the particular model selected is available from various public agencies.

In 1961, the U.S. Weather Bureau published the “Rainfall Frequency Atlas of the United States,” commonly known as *Technical Paper No. 40* (TP-40). (At the time of this publication, TP-40 had not been updated. However, there is some discussion that an updated version will be published. Additional information regarding TP-40 updates can be obtained from the National Weather Service (NWS).) This document contains rainfall depth maps of the United States for the 1-, 2-, 5-, 10-, 25-, and 100-year recurrence interval storms for durations of 1, 2, 3, 6, 12, and 24 hours for areas east of the 105° meridian. For storm durations of less than 1 hour, the TP-40 information has been superseded by National Oceanic and Atmospheric Administration *Technical Memorandum NWS HYDRO-35*. Precipitation data west of the 105° meridian is available through NOAA on a state-by-state basis. Examples of isopluvial maps in these documents are shown in Figures 21.6 through 21.11. (Isohyets depict spatial variation of rainfall—lines that connect points on a map of equal rainfall depth. Isopluvials are isohyets shown on regional rainfall maps.) Table 21.4 lists the precipitation atlas series, comparable to TP-40, for areas west of the 105° meridian. Other selected depth-frequency references are listed in Table 21.5.

Point depths, such as those on TP-40, apply to areas less than about 10 square miles. Reduction in point depths are required for large catchments to account for variations of storm depths within catchment areas. A depth-area reduction chart is used to determine the percent reduction to be applied to point depths for large catchment areas. However, since most catchments in land development projects are less

**TABLE 21.3 Guidelines for Design Storms for Various Hydraulic Systems**

HYDRAULIC SYSTEM	DESIGN RECURRENCE INTERVAL
Minor Storm Drain System	2- to 25-year
Major Storm Drain System	10- to 50-year
Road Culverts Crossing Minor Streams	10- to 50-year
Road Culverts Crossing Major Streams	25- to 100-year
Small On-Site Detention/Retention Ponds	2, 10, 25, 100-year
Large On-Site or Regional Pond	100-year to PMF*
Floodplains on Minor Streams	10-year to 100-year
Floodplains on Major Streams	100-year +

\* Probable maximum flood.

than 10 square miles, this is of little consequence to the design engineer of most land development projects.

Intensity-duration-frequency (IDF) curves present hydrologic data in another format for use as design storm information. These curves show precipitation intensity on the ordinate, duration along the abscissa, and a series of curves representing individual storm frequencies. The IDF curves are developed through statistical analysis of long-time series rainfall data. They graphically represent the probability that a certain average rainfall intensity will occur given a duration. This is quite different from the misconception that they represent an actual duration or actual time histories of rainfall. A single IDF curve represents data from several different storms. The IDF is fabricated from extracting rainfall depths from selected time segments of longer storms. Procedures for constructing IDF curves are discussed in McPherson (1978). These curves are mainly used in conjunction with the rational method for determining peak runoff. See Figure 21.12 for a typical IDF curve.

Intensity-duration-frequency curves are available through many local agencies such as the state highway departments and the Soil Conservation Service (SCS). In rare cases where IDF curves cannot be obtained, they can be developed from U.S. Weather Service Maps of TP-40 and HYDRO-35 or from frequency analysis using local rainfall information. Discussions on developing IDF curves through frequency analysis are provided in Chow and in American Geophysical Union (AGU) in the references. Besides TP-40 and HYDRO-35, the NWS provides other documents relating the depth-duration-frequency of storms as listed in Table 21.5.

### RATIONAL METHOD FOR PEAK DISCHARGE

For small urban drainage areas, common in minor storm drainage design, it is assumed that short-duration, high-intensity storms are the cause of flooding. For such short-

duration storms and small drainage areas, the rainfall intensity is often assumed constant and the peak runoff rate occurs when the entire drainage area is contributing to the runoff—that is, when the drainage area is in steady-state equilibrium. If a storm of constant intensity begins instantaneously, the rate of runoff for the catchment steadily increases until the entire drainage area is contributing to the discharge at the outlet point. From then on, the drainage area is in equilibrium. All precipitation is converted to runoff and the peak runoff remains uniform for the duration of the constant intensity rainfall.

Peak runoff from the rational method is given by,

$$Q_p = CiA \quad (21.10)$$

where  $Q_p$  is the peak discharge in cfs,  $A$  is the drainage area in acres,  $C$  is a runoff coefficient characteristic of the ground surface ( $0 < C < 1$ ), and  $i$  is the averaged rainfall intensity in inches per hour (in./hr). The precision of the peak discharge depends on the estimated values of  $C$  and  $i$ . The averaged rainfall intensity is a function of the time of concentration of the drainage area.

The rational method is best utilized for small drainage areas. Most localities have restrictions on the applicability of the rational method, ranging from 20 to 200 acres. Common practice limits the use of the rational method to areas less than 100 acres. Other jurisdictions also place time-of-concentration restrictions on the use of the rational method, limiting the maximum time of concentration to 60 minutes.

### Runoff Coefficient

In Eq. (21.10), the product  $iA$  can be considered as the inflow to the catchment while also representing the maximum possible runoff rate. The ratio of peak discharge,  $Q_p$ , to inflow,  $iA$ , is the runoff coefficient,  $C$ . This coefficient can be considered as a lump-sum parameter that accounts for

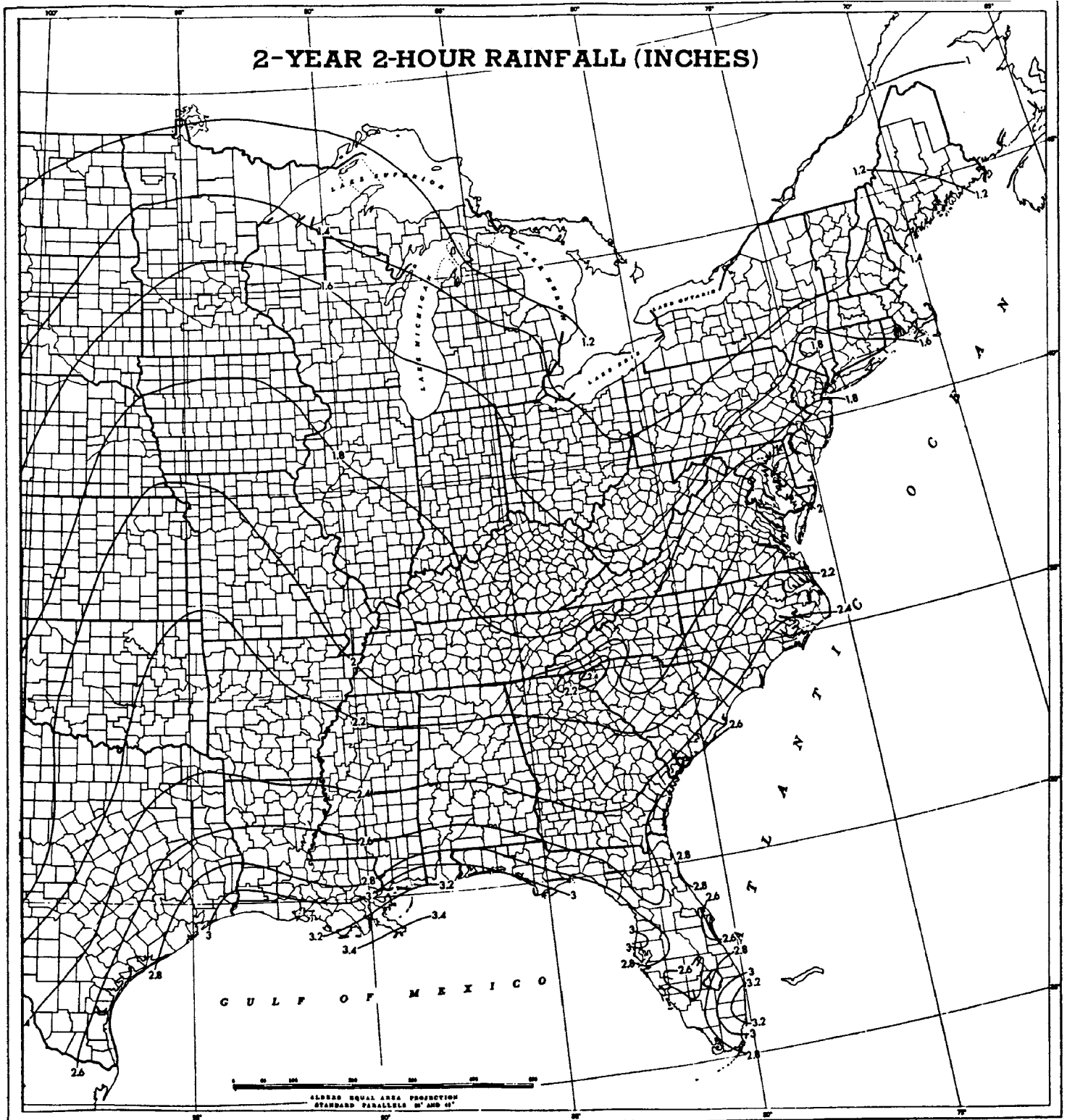


FIGURE 21.6 Two-year 2-hour isopluvial map.

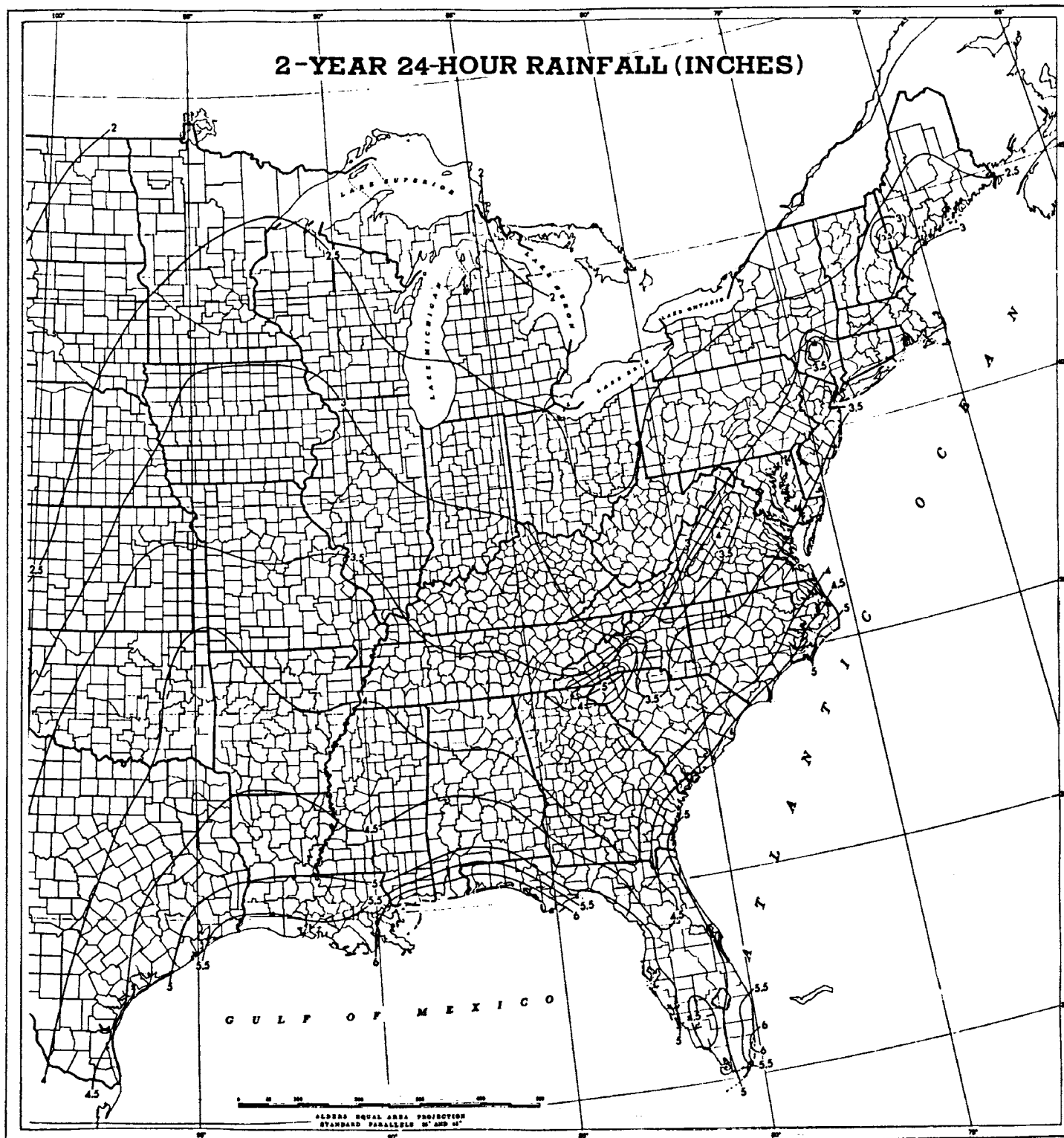


FIGURE 21.7 Two-year 24-hour isopluvial map.

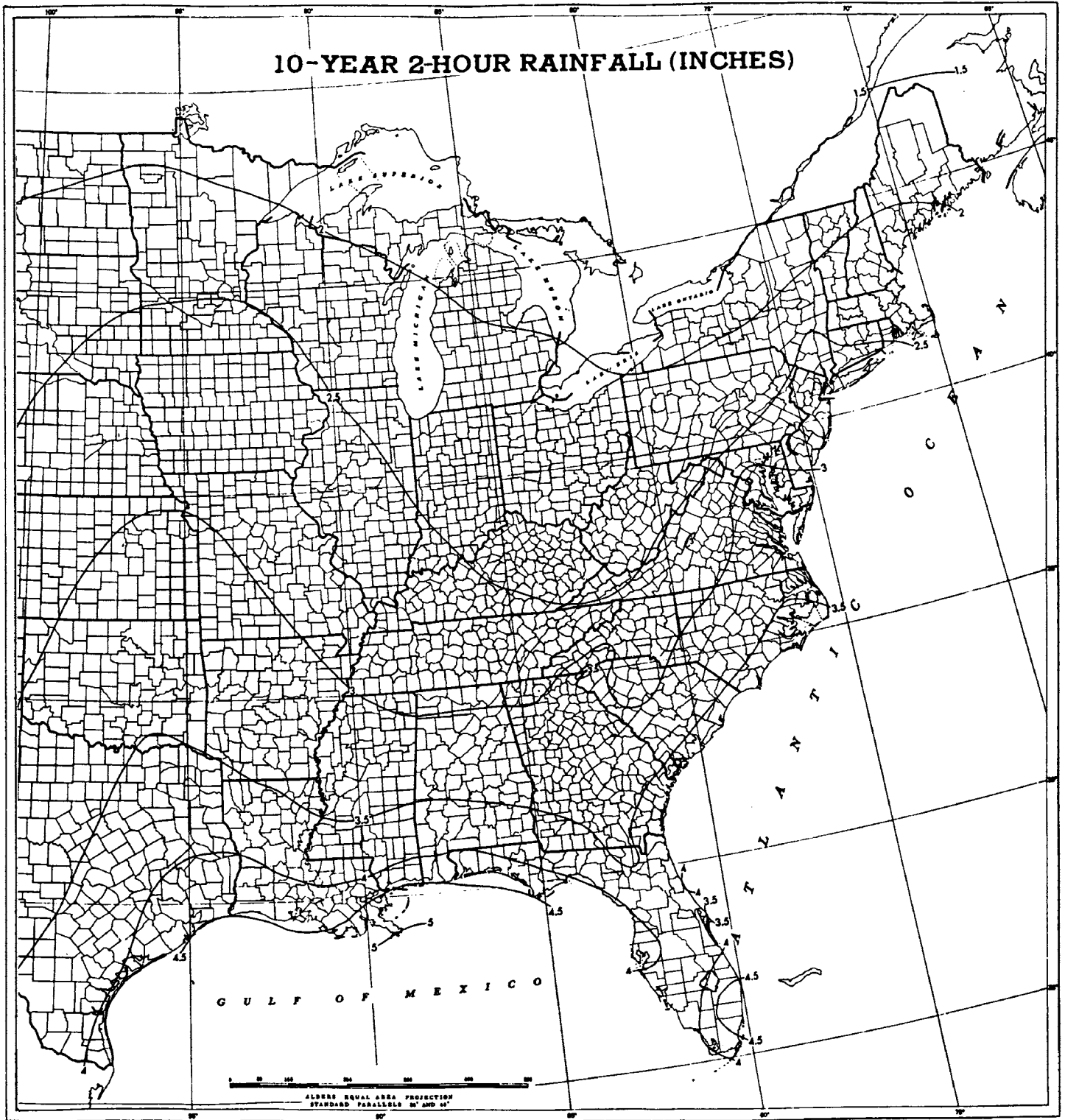


FIGURE 21.8 Ten-year 2-hour isopluvial map.

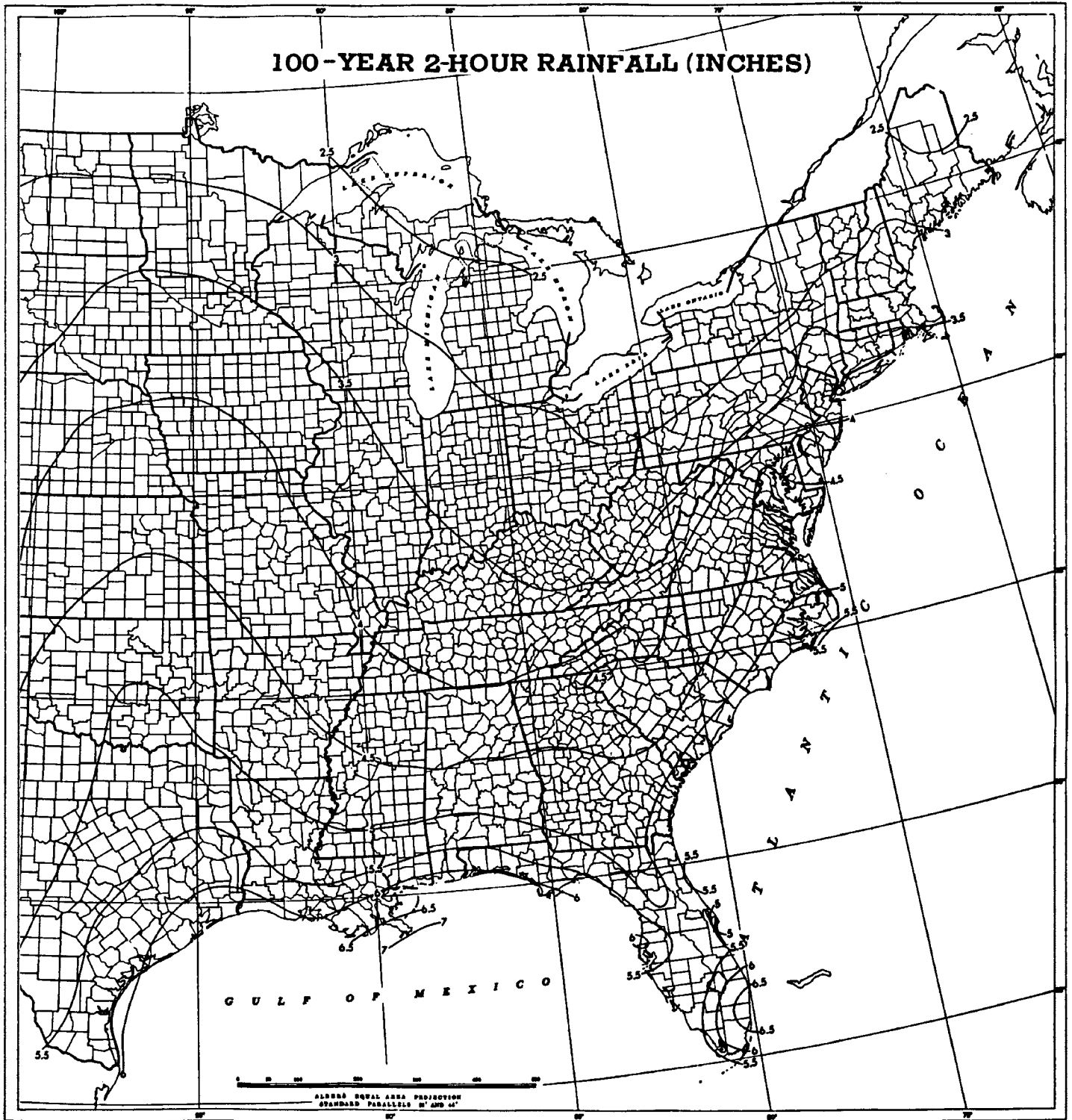


FIGURE 21.9 One-hundred-year 2-hour isopleth map.



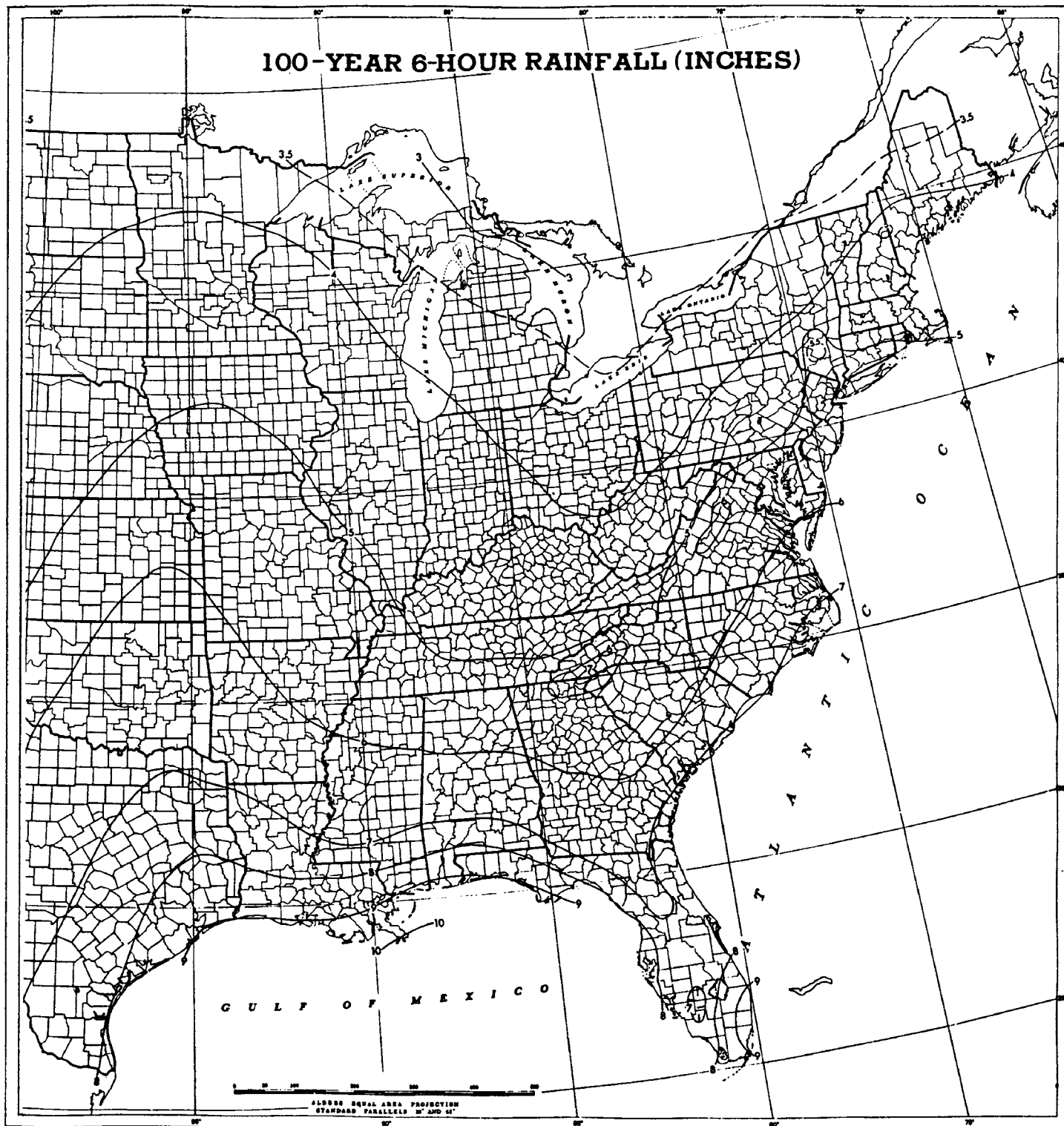


FIGURE 21.10 One-hundred-year 6-hour isopluvial map.

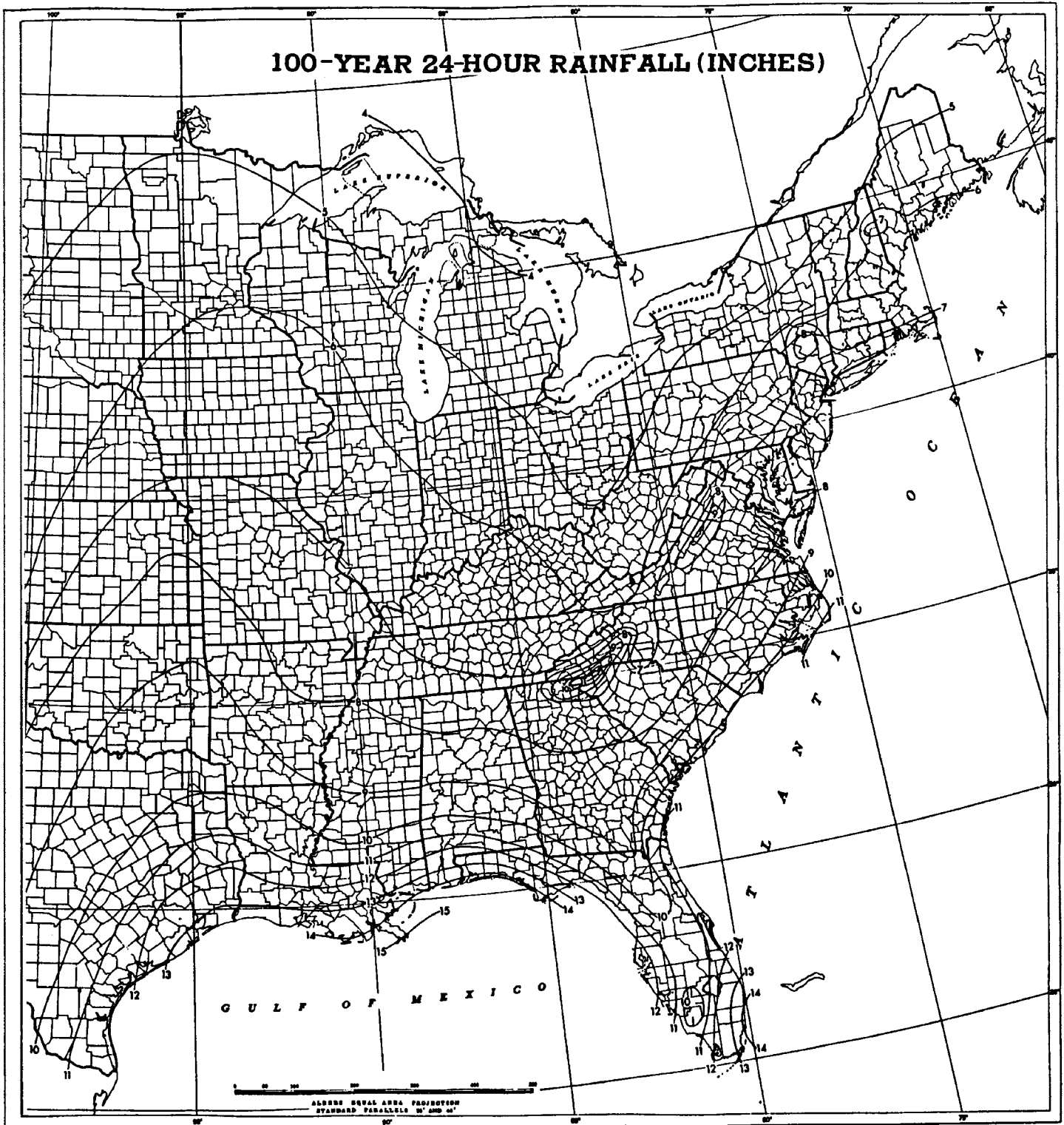


FIGURE 21.11 One-hundred-year 24-hour isopluvial map.

**TABLE 21.4 NOAA Atlas 2: Precipitation Atlas of the Western United States**

Vol. I, Montana	Vol. V, Idaho	Vol. IX, Washington
Vol. II, Wyoming	Vol. VI, Utah	Vol. X, Oregon
Vol. III, Colorado	Vol. VII, Nevada	Vol. XI, California
Vol. IV, New Mexico	Vol. VIII, Arizona	
Technical Paper 42, Puerto Rico and Virgin Islands, 1961		
Technical Paper 43, Hawaii, 1962		
Technical Paper 47, Alaska, 1963		

abstractions, antecedent moisture conditions and other variables affecting the runoff rate. Table 21.6 identifies the ASCE's (American Society of Civil Engineers) version of the runoff coefficient, and the standards used in Austin, Texas, are shown as an example in Table 21.7.

Note that the coefficient,  $C$ , is also a function of the recurrence interval of the storm. The reason for this function is an attempt to approximate soil saturation conditions. For larger storm events, it is agreed that the soil has already been saturated to such an extent that it no longer has the infiltration characteristics associated with everyday conditions. Therefore, since the soil is saturated, the rainfall will produce more runoff. The greater the saturation of the soil, the higher the  $C$  coefficient and, hence, the greater the runoff.

Other localities account for the change in  $C$ -coefficient versus recurrence interval by using a correction factor. For example, using the correction factor, Eq. (21.10) would become

$$Q_p = C_f C_i A \quad (21.10a)$$

where the correction factor  $C_f$  varies by recurrence interval. Comparing the City of Austin example in Table 21.7, the  $C$  coefficient would remain the same for all storms; however, the  $C_f$  factor would change for storms greater than the 2-year event.  $C_f$  would equal 1.066, 1.107, 1.178, 1.233, 1.301, and 1.370 for the 5-, 10-, 25-, 50-, 100-, and 500-year events. Check with the local agency to determine whether correction factors exist.

**TABLE 21.5 NWS Depth-Duration-Frequency References**

U.S. Weather Bureau, "Generalized Estimate of Probable Maximum Precipitation and Rainfall Frequency Data for Puerto Rico and Virgin Islands for Areas to 400 Square Miles, Durations to 24 Hours, and Return Periods from 1 to 100 Years," *Technical Paper No. 42*, 1962.

U.S. Weather Bureau, "Rainfall Frequency Atlas of Hawaiian islands for Areas to 200 Square Miles, Durations to 24 Hours, and Return Periods from 1 to 100 Years," *Technical Paper No. 43*, 1962.

U.S. Weather Bureau, "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years," *Technical Paper No. 40*, 1963, applicable to States East of the 105th Meridian.

U.S. Weather Bureau, "Probable Maximum Precipitation and Rainfall Frequency Data for Alaska for Areas to 400 Square Miles, Durations to 24 Hours and Return Periods from 1 to 100 Years," *Technical Paper No. 47*, 1963.

U.S. Weather Bureau, "Two-to-Ten Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States," *Technical Paper No. 49*, 1964, applicable to the contiguous United States.

NOAA National Weather Service, "Atlas 2: Precipitation Atlas of the Western United States," 1973, applicable to the 11 Western States.

NOAA National Weather Service, "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States," *Technical Memorandum NWS HYDRO-35*, 1977.

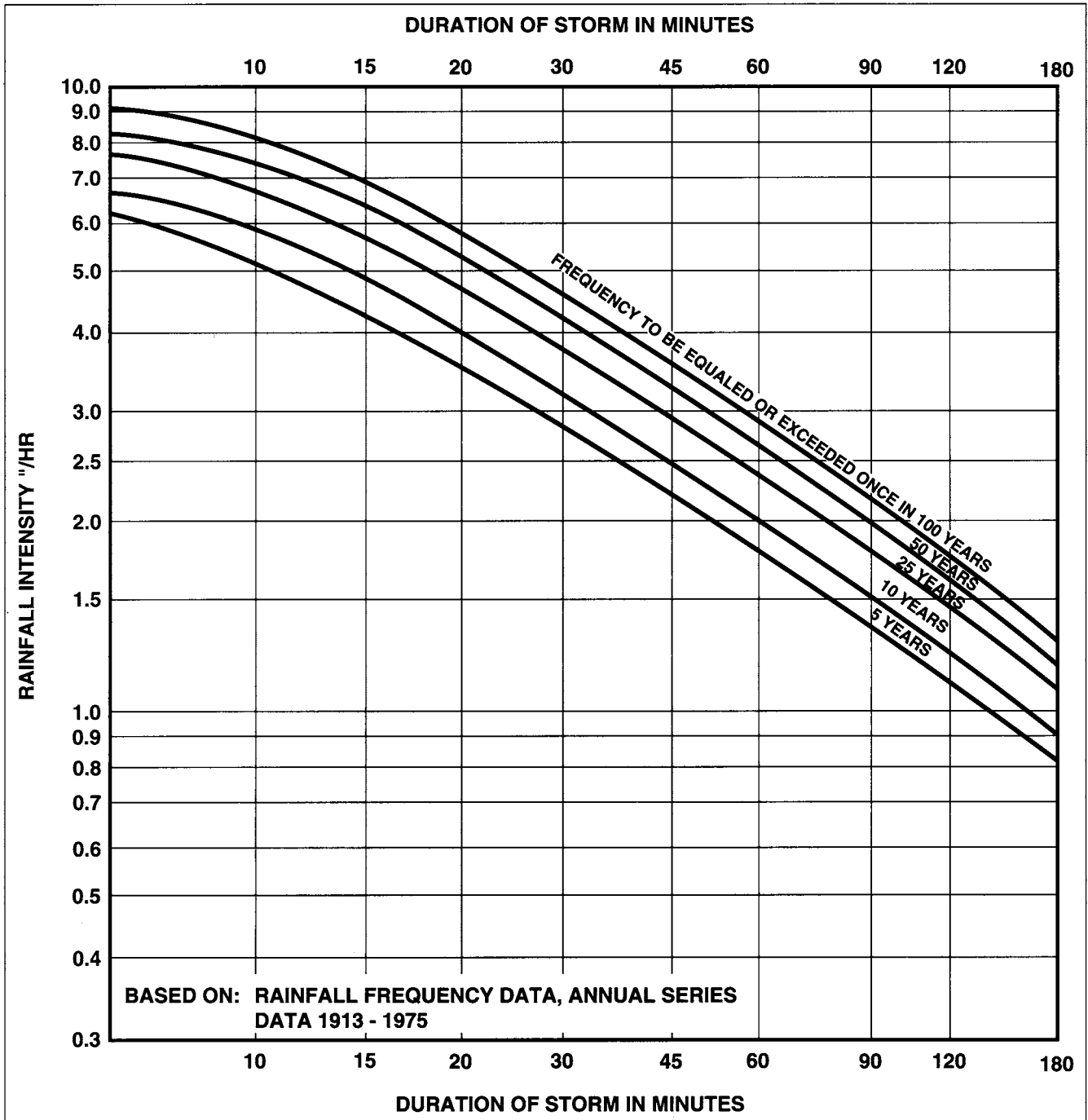


FIGURE 21.12 Intensity-Duration-Frequency Curves.

SCS methodology for hydrologic modeling (TR-55) classifies soil series into four hydrologic groups (A, B, C, D) according to soil infiltration and permeability characteristics. Soils in group A have the highest infiltration rates—that is, the lowest runoff potential—and group D soils have the highest runoff potential. Table 21.8 is useful in that it correlates the  $C$  coefficient to hydrologic soil groups and slope ranges with various types of land use. Whenever a single catchment area consists of several areas with different  $C$  coefficients, a weighted coefficient is computed. The weighted coefficient is found by

$$C_w = \frac{\sum_{i=1}^m C_i A_i}{A_T} \quad (21.11)$$

where  $C_w$  is the weighted  $C$  coefficient,  $A_i$  is the area of the sub-area with  $C_i$  coefficient, and  $A_T$  is the total area of the catchment.

#### Time of Concentration

The time of concentration is the time for water to flow from the most hydraulically remote point of the drainage area to

**TABLE 21.6 Runoff Coefficients  $C$  Recurrence Interval  $\leq 10$  years<sup>a</sup>**

DESCRIPTION OF AREA	RUNOFF COEFFICIENTS	CHARACTER OF SURFACE	RUNOFF COEFFICIENTS
Business		Pavement	
Downtown	0.70–0.95	Asphalt or concrete	0.70–0.95
Neighborhood	0.50–0.70	Brick	0.70–0.85
Residential		Roofs	
Single family	0.30–0.50	Lawns, sandy soil	
Multiunits, detached	0.40–0.60	Flat, 2%	0.05–0.10
Multiunits, attached	0.60–0.75	Average, 2–7%	0.10–0.15
Residential, suburban	0.25–0.40	Steep, 7% or more	0.15–0.20
Apartment	0.50–0.70	Lawns, heavy soil	
Industrial		Flat, 2%	0.13–0.17
Light	0.50–0.80	Average, 2–7%	0.18–0.22
Heavy	0.60–0.90	Steep, 7% or more	0.25–0.35
Parks, cemeteries	0.10–0.25		
Railroad yard	0.20–0.35		
Unimproved	0.10–0.30		

Source: From "Design and Construction of Sanitary and Storm Sewers," *ASCE Manual of Practice No. 37*, revised by D. Earl Jones, Jr., 1970.

<sup>a</sup> For 25- to 100-yr recurrence intervals, multiply coefficient by 1.1 and 1.25, respectively, and the product cannot exceed 1.0.

the outlet point. Recognize that this does not imply the most distant point in terms of length. Rather, it is considered as the longest flow time from some point in the drainage area to the outlet point. For example, the point most distant could be drained by storm sewers, which would quicken the travel time to the outlet point, while an area closer to the outlet point would travel over natural terrain, thus slowing it down. When runoff from the most hydraulically remote point reaches the outlet, the entire catchment area is then contributing to the discharge.

The time of concentration is the sum of two components: (1) the overland flow time (or inlet time) and (2) the channel (or conveyance) time. Overland flow is typically thought of as flowing as a thin layer without any significant depth, before it concentrates in defined swales and channels. This could also be referred to as inlet time, since overland flow is basically confined to a short stretch draining to a catchment, such as a street inlet. Channel time is that part of the flow time when the runoff proceeds as concentrated flow in perhaps irregular but well defined channels. Often, time is estimated using the average velocity for the hydraulic characteristics of the channel. Although overland flow is occasionally referred to as sheet flow—that is, flow over plane surfaces—it should not, in the context of the rational method, be confused with the SCS's hydrologic definition of sheet flow and overland flow. Refer to Chapter 23—Stormwater Management Facilities for discussion of SCS hydrology.

The time of concentration varies according to hydraulic characteristics of the watershed and the storm event itself.

Generally, for use in the rational method, the time of concentration is never taken as less than five minutes—even for the smallest catchment and nearly impervious ground surface. The time of concentration varies from 5 to 10 minutes for paved areas with average slopes in the 2 to 10% range and where the flow path to the inlet is 100 to 500 ft. For grassed areas, the time of concentration may vary between 10 to 30 minutes for flow paths between 100 and 500+ ft.

There are numerous empirical methods to determine the inlet time of concentration. The method selected depends on the information available and the preferences dictated by local review agencies. Some of the various methods are listed in Table 21.9. Two of these methods are subsequently discussed.

One of the better-known methods that relates the overland flow time to slope and length parameters is the Kirpich equation. Initially, the equation was developed for small agricultural watersheds with drainage areas less than 200 acres. Over time, adjustment factors have been applied to the equation for application to pavement surfaces (see Table 21.9). The Kirpich equation is

$$t_c = 0.0078 \left( \frac{L^{0.77}}{S^{0.385}} \right) \quad (21.12)$$

where  $t_c$  is the time of concentration in minutes,  $L$  is the length of the flow path in feet and  $S$  is the average slope of the flow path =  $\Delta\text{Elev}/L$ .

Manning's kinematic solution, in the form shown in Eq. (21.13), can be used to compute sheet flow travel time.

TABLE 21.7 Runoff Coefficients for Use in the Rational Method

CHARACTER OF SURFACE	RETURN PERIOD (YEARS)						
	2	5	10	25	50	100	500
Developed							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass areas (lawns, parks, etc.)							
<i>Poor condition</i> (grass cover less than 50% of the area)							
Flat, 0–2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2–7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
<i>Fair condition</i> (grass cover on 50% to 75% of the area)							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
<i>Good condition</i> (grass cover larger than 75% of the area)							
Flat, 0–2%	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2–7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
Undeveloped							
Cultivated land							
Flat, 0–2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2–7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/range							
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/woodlands							
Flat, 0–2%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2–7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Note: The values in the table are the standards used by the City of Austin, Texas. Used with permission.

$$T_t = \frac{(0.93)(NL)^{1/m}}{(i)^{(m-1/m)}S^{1/2m}} \quad (21.13)$$

where  $T_t$  is the travel time in minutes,  $n$  is Manning's roughness coefficient adjusted for overland flow conditions due to an increase in friction for very shallow flows (see Table 21.10),  $L$  is the flow length in feet,  $i$  is rainfall intensity (in./hr), and  $S$  is the average land slope (ft/ft) of the overland flow path. The exponent  $m$  varies from 1.67 to 3.0 depending on whether the overland flow regime is laminar or turbulent. For fully turbulent flow,  $m$  is taken as 1.67. Use of this equation is limited to very shallow depths (<0.1 ft) and for  $L < 300$  ft. The solution to this equation is a trial and error procedure performed as follows:

1. Assume a value of  $i$ .
2. Use Eq. 21.13 to find  $T_t$ .
3. Find the actual rainfall intensity from IDF chart for storm duration of computed  $T_t$ .
4. Compare the assumed value of  $i$  to the one read from the IDF curve. If they are not close, repeat steps 1 through 4.

Figures 21.13 and 21.14 represent nomographs used to solve the time of concentration equations for the Kirpich method and the Manning's kinematic solution, respectively.

Large catchments will require the consideration of several flow paths in determining which represents the time of

**TABLE 21.8 Runoff Coefficients for the Rational Formula  
by Hydrologic Soil Group and Slope Range**

LAND USE	A			B			C			D		
	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+	0-2%	2-6%	6%+
Cultivated land	0.08 <sup>a</sup>	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14 <sup>b</sup>	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.16	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential Lot Size 1/8 acre	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Lot Size 1/4 acre	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Lot Size 1/3 acre	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Lot Size 1/2 acre	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.46
Lot Size 1 acre	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.24	0.23	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

Source: Kibler, D.F. et al. *Recommended Hydrologic Procedures for Computing Urban Runoff in Pennsylvania*, Commonwealth of Pa., Harrisburg Pa.: Dept. of Environmental Resources, 1982.

<sup>a</sup> Runoff coefficients for storm recurrence intervals less than 25 years

<sup>b</sup> Runoff coefficients for storm recurrence intervals of 25 years or more

TABLE 21.9 Summary of Time of Concentration Formulas

METHOD AND DATE	FORMULA FOR $t_c$ (min)	REMARKS
Kirpich (1940)	$t_c = 0.0078L^{0.77}S^{-0.385}$ $L$ = length of channel/ditch from headwater to outlet (ft) $S$ = average watershed slope (ft/ft)	Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3–10%); for overland flow on concrete or asphalt surfaces multiply $t_c$ by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches.
California Culverts Practice (1942)	$t_c = 60 \left( 11.9 \frac{L^3}{H} \right)^{0.385}$ $L$ = length of longest watercourse (mi) $H$ = elevation difference between divide and outlet (ft)	Essentially the Kirpich formula; developed from small mountainous basins in California (U.S. Bureau of Reclamation, pp. 67–71 1973).
Izzard (1946)	$t_c = \frac{41.025(0.007 i + c)L^{0.33}}{S^{0.33}i^{0.67}}$ $i$ = rainfall intensity (in/hr) $c$ = retardance coefficient $L$ = length of flow path (ft) $S$ = slope of flow path (ft/ft)	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product $i$ times $L$ should be $\leq 500$ .
Federal Aviation Administration (1970)	$t_c = 1.8(1.1 - C) \frac{L^{0.5}}{S^{0.33}}$ $C$ = rational method runoff coefficient $L$ = length of overland flow (ft) $S$ = surface slope (%)	Developed from airfield drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.
Kinematic wave formulas Morgali and Linsley (1965) Aron and Erborge (1973)	$t_c = \frac{0.94 L^{0.6} n^{0.6}}{i^{0.4} S^{0.3}}$ $L$ = length of overland flow (ft) $n$ = Manning roughness coefficient $i$ = rainfall intensity (in/hr) $S$ = average overland slope (ft/ft)	Overland flow equation developed from kinematic wave analysis surface runoff from developed surfaces; method requires iteration since both $i$ (rainfall intensity) and $t_c$ are unknown; superposition of intensity-duration-frequency curve gives direct graphical solution to $t_c$ .
SCS average velocity charts (1975, 1986)	$t_c = \frac{1}{60} \leq \frac{L}{V}$ $L$ = length of flow path (ft) $V$ = average velocity in feet per second from Figure 3-1 of TR 55 for various surfaces	Overland flow charts in Figure 3-1 of TR 55 show average velocity as function of watercourse slope and surface cover (see also Table 5.7.1)

Source: Kibler, David F., ed., *Urban Stormwater Hydrology Monograph 7*, Copyright 1982 by the American Geophysical Union, Washington, D.C.

concentration. The flow path with the longest travel time is typically selected for design. However, there are exceptions to this blanket statement. Since the rational method is best suited for homogeneous drainage areas (i.e., consistent land use and topography) that increase linearly with length, both the shape of the drainage basin and its homogeneity affect the peak discharge at various points within the catchment. For those situations where catchment area and length are not linearly related, or when the catchment has widely varied land use, selecting the flow path with the longest  $t_c$  does not always produce the peak discharge at the specified location. The following are cases to illustrate this.

The example below shows when the  $t_c$  should be carefully considered for non-homogeneous catchments. Con-

sider the situation given in Figure 21.15 and determine the peak discharge at the outlet using the rational method.

The rainfall intensity is given as

$$i = \frac{97.5}{T_d^{0.83} + 6.88} \quad (21.14)$$

where  $T_d$  is the intensity duration (in minutes). The  $t_c$  for the entire watershed is 65 minutes. Assuming  $T_d = t_c$ , the rainfall intensity is 2.51 in./hr. The weighted  $C$  coefficient for the catchment is

$$C_w = \frac{(0.2)(60) + (0.7)(30)}{90} = 0.37 \quad (21.15)$$

The peak discharge per the rational method is 83.6 cfs.



**TABLE 21.10 Effective Roughness,  $N$ , for Overland Flow\***

SURFACE	$N$ VALUE
Dense growth	0.40–0.50
Pasture	0.30–0.40
Lawns	0.20–0.30
Bluegrass sod	0.20–0.50
Short prairie grass	0.10–0.20
Sparse vegetation	0.05–0.13
Bare clay-loam soil (eroded)	0.01–0.03
Concrete-asphalt very shallow depths <6 mm	0.10–0.15

\* Hydrologic Engineering Center, U.S. Army Corps of Engineers, *HEC-1 Flood Hydrograph Package Users Manual*, 1990.

Compare this to the peak discharge only from the developed area of 108.4 cfs, i.e.,

$$Q_p = (30 \text{ ac})(0.7)(5.16 \text{ in./hr}) \quad (21.16)$$

$$= 108.4 \text{ cfs } (i = 5.16 \text{ in./hr for } t_c = 20 \text{ min})$$

The peak discharge for the entire catchment is less than the peak discharge from the developed areas. This shows the necessity for carefully analyzing the situation when the catchment is nonhomogeneous.

Another example, illustrating the need to interpret the  $t_c$ , is the occasion of composite catchments. Consider the situation where the discharge point drains two widely varied catchments, as in Figure 21.16. For this discussion, the rainfall intensity is given by Eq. (21.13). Using the rational method and the longest  $t_c$  (60 minutes), a misleading peak discharge at point P is calculated. The weighted  $C$  coefficient is

$$C_N = \frac{(0.7)(30) + (0.3)(80)}{(30 + 80)} = 0.41 \quad (21.17)$$

The peak discharge is

$$Q_p = (110 \text{ ac})(0.41)(2.65 \text{ in./hr}) = 119.5 \text{ cfs} \quad (21.18)$$

However, the actual peak discharge occurs earlier, when the combined effects of all catchment B and some part of catchment A are contributing. Without sufficient data on catchment A, an assumption must be made on the  $t_c$  to area relationship of the catchment. This would require a trial-and-error approach to incrementally add portions of the drainage area of catchment A, to determine the impacts of

peak discharge to point P. Since the rational method is mathematically linear, the trial-and-error process is simplified, and the user should be able to identify the portion of catchment A that increases discharge to point P with only a few iterations.

The equation for  $Q_p$  for the combined effects is

$$Q_p = i \left[ (CA)_B + \left( \frac{(t_c)_{inc}}{t_c} (CA) \right)_A \right] \quad (21.19)$$

where the  $(t_c)_{inc}$  corresponds to an incremental time of concentration greater than 20 minutes and less than 60 minutes and  $i$  is the corresponding rainfall intensity. The incremental  $t_c$  producing the largest peak discharge is the design discharge. In this case the largest discharge occurs for a rainfall duration of 20 minutes, as shown in Table 21.11.

### Rational Method Summary

The rational method provides adequate results for computing peak discharges as long as it is used properly, with an understanding of the underlying assumptions and limitations. Even with proper understanding of the rational method, as the catchment increases in size, the results become suspect due to the assumption of a steady uniform rain over the entire catchment. Additionally, the inherent uncertainties in the  $C$  coefficient are magnified as the catchment area increases. There is disagreement within the engineering community on the upper limit of the catchment size that can effectively utilize the rational method. Values of 200 acres (ac) to 1 square mile (640 ac) have been proposed. Certainly for the relatively small catchments (<50 ac) encountered in minor storm drain design, the rational method should be satisfactory for use. Application of the rational method to larger catchments requires subdividing the overall catchment into subcatchments and then including the effects of channel routing.

The key element in using the rational method is proper determination of the time of concentration. Due to the hyperbolic shape of the IDF curves, a small error in  $t_c$  (i.e., rainfall duration) causes large discrepancies on the intensity. If the estimated  $t_c$  is less than the actual  $t_c$ , the rainfall intensity will be too high, resulting in a high  $Q_p$ . Another important consideration, when performing the hydrologic analysis, is the dynamics of the land use in the catchment. For projects within a catchment undergoing development, the runoff coefficient should represent the catchment as it might ultimately appear, rather than current conditions.

To summarize, these are the basic assumptions in the rational method:

- Rainfall intensity is uniform and constant over the catchment and the duration of this rainfall intensity is at least as long as the time of concentration of the catchment.

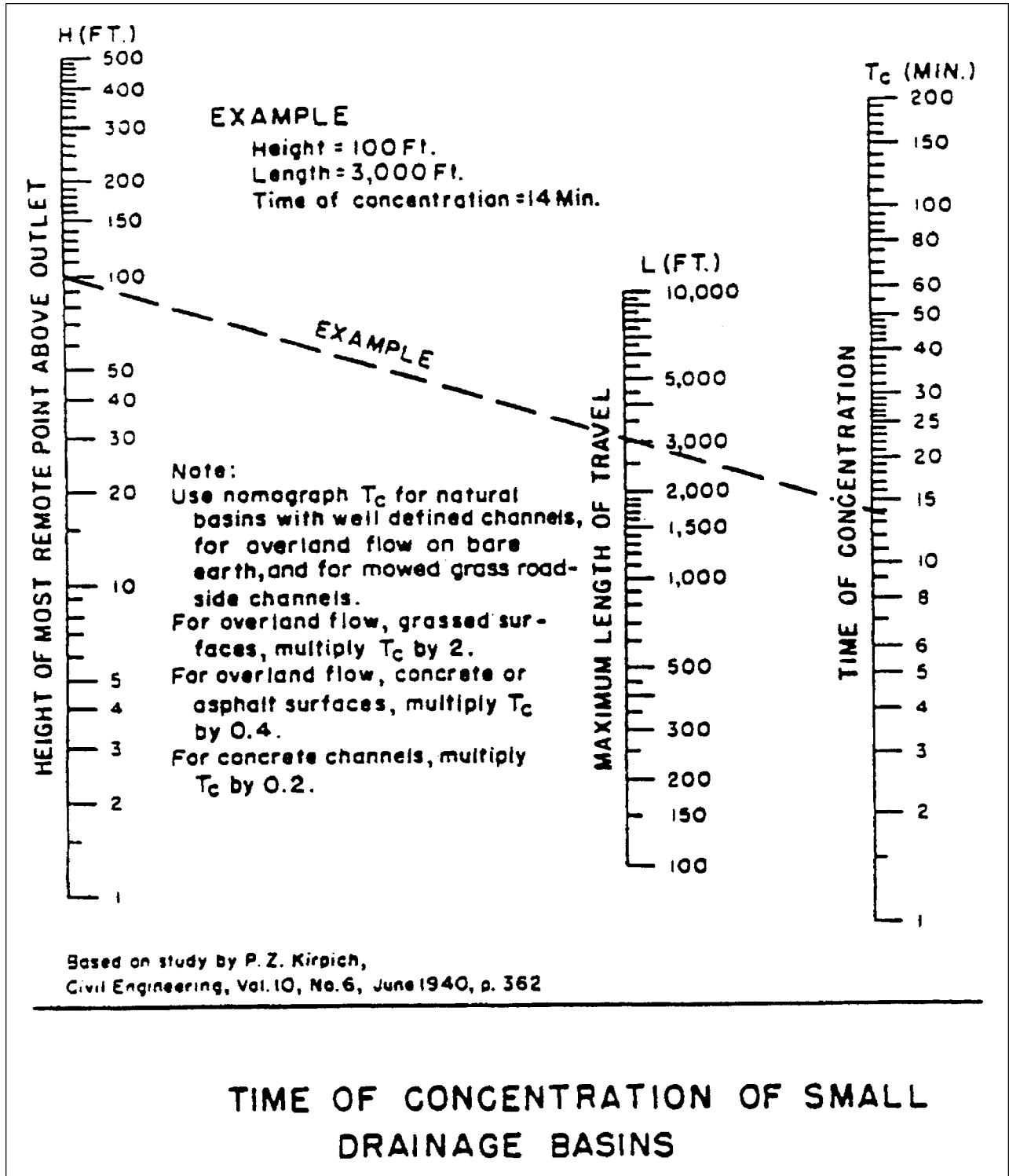
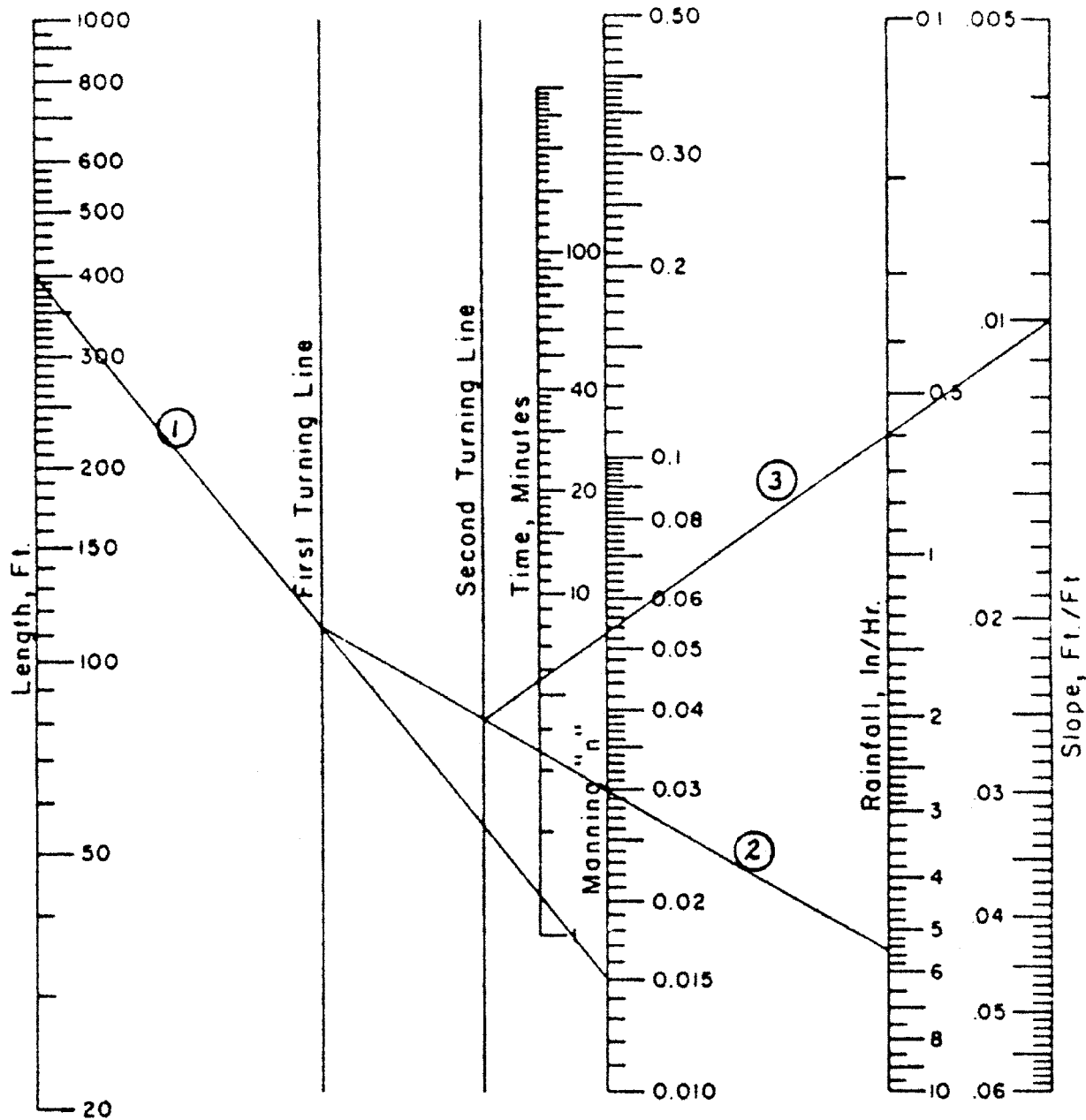


FIGURE 21.13 Time of concentration of small drainage basins (Kirpich method).

Equation solved by nomograph:

$$t_c (\text{min}) = .94 \frac{L^{0.6} n^{0.6}}{i^{0.4} S_0^{0.3}}$$



The initially assumed value of  $i$  and the nomograph value of  $t$  must be checked against the applicable intensity-duration-frequency curve by trial and error.

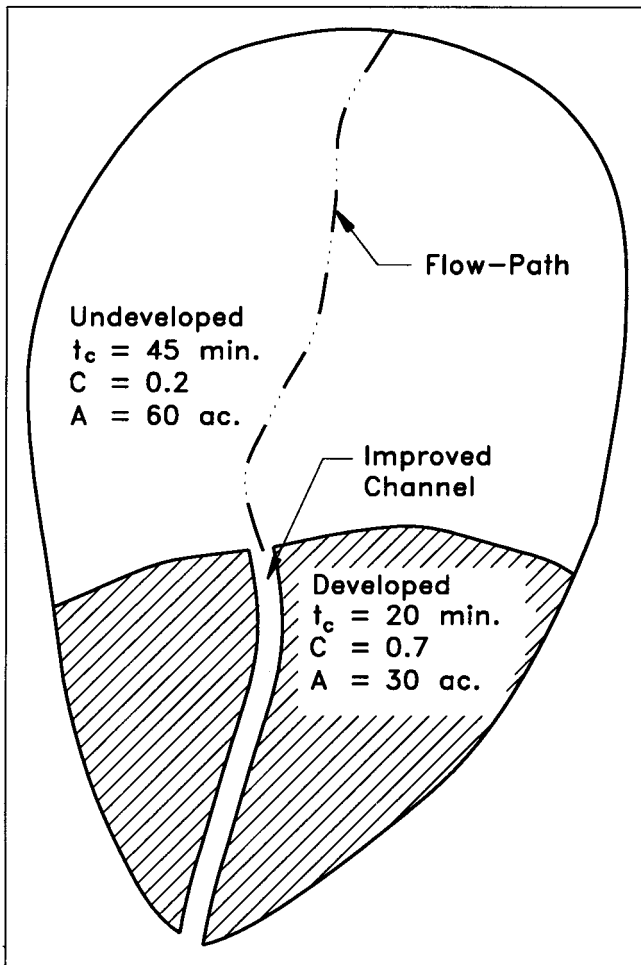
Example:

$L = 400$  ft.  
 $n = 0.015$   
 $i = 5.5$  in./hr.  
 $S_0 = 0.01$   
 $t = 5.5$  min.

ONE INCH is 25.4 mm  
 ONE FOOT is 0.3048 m

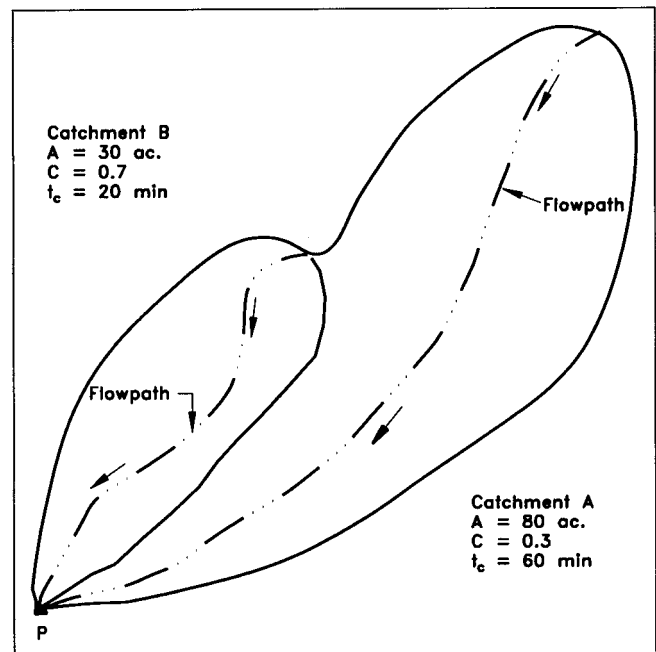
Nomograph for determining time of concentration for overland flow, Kinematic Wave Formulation. (After Ragan.)

FIGURE 21.14 Time of concentration for overland flow, Manning's kinematic wave formulation.



**FIGURE 21.15** Catchment with development at the lower end.

- The maximum rate of runoff occurs when a constant rainfall intensity falls on a catchment for as long or longer than the time of concentration.
- The runoff coefficient is the same for each rainfall intensity and for all return intervals. This is an assumption inherent in the original proposal by Kuich-



**FIGURE 21.16** Composite catchments. (Ponce, Miguel Victor, 1989, *Engineering Hydrology: Principles and Practices*, pp. 128, 129, 138. Adopted by permission of Pearson.)

ling in 1877. Runoff coefficients are typically higher for the less frequent storms because of the reduction of the effect of the rainfall abstractions. Runoff coefficients are also increased for the higher intensity rainfalls.

- The frequency of the peak discharge is the same as that of the rainfall intensity for the given time of concentration. However, this may not be necessarily true due to variations in surface conditions.
- Most localities restrict the use of the rational method to small drainage basins, ranging from 20 to 200 acres, and may limit the applicability of the rational method with time-of-concentrations greater than 60 minutes.

**TABLE 21.11** Peak Discharge for Incremental Intensity Duration

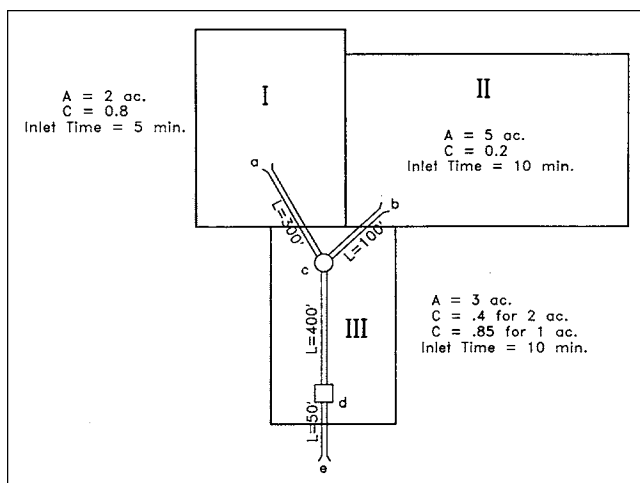
INTENSITY DURATION (min)	RAINFALL INTENSITY (in/hr)	CONTRIBUTING AREA OF B (ac)	$Q_p$ (cfs)
20	5.16	26.7	149.7
30	4.11	40.0	135.6
40	3.10	53.3	114.6
50	2.99	66.7	122.6
60	2.65	80	119.3

### Design Example for the Rational Method

Using the rational equation to size pipes and inlets in a storm drain network involves a systematic method of determining a time of concentration to the design point, finding the corresponding rainfall intensity and applying the rational equation. For each new design point, a new  $t_c$  and intensity are computed to determine the corresponding peak discharge.

An example will illustrate the procedure. Consider the schematic layout in Figure 21.17. The requirement is to determine the discharge at point e using the rational method for a ten-year recurrence interval storm. A storm drainage network is dendritic in shape—that is, the branches of the pipe network converge in the downstream direction. As such, the discharge increases and the pipes become larger in the downstream direction. Therefore, the design begins at the most upstream catchment for a particular branch of the pipe network. The design discharge for a pipe, using the rational method, is found by determining the longest time of concentration to the inlet of the pipe at the most upstream catchment. Recall that the time of concentration is a combination of overland flow time (i.e., inlet time) plus channel time where the time of concentration is, typically, the longest accumulated time of overland flow plus channel time.<sup>4</sup> To compute the channel time in a stream, the velocity is computed from Manning's equation using bank full conditions.

The computations are facilitated with the aid of the pipe design Table 21.12. This example is to illustrate the application of the rational method; thus, the pipe design part is ignored. The velocity for flow through the pipes is assumed to be 2.5 fps. In actual practice, the flow time is based on the actual discharge, pipe size, and pipe slope. The design



**FIGURE 21.17** Schematic for application of the rational method.

discharge for pipe a-c is based on the inlet time of 5 minutes. The area of the catchment is determined by drawing the drainage divides and planimetrying the area.<sup>5</sup> The runoff coefficient is determined from the land use, topography and other contributing factors. From the IDF curve, Figure 21.12, the rainfall intensity is 6.5 in./hr and the corresponding peak discharge (from Eq. (21.10)) to the upper end of pipe a-c is given in column 9 as 10.4 cfs. Similar analysis was applied to pipe b-c to find  $Q_p = 5.9$  cfs.

Consider pipe c-d. The discharge to design point c must account for the accumulated CA value of all contributing catchments draining to point c as shown in column 6 ( $1.6 + 1.0 = 2.6$ ). There are two flow paths that must be considered to determine the  $t_c$ : the overland flow from catchment I, column 7 ( $= 5$  min), plus time in pipe from a to c, column 15 ( $= 2$  min); or the overland flow from catchment II ( $= 10$  min) plus time in pipe from catchment II to point c ( $= 0.7$  min). The larger  $t_c$  is 10.7 minutes. This time is used to determine the intensity  $i$ , column 8. The peak discharge to point c is

$$Q_p = i \Sigma(CA) = (5.8)2.6 = 15.1 \text{ cfs} \quad (21.20)$$

Point d is a surface inlet structure. The CA value used in calculating the peak discharge at point d is the accumulated CA ( $= 2.6$ ) from all upstream catchments contributing to point d plus the CA ( $= 1.65$ ) that contributes the surface runoff from catchment III. The time of concentration is the longest flow path to point d, the flow path already shown to be b-c, plus the time of flow in pipe from point c to d ( $= 2.7$  min). The total time of 13.4 minutes is used to determine the intensity of 5.3 in/hr. The corresponding peak discharge is 22.8 cfs flowing through pipe d-e. Note that the  $t_c$  used for pipe section d-e was the accumulated time from catchment II to point d. Point d is a surface inlet with a  $t_c$  of 10 minutes. If this  $t_c$  had been greater than the accumulated time from catchment II, the intensity would have been selected using the  $t_c$  from catchment III instead.

Note in the foregoing example problem that the flows at each design point are obtained by successive applications of the rational equation. The time of concentration accumulates as the design progresses downstream. Accordingly, new rainfall intensities are determined at each design point. One typical misuse of the rational method is to calculate flows for each individual catchment and add them at each successive design point. This procedure results in an overestimate of the design flow that accrues in the downstream sections.

<sup>5</sup> A drainage divide shows the area contributing runoff to a particular point. In the simplest case, a drainage divide is determined by starting at the inlet location or other point of interest and tracing a line that is perpendicular to the contour lines. Eventually the line will loop back to the point of origin. However, curbs, drainage ditches and other conveyance structures may alter the direction of the drainage divide.

<sup>4</sup> Some exceptions to this generalized rule have been previously discussed.

**TABLE 21.12 Pipe Design Table for Rational Method Design Example**

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
FROM	T <sub>0</sub>	INCR. AREA ac	C	A × C	ACCUM. (ac)	t <sub>c</sub> (min) TO UPPER END (OR TO INLET)	RAINFALL INTENSITY (in./hr)	Q <sub>p</sub> (cfs)	DIAM. (in.)	L (ft)	SLOPE	VEL (fps)	Q <sub>max</sub> (cfs)	TIME IN PIPE (min)	ACCUM. TIME (min)	INV. UP	INV. LOW
a	c	2	0.8	1.6	1.6	5	6.5	10.4	300	300	2.5	2.5	2.0	7.0			
b	c	5	0.2	1.0	1.0	10	5.9	5.9	100	100	2.5	2.5	0.7	10.7			
c	d	—	—	—	2.6	10.7	5.8	15.1	400	400	2.5	2.5	2.7	13.4			
d	e	3	.55	1.65	4.3	13.4	5.3	22.8	50	50	2.5	2.5	0.3	13.7			

Column

- 1,2 Defines the pipe section. Column 1 is the upper end and column 2 the lower end of the pipe
- 3 The incremental area contributing to runoff to the pipe section; this column has an entry only if there is an inlet at upper end of pipe
- 4 The runoff coefficient for the incremental area (column 3)
- 5 Column 3 multiplied by column 4
- 6 Accumulated (area × runoff coefficient) draining to the pipe section at the upper end (column 1)
- 7 Time of concentration to the upper end of the pipe section
- 8 Rainfall intensity based on t<sub>c</sub> of column 7
- 9 Peak discharge to upper end of pipe section equal to column 6 × column 8
- 10 Pipe diameter of pipe section defined by columns 1 and 2
- 11 Length of pipe
- 12 
$$\text{Slope} = \frac{\text{Upper Inv. Elev} - \text{Lower Inv. Elev}}{L}$$
- 13 Velocity of discharge in pipe, = Q<sub>p</sub>/area of flow
- 14 Maximum flow capacity of pipe
- 15 Flow time in pipe = Vel/Length
- 16 Accumulated time to lower end on pipe section equal to column 7 + column 15
- 17 Upper invert elevation of pipe
- 18 Lower invert elevation of pipe

### OPEN CHANNEL FLOW IN PIPES AND CHANNELS

The two basic forces that act on flow in an open channel are gravity (pulling it downward) and friction (retarding its movement). These factors then have to be combined with the basic concept of continuity—that is, the flow in one discrete section of channel or pipe must equal the flow out of that section. This is the basic premise for estimating open channel flow in channel and pipes. Therefore, open channel flow,  $Q$ , is estimated by combining the continuity equation,  $Q = VA$  with Manning's equation for velocity.

$$Q = \left( \frac{1.49}{n} \times R^{2/3} S^{1/2} \right) \times A \quad (21.21)$$

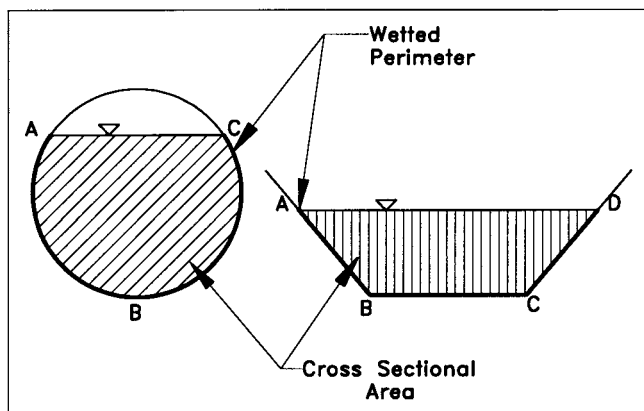
where the term in parentheses represents Manning's velocity for open channel flow. In Eq. (21.21),  $n$  = Manning's coefficient of roughness,  $R$  = hydraulic radius (ft), defined as the ratio of the cross-sectional area to the wetted perimeter,  $S$  is the slope of the energy grade line (ft/ft), and  $A$  is the cross-sectional area of the flow (ft<sup>2</sup>). In the equation, the bracketed term represents the velocity of flow as per the Manning equation.

The cross-hatched area of Figure 21.18 shows the cross-sectional area of flow for a circular section and a trapezoidal section. The wetted perimeter for the circular section is arc ABC. Line segments ABCD show the wetted perimeter on the trapezoidal section.

The roughness coefficient represents an estimate of the resistance of flow. Large  $n$  values correspond to high resistance to flow. Table 21.13 provides  $n$  coefficients for commonly occurring channel materials.

Following are some of the factors that affect flow resistance.

- Surface roughness, due to size and shape of the grains of the material forming the wetted perimeter, increases the roughness coefficient as the coarseness of the grains increase.
- Vegetation is also a form of surface roughness. The degree to which vegetation impacts flow resistance depends on density, height and type of vegetation.



**FIGURE 21.18** Cross-sectional area and wetted perimeter for circular section and trapezoidal section.

- Channel irregularities such as those found in natural streams. Channel irregularities include depressions in the channel bed, humps, sand bars, ridges, etc., all of which change the wetted perimeter size and shape of cross section. Abrupt channel changes have a larger effect on the  $n$  value than small or gradual irregularities.

- Channel alignment and curvature has some impact on flow resistance. A large radius of curvature has less effect than a short radius.

- Obstructions such as logs, stumps, debris, trash, rocks, and other items that interrupt and severely alter the flow path increase the flow resistance.

In a circular pipe flowing full with diameter  $D$ , the hydraulic radius is  $D/4$ . In a pipe flowing less than full (Figure 21.19), the relationship between depth  $y$ , cross-sectional flow area  $A$ , wetted perimeter  $P$ , and hydraulic radius  $R$  is given by the following relationships:

$$y = \frac{D}{2} (1 - \cos \Theta) \quad (21.22)$$

$$A = \frac{D^2}{4} (\Theta - \sin \Theta \cos \Theta) \quad (21.23)$$

$$= \frac{D^2}{4} (\Theta - \frac{1}{2} \sin 2\Theta)$$

$$P = D\Theta \quad (21.24)$$

$$R = \frac{A}{P} = \frac{D}{4} \left( 1 - \frac{\sin \Theta \cos \Theta}{\Theta} \right) \quad (21.25)$$

$$= \frac{D}{4} \left( 1 - \frac{\sin 2\Theta}{2\Theta} \right)$$

### Energy Losses in Pipe Networks

Flow in a conduit is retarded by resistance and turbulence. Resistance and turbulence are measured in terms of the energy consumed to overcome them, which is mostly in the form of frictional losses along straight sections of the conduit. Turbulence is also created where there is an abrupt change in cross section of the flow (e.g., sudden contraction or sudden expansion) or by other interferences of the flow. The concept of uniform flow in open channels is a balance between the available energy that is generated by gravity and the consumed energy of the flow resistance.

Uniform flow occurs whenever the depth, water area, velocity, and discharge at every section in a channel reach remain constant. For such stringent conditions to exist, not only must channel geometry be consistent, but the drop in potential energy (due to the fall in elevation in the channel bed) must be equal to the energy consumed through boundary friction and turbulence. Hence, if the channel geometry and slope remains constant and the reach is sufficiently long, then for a given discharge there will exist one,

and only one, depth at which the flow will be uniform. This depth is referred to as normal depth.

The energy at a particular location in a pipe is given by Bernoulli's equation:

$$E = \frac{p}{\gamma} + \frac{V^2}{2g} + z \quad (21.26)$$

where  $p/\gamma$  is the pressure head,<sup>6</sup>  $V^2/2g$  is the velocity head and  $z$  is the elevation head. A plot of points given by Eq. (21.26) at every section along a length of a pipe is known as the energy grade line (EGL). By utilizing the conserva-

<sup>6</sup> In hydraulics, the term "head" is synonymous with energy.

**TABLE 21.13 Values of the Roughness Coefficient  $n$**

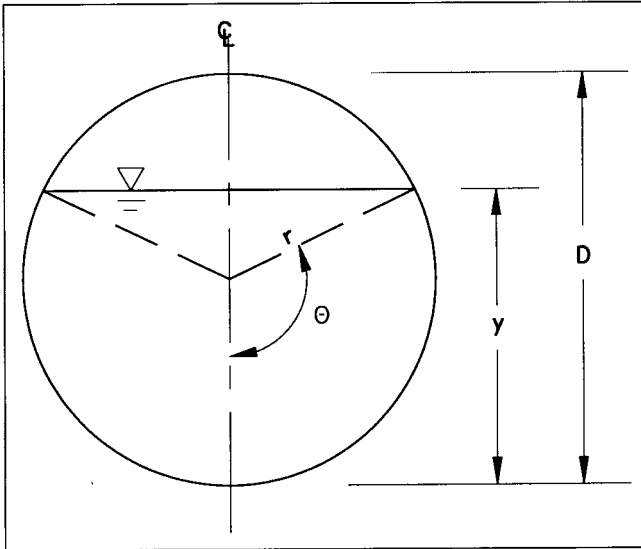
TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
A. Closed Conduits Flowing Partly Full			
A-1. Metal			
a. Brass, smooth	0.009	0.010	0.013
b. Steel			
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlets, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminate, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030



TABLE 21.13 Values of the Roughness Coefficient  $n$  (Continued)

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
B. Lined or Built-up Channels			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom bloat finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	—	0.500

Reprinted from Chow, V.T., *Open Channel Hydraulics*, McGraw-Hill, New York, 1959. Reprinted with permission from McGraw-Hill.



**FIGURE 21.19** Nomenclature for pipe flowing partially full.

tion-of-energy principle along with the Bernoulli equation, the energy at point B downstream of A is less than the energy at point A by an amount equal to the energy lost between point A and B. This is mathematically expressed as:

$$\frac{P_A}{\gamma} + \frac{V_A^2}{2g} + z_A = \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + z_B + \Sigma h_L \quad (21.27)$$

where the  $\Sigma h_L$  term accounts for the total energy lost between points A and B.

The energy gradient (hydraulic slope) is defined as the energy loss per length of channel:

$$S = \frac{h_L}{L} \quad (21.28)$$

In closed conduits under pressure flow, the potential energy plus the pressure energy in the system is given as  $z + p/\gamma$ . This term, also referred to as the static head or piezometric head, represents the level to which the liquid would rise if a piezometer tube were installed at this location. A continuous line drawn through the tops of these piezometer columns is the hydraulic grade line (HGL). In open channel flow, the water surface itself is the hydraulic grade line. In uniform flow, the slope of the channel, the slope of the water surface, and the slope of the hydraulic gradient are all parallel.

Head loss is attributed to pipe friction, abrupt changes in the cross-sectional area (due to entrance and exit of flow to or from a reservoir or manhole) or other types of appurtenances and obstructions within the flow path. The two most commonly used friction-loss equations for determining hydraulic resistance are the Darcy equation and the Manning equation. The Darcy-Weisbach equation is given as

$$h_f = f \left( \frac{L}{D} \right) \left( \frac{V^2}{2g} \right) \quad (21.29)$$

where  $h_f$  = the friction energy loss,  $f$  = the Darcy-Weisbach friction factor,  $L$  = length of conduit,  $D$  = diameter of the conduit,  $V$  = mean velocity in the conduit, and  $g$  = gravitational acceleration.

The Darcy-Weisbach friction factor is obtained from the Moody diagram (see Figure xx.xx), which relates  $f$  to the Reynolds number and the relative roughness of the conduit.<sup>7</sup> Conversion of the Darcy-Weisbach friction factor  $f$  to the Manning roughness coefficient  $n$  is given by

$$n = 0.0926 R^{1/6} f^{1/2} \quad (21.30)$$

where  $R$  is the hydraulic radius.

In the Manning equation, the slope of the energy line  $S$  is given as  $S = h_f/L$  where  $h_f$  is the friction loss and  $L$  is the length of conduit. Substituting for  $S$  and solving for  $h_f$  in the Manning equation gives

$$h_f = \left( \frac{29 n^2 L}{R^{4/3}} \right) \frac{V^2}{2g} \quad (21.31)$$

In addition to pipe friction energy losses, there are minor energy losses through manholes, bends, and other appurtenances. Energy losses through such appurtenances are proportional to the velocity head in the appurtenance and are accounted for as follows:

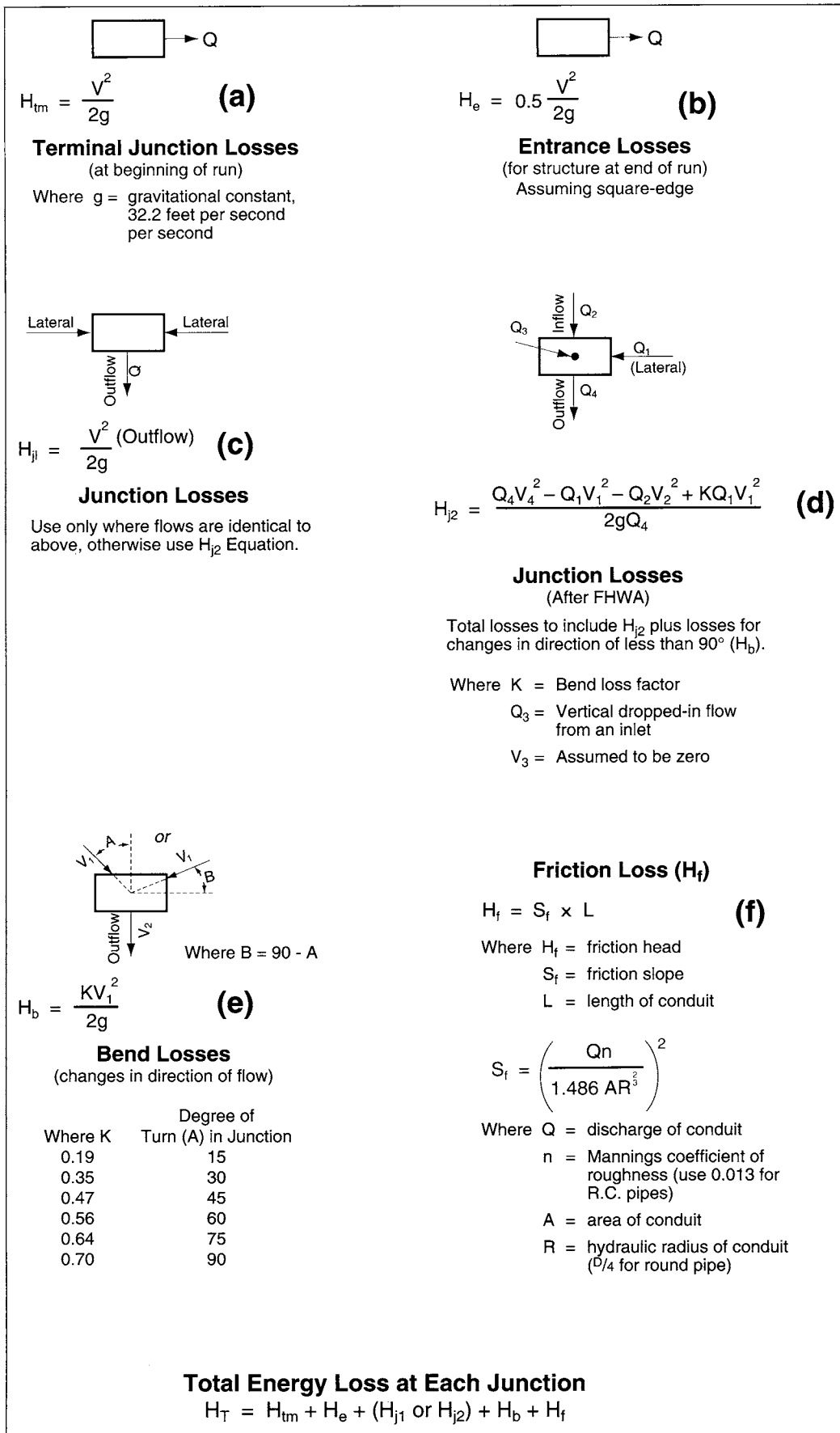
$$h_L = k \frac{V^2}{2g} \quad (21.32)$$

where  $k$  is an empirical coefficient that accounts for the energy loss for a particular appurtenance or configuration. (Table 26.13 gives  $k$  values for various conditions.)

Minor energy losses through manholes and junctions are the result of the deflection angle of the flow, exit loss as the flow leaves the incoming pipe, and entrance losses as the flow enters the discharge pipe of the manhole. As indicated by Eq. (21.32), high velocities result in high energy losses. However, the energy loss through the manhole can be significantly reduced through inlet shaping. Inlet shaping provides a flow trough at the bottom of the manhole to direct the flows from the incoming pipes to the downstream pipe. Exit loss coefficients, energy loss coefficients, and other loss coefficients are typically identified in the local standards criteria. Figure 21.20 gives some values for  $k$  as prescribed by American Association of State Highway and Transportation Officials (AASHTO).

In addition, losses for deflection of flows from the incoming pipes to the outgoing pipe have to be considered. When a junction or manhole has more than one incoming

<sup>7</sup> The Reynolds number is a dimensionless parameter that is a ratio of inertial forces to viscous forces. Relative roughness is the ratio of element roughness size to conduit size. Consult basic hydraulic texts for further discussion.



**FIGURE 21.20** Summary of energy losses through manholes. From *Model Drainage Manual*, copyright 1991 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

pipe, each pipe is analyzed for the total head loss of the combined effects (entrance, exit, deflection, etc.). The pipe producing the greatest head loss is assumed to be the controlling pipe and the EGL and HGL is based on this greatest value.

Energy losses also occur at inlets that collect surface runoff and discharge directly into the storm sewer system. The water dropping into the bottom of the manhole produces turbulence within the water that is already flowing through the manhole, thus disturbing (even further) the flow from the upstream pipes. These energy losses are added to the conventional losses attributed to the exit and entrance losses from the discharge ends of the incoming pipes. Other losses, which are less frequently encountered in the pipe network, include difusor/confusor losses, losses associated with converging or diverging wyes, and bends. Figure 21.20 summarizes the various minor energy losses through manholes.

### Hydraulic Grade Line in Pipe Networks

Most storm sewers are intended to operate as gravity flow for the design discharge. Occasionally, due to unavoidable field conditions like high flow rates or low-lying flat areas where little fall is available, the system is surcharged (i.e., operates as pressure flow). Even though the pipes were sized to function under gravity flow, accumulated head losses can cause surcharge in the system if, during design, sufficient allowances were not made to compensate for the accrued head losses. Surcharge in the system occurs when the hydraulic grade line rises above the crown of the pipe.

Most localities prefer the storm drainage system to operate as gravity flow for the design discharge. However, surcharge of the system for the design flow is tolerable if the hydraulic grade line is maintained below a certain limit. Typically, this is a specified distance above the crown of the pipe and below the ground surface elevation (e.g., the inlet elevation or rim elevation). Some localities have developed criteria requiring the storm sewer system to be evaluated for specified storms larger than the design storm, such as in low-lying areas where there is little to no overland relief should runoff surcharge out of the system.

Hydraulic grade line calculations are easily summarized with the aid of an HGL calculation form in Figure 21.21. The hydraulic grade line is parallel to and below the energy grade line along the length of conduit, a distance equal to the velocity head  $V^2/2g$ . Energy losses through manholes and other appurtenances are graphically shown as abrupt jumps in the EGL. Figure 21.22 is a schematic representation of the HGL table.

Computations for the HGL begin at a position where the water surface elevation is known or where a beginning HGL elevation can easily be determined, such as with the end of a pipe discharging into a lake. In this case, assuming the velocity head of the water at the lake's surface near the discharge end of the pipe is negligible, the starting EGL

elevation is equal to the water surface elevation of the lake as well as the HGL of the lake. Should the end of the pipe discharge into a channel, the beginning water surface elevation can be found from backwater computations in the stream channel.

Once you have established the starting point, the computations proceed in the upstream direction by adding the energy losses attributed to exit, entrance, and junction losses to the EGL.

If storm sewer systems are assumed to operate with velocities in the 3–6 fps range, the HGL is approximated by the EGL. For normal velocities in the storm drain system, the difference between the EGL and the HGL elevations is less than 0.5 ft (i.e., for  $V = 5$  fps,  $V^2/2g \approx 0.4$  ft). Although the HGL is actually lower than the EGL by this amount, a relatively small velocity head is considered negligible, and many localities assume that the EGL approximates the HGL. A common design practice to anticipate and compensate for these velocity heads is to match crowns of the outgoing and incoming pipes of the manhole when the outgoing pipe is larger. If the two pipes are of equal size, common design practice would drop the outgoing pipe approximately 0.25 to 0.50 ft to account for losses that occur within the structure.

However, recognize that the occasion occurs, perhaps rarely, where such approximations can be substantially different. One case occurs when the discharge velocity in the incoming pipe is significantly greater than the velocity of the outgoing pipe in a manhole. This is illustrated at structure number 3 in the following example.

**EXAMPLE 4:** Determine the HGL for the data given in Table 21.14. All pipe is reinforced concrete pipe (RCP),  $n = 0.015$ . A profile of the pipe system is shown in Figure 21.23.

In this example, the water surface is known at the downstream side of structure 1. The head loss through the structure is computed to find the EGL on the upstream side. The head loss through the section of pipe is added to this to find the EGL on the downstream side of the next upstream structure. The process is repeated until the HGL falls below the crown of the pipe or until the end of the run. Detailed computations follow with reference to HGL computation form shown in Table 21.15. The form is utilized best if each horizontal line is computed sequentially.

### Structure 1

Columns 2, 3, and 4. Structure 1 is a headwall with pipe discharging into a reservoir at water surface (WS) elevation = 42.5. The velocity of the water surface of the reservoir is assumed to be zero if we ignore the disturbance in the immediate vicinity of the headwall caused by the momentum of the incoming flow. The EGL at the downstream side of the headwall is

PROJECT: \_\_\_\_\_ DESIGNED BY: \_\_\_\_\_ CHECKED BY: \_\_\_\_\_  
 DESIGN STORM: \_\_\_\_\_ DATE: \_\_\_\_\_ DATE: \_\_\_\_\_

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Struct. No.	D/S	$h_v$	D/S	D	Q	V	L	$S_f$	$H_f$	Terminal Structure Losses		Junction Losses						U/S	$h_v$	U/S	Rim	
	EGL	out	HGL		in	cfs	fps	ft	ft/ft	ft	$H_{ex}$	$H_{entr}$	$H_0$	$H_1$	$H_b$	$H_T$	$H_T$	0.5 $H_T$	EGL	in	EGL	Elev.

- Column
- 1 Structure identification label
  - 2 EGL elevation at downstream side of structure = preceding column 19 + preceding column 10
  - 3 Velocity head at the discharge side of the structure =  $V^2/2g$
  - 4 HGL elevation = column 2 - column 3
  - 5 Pipe diameter
  - 6 Discharge
  - 7 Discharge velocity
  - 8 Pipe length between structures
  - 9 Friction slope =  $\left( \frac{Vn}{1.49 R^{2/3}} \right)^2$
  - 10 Friction head loss = column 8 × column 9 (fig. 14.20-f)
  - 11 Exit head loss at terminal structure on D/S end of run (fig. 14.20-a)
  - 12 Entrance head loss at terminal structure on U/S end of run (fig. 14.20-b)
- Column
- 13 Contraction head loss for outflow from a nonterminal structure =  $k_{ex}(V_{out}^2/2g)$
  - 14 Expansion head loss for inflow into nonterminal manhole =  $k_{entr}(V_{in}^2/2g)$
  - 15 Bend head loss due to deflection of flow through manhole (fig. 14.20-e)
  - 16 Total head loss through the junction = column 13 + column 14 + column 15
  - 17 Head loss for contributing flow from surface inlet = column 16 × 1.3 if surface discharge is greater than 10% of the mainline flow
  - 18 Reduction in head loss of column 16 for inlet shaping = column 16 or column 17 × 0.5
  - 19 EGL on the upstream side of the manhole structure = column 2 + column 11 or column 16, 17, or 18
  - 20 Velocity head of the incoming pipe =  $V_{in}^2/2g$
  - 21 HGL on the upstream side of the manhole structure = column 19 - column 20
  - 22 Elevation of top of manhole or inlet structure

**FIGURE 21.21** Hydraulic Grade Line Computation Form.

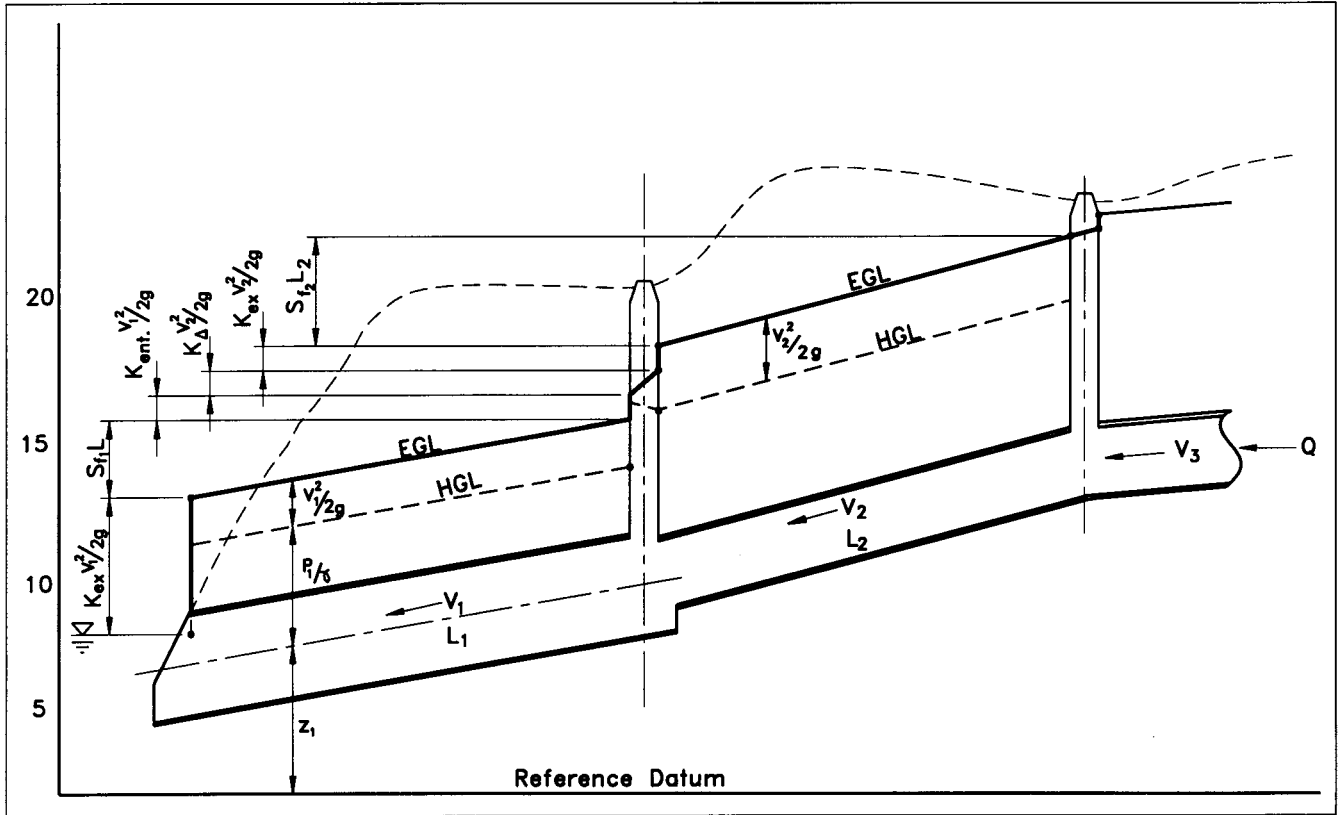


FIGURE 21.22 Schematic diagram for energy grade line and hydraulic grade line.

$$EGL = \frac{p}{\gamma} + \frac{V^2}{2g} + z \quad (21.33)$$

If, as assumed,  $V = 0$ , and we use gage pressure as the reference, then  $p/\gamma = 0$  and the EGL is equal to the water surface elevation  $z$ . Because  $V = 0$ , the HGL is also the water surface elevation (i.e.,  $HGL = EGL - V^2/2g$ ).

Column 11. Assuming a loss coefficient  $k$  for a square-edged headwall = 1.0. The head loss through the headwall is

$$H_{ex} = (k_{ex}) \frac{V^2}{2g} = 1 \left( \frac{6.69^2}{2g} \right) = 0.69 \text{ ft} \quad (21.34)$$

where  $V$  is the average discharge velocity in the pipe ( $V = Q/A$ ).

Column 19. On the upstream side of the headwall, the EGL is equal to the EGL at the downstream side plus the head loss through the structure.

$$(U/S \text{ EGL})_1 = 42.50 + 0.69 = 43.19 \text{ ft} \quad (21.35)$$

Columns 20 and 21. The HGL at the upstream side of the headwall is equal to the EGL minus the velocity head in the pipe:

$$(U/S \text{ HGL})_1 = 43.19 - \frac{6.69^2}{2g} = 42.50 \text{ ft} \quad (21.36)$$

Columns 6 and 7. From Manning's equation the maximum discharge through the 24-inch pipe is

$$Q_{max} = \frac{1.49}{0.015} \left( \frac{2}{4} \right)^{0.67} (0.0075)^{0.5} \left( \frac{2^2 \pi}{4} \right) = 17.0 \text{ cfs} \quad (21.37)$$

The pipe is carrying 21 cfs and is, therefore, under pressure—that is, the HGL is above the crown of the pipe. The velocity in the pipe is

TABLE 21.14 Data for HGL/EGL Example 4

FROM	TO	DIAMETER (in)	LENGTH (ft)	SLOPE (ft/ft)	DISCHARGE (cfs)
4	3	15	200	0.045	15
3	4	21	400	0.010	15
2	1	24	300	0.0075	21

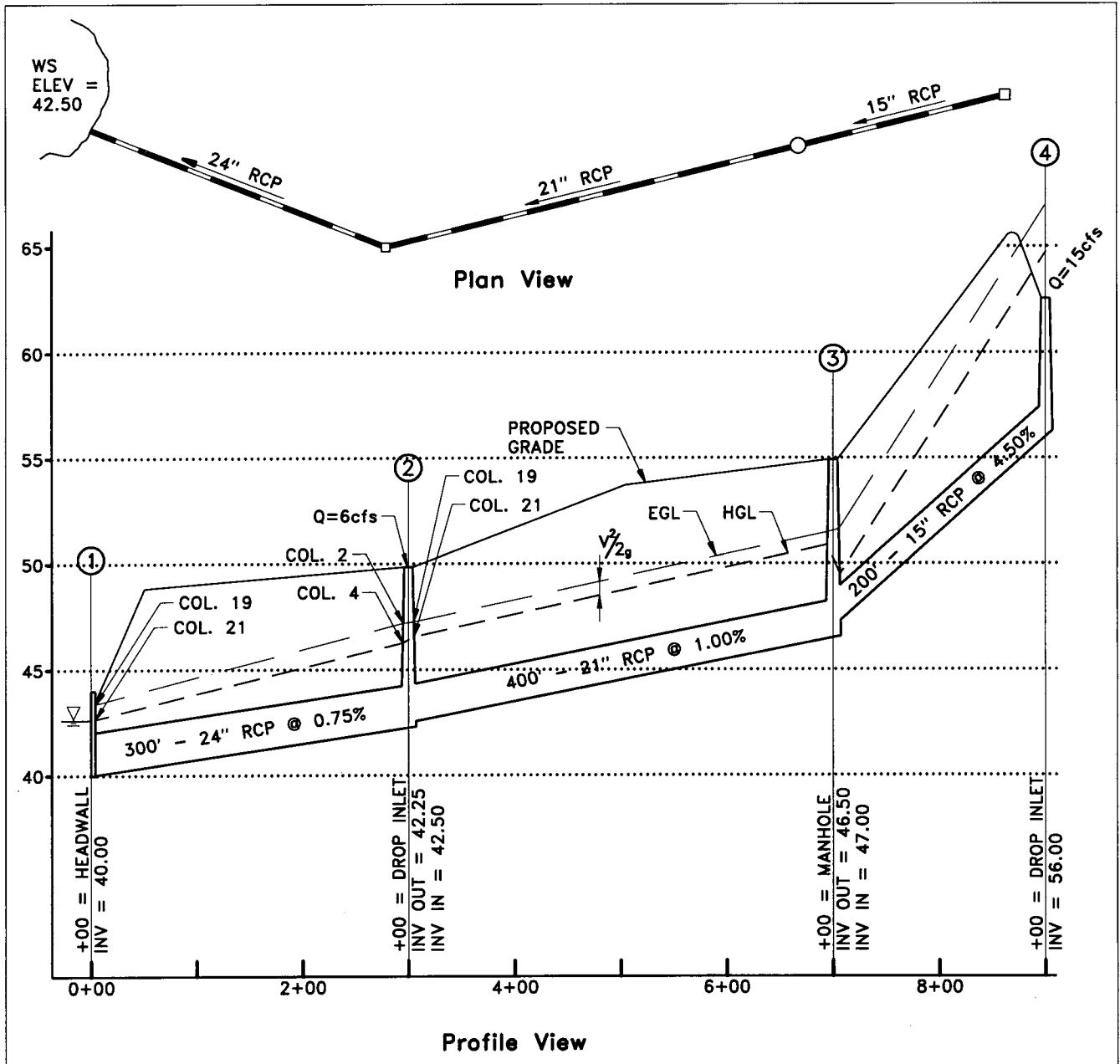


FIGURE 21.23 Plan and profile view of pipe system for Example 4.

$$V = \frac{Q}{A} = \frac{21}{\frac{\pi 2^2}{4}} = 6.69 \text{ fps} \quad (21.38)$$

Columns 9 and 10. The friction slope of the water is

$$S_f = \left( \frac{(6.69)(0.015)}{(1.49) \left(\frac{2}{4}\right)^{0.67}} \right)^2 = 0.011 \text{ ft/ft} \quad (21.39)$$

The total head loss in the 24-inch pipe is

$$H_f = (0.011)(300) = 3.30 \text{ ft} \quad (21.40)$$

**Structure 2**

Columns 2, 3, and 4. The EGL on the downstream side of structure 2 is equal to the EGL on the upstream side of structure 1 plus the head loss in the pipe between structures 1 and 2:

$$(D/S \text{ EGL})_2 = 43.19 + 3.30 = 46.49 \text{ ft} \quad (21.41)$$

The HGL on the downstream side of structure 2 is one velocity head below the EGL:

**TABLE 21.15 Hydraulic Grade Line Computation Form**

PROJECT: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
DESIGN STORM: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
PROJECT: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
DESIGN STORM: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
PROJECT: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
DESIGN STORM: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
PROJECT: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
DESIGN STORM: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
PROJECT: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
DESIGN STORM: _____		DESIGNED BY: _____		CHECKED BY: _____		DATE: _____		DATE: _____		DATE: _____		DATE: _____										
STRUC. No.	D/S EGL	$H_v$ OUT	D/S HGL	D (in.)	Q (cfs)	V (fps)	L (ft)	$S_r$ (ft/ft)	$H_r$ (ft)	$H_{ex}$ (ft)	$H_{ant}$ (ft)	$H_o$ (ft)	$H_i$ (ft)	$H_b$ (ft)	$H_T$ (ft)	$H_r$ (ft)	$H_T$ (ft)	0.5 $H_T$ (ft)	U/S EGL	$h_v$ IN (ft)	U/S HGL	RIM ELEV.
1	42.50	0	42.50							0.69									43.19	0.69	42.50	
2	46.49	0.69	45.80	24	21	6.69	300	0.011	3.30			0.17	0.21	0.28	0.66	0.86	0.43	0.43	46.92	0.60	46.32	49.50
3	51.72	0.60	51.12	21	15	6.24	400	0.012	4.80			0.15	0.81	0.96				0.48	52.20	2.31	49.89	54.50
4	66.50	2.31	64.19	15	15	12.2	200	0.072	14.3										67.66		67.66	62.00
Column											1.16	Column										
1	Structure identification label																					
2	EGL elevation at downstream side of structure = preceding column 19 + preceding column 10																					
3	Velocity head at the discharge side of the structure = $V^2/2g$																					
4	HGL elevation = column 2 – column 3																					
5	Pipe diameter																					
6	Discharge																					
7	Discharge velocity																					
8	Pipe length between structures																					
9	Friction slope = $\left(\frac{Vn}{1.49 R^{2/3}}\right)^2$																					
10	Friction head loss = column 8 × column 9 (Figure 21.20(f))																					
11	Exit head loss at terminal structure on D/S end of run (Figure 21.20(a))																					
12	Entrance head loss at terminal structure on U/S end of run (Figure 21.20(b))																					



$$(D/S \text{ HGL})_2 = 46.49 - \frac{6.69^2}{2g} = 45.80 \text{ ft} \quad (21.42)$$

Columns 13, 14, 15, and 16. The head loss through the structure is the combined effects of sudden contraction, sudden expansion, and momentum loss as the flow is deflected through the 45°. Proceeding in the upstream direction, the head loss due to sudden contraction as the flow leaves structure 2 and enters the pipe is

$$H_0 = k_0 \left( \frac{V^2}{2g} \right) = 0.25 \left( \frac{6.69^2}{2g} \right) = 0.17 \text{ ft} \quad (21.43)$$

The head loss for sudden expansion for incoming flow into structure 2 is

$$H_i = 0.35 \left( \frac{6.24^2}{2g} \right) = 0.21 \text{ ft} \quad (21.44)$$

The bend loss coefficient found on Figure 21.20 is 0.47. Using the incoming velocity,

$$H_b = 0.47 \left( \frac{6.24^2}{2g} \right) = 0.28 \text{ ft} \quad (21.45)$$

The total head loss through the structure  $H_T = 0.17 + 0.21 + 0.28 = 0.66$  ft. The head loss coefficients for sudden expansion and sudden contraction vary according to the standard design criteria of the local jurisdiction.

Column 17. Surface water entering through the top of the structure disrupts the flow in the mainline and thus creates head loss. AASHTO's recommendation is to increase the total head loss through the structure by 30% to account for this additional head loss. This adjustment is only applicable for surface water entering the structure in the amount equal to or greater than 10 to 20% of the flow in the mainline of the pipe, based on local criteria. In this case, 6 cfs  $\geq$  0.1 (15 cfs) and the adjustment is made;  $1.3(0.66) = 0.86$  ft. Column 18. Efficiency of flow through the manhole can be improved by constructing a trough through the bottom of the structure to direct the flow. This is known as inlet shaping. Most design standards for storm sewer manholes incorporate inlet shaping. If the storm sewer structure incorporates inlet shaping, the total head loss through the structure, in this case, is reduced 50% (this value varies according to local criteria). Column 16 (or column 17 for surface inlet structures) is decreased. For this structure, col. 18 =  $0.5(0.86) = 0.43$  ft. However, hydraulic experts have questioned the effectiveness of inlet shaping for highly turbulent excessive surcharge flows.

**Structure 3.** Computations for structure 3 are similar to structures 1 and 2. Since structure 3 is a manhole type structure, and no surface water enters through the top, column 17 does not apply. Note that the EGL across structure 3 increases, while the HGL decreases, due to the high incoming discharge velocity. This is one case where the HGL cannot be approximated to the EGL.

**Structure 4.** Structure 4 is a terminal structure, and therefore only column 12 applies for computing the head loss. Notice the HGL elevation is above the rim elevation of the structure. This shows that for the given discharges and pipe design, the top of the structure is inundated by three ft of water. If the design discharges are for a frequent storm (i.e., recurrence interval  $\leq$  10 yr), this pipe design, in all likelihood, would be unacceptable.

### HGL Summary

HGL computations are vital to the proper design and construction of storm sewer systems. Keep in mind that the method described in the previous sample problem is typical; however, many localities have additional or different techniques. Some localities allow head losses at junctions to be reduced by inlet shaping, as suggested by column 18 in the HGL spreadsheet, while others do not. Some state Departments of Transportation (DOTs) also take conservative approaches to bend losses and junction losses, so always consult the local design criteria, if available.

Another conservative approach is to assume that the minimum depth of the HGL along a pipe at the junction location (going upstream) can be no lower than a certain depth in the pipe, such as normal depth of  $(D_c + D)/2$ . For example, if calculated junction losses show that the elevation of the HGL is near the bottom of the upstream pipe, the user would start the HGL for the upstream pipe at  $(D_c + D)/2$ , not the calculated HGL value.

### PAVEMENT DRAINAGE

Stormwater runoff on roads is a safety hazard in numerous ways. Hydroplaning, reduced visibility, and ice conditions are dangerous to vehicle occupants and pedestrians. In an urban setting, these hazardous conditions are substantially magnified due to the increased traffic and pedestrian density. It is imperative to remove the stormwater runoff from roads quickly and efficiently to mitigate the safety risks. Effective removal of runoff is affected by the geometric characteristics of the roadway, such as longitudinal slope, cross slope, type of curb and gutter section, and ditch section. These geometric characteristics dictate the location and spacing of inlets and catch basins.

### Flow in Curb and Gutter

Most urban and suburban land development projects, of moderate to high density, use curb or curb and gutter as an integral part of the street section. For drainage purposes, the gutter is considered as that part of the pavement near the curb that conveys the runoff. In many cases, streets are constructed with composite gutter and pavement sections, where the gutter portion has a greater cross slope than the pavement. A composite gutter section results in a decrease in elevation of the flow line at the face of curb relative to where the flow line would be if the pavement were extended to the face of curb at the designed cross slope. In these instances, the gutters typically vary between 1 and 3

ft wide depending on local standards. The steeper cross slope of the gutter concentrates the flow towards the gutter to improve inlet efficiency and minimize the width of the runoff as it encroaches the travel lanes (spread).

There are some examples, such as on the inside (left) lane adjacent to a raised median, where composite gutters are not provided, and the roadway surface and slope tie directly into the face of the curb. These are sometimes referred to as header curbs, and the engineer should consider the lack of the gutter section when calculating flow along curb and gutter sections. The Federal Highway Administration's HEC-12, *Drainage of Highway Pavements*, is the basis for many local guidance design standards and criteria.

Flow conveyed in gutter sections with uniform cross slope is

$$Q = \frac{0.56}{n} S_x^{5/3} S^{1/2} T^{8/3} \tag{21.46}$$

which is a modification of the Manning equation and accounts for the disproportionate ratio of depth to top width of water surface. In the equation,  $Q$  = discharge (cfs),  $n$  = Manning's  $n$  coefficient,  $S_x$  = pavement cross, slope (ft/ft),  $S$  = longitudinal slope of the gutter (ft/ft),  $T$  = width of flow or spread (ft), which is the distance from the face of curb to the waterline limits in the pavement. Values for Manning's  $n$  coefficient for various pavement types are given in Table 21.16. The nomograph of Figure 21.24 is the so-

**TABLE 21.16 Manning's  $n$  Values for Street and Pavement Gutters**

TYPE OF GUTTER OR PAVEMENT	RANGE OF MANNING'S $n$
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, <i>increase</i> above values by $n$ of	0.002

Note: Estimates are by the Federal Highway Administration. Reference: USDOT, FHWA, HDS-3 (1961).

lution to this equation for either V-shaped or triangular gutter configurations.

The cross slope of the depressed gutter section measured relative to the pavement cross slope is designated as  $S'_w$  and is equal to the depression divided by the width of the gutter:

$$S'_w = \frac{a}{W} \tag{21.47}$$

Hence, the actual cross slope  $S_w$  is:

$$S_w = S'_w + S_x \tag{21.48}$$

The relationship between these terms is illustrated in Figure 21.25.

Flow in composite gutter sections is computed by separating the flow into two prisms: (1) the flow in the pavement section and (2) the flow in the depressed gutter area. The flow in the pavement section is:

$$Q_s = \frac{0.56 d_2^{8/3} S^{1/2}}{n S_x} \tag{21.49}$$

where  $d_2 = (T - W)S_x$ , as shown in Figure 21.26,  $d_2$  is the depth of flow at the break point of the gutter-pavement interface and  $W$  is the width of the gutter.

The flow in the gutter section is given as:

$$Q_w = \frac{0.56(d_1^{8/3} - d_2^{8/3})S^{1/2}}{n S_w} \tag{21.50}$$

where  $d_1 = TS_x + a = TS_x + W(S_w - S_x)$ ,  $d_1$  is the depth of flow at the curb and  $S_w$  is the cross slope of the gutter. Note that Eq. (21.50) computes the flow in prism ABCD' by subtracting the flow in triangle CDD' from the flow in triangle ABCD of Figure 21.27.

Because of the numerous variables involved in the computations of flow in composite gutters, a relationship between the gutter flow ratio  $Q_w/Q (= E_0)$ ,  $W/T$ , and  $S_w/S_x$  has been developed. Figure 21.28 is a nomograph depicting this relationship, and assumes that the Manning's roughness coefficient for the gutter and pavement are the same.

In most urban areas, the allowable spread of the flow into the street is given in local design manuals. The general rule of thumb for urban-type secondary streets is to limit the spread to half the lane width as measured from the curb. Some primary road designs, such as Interstates, require that the spread not encroach at all on the travel lane. The spread is a function of the longitudinal and transverse slope of the pavement. Typically, drainage design of streets involves the sizing and spacing of the inlets based on limitations of the spread, with a given geometry of the street. However, to determine the spread  $T$ , given other parameters, involves a trial-and-error procedure since no direct solution for  $T$  is available from Eqs. (21.49) and (21.50).

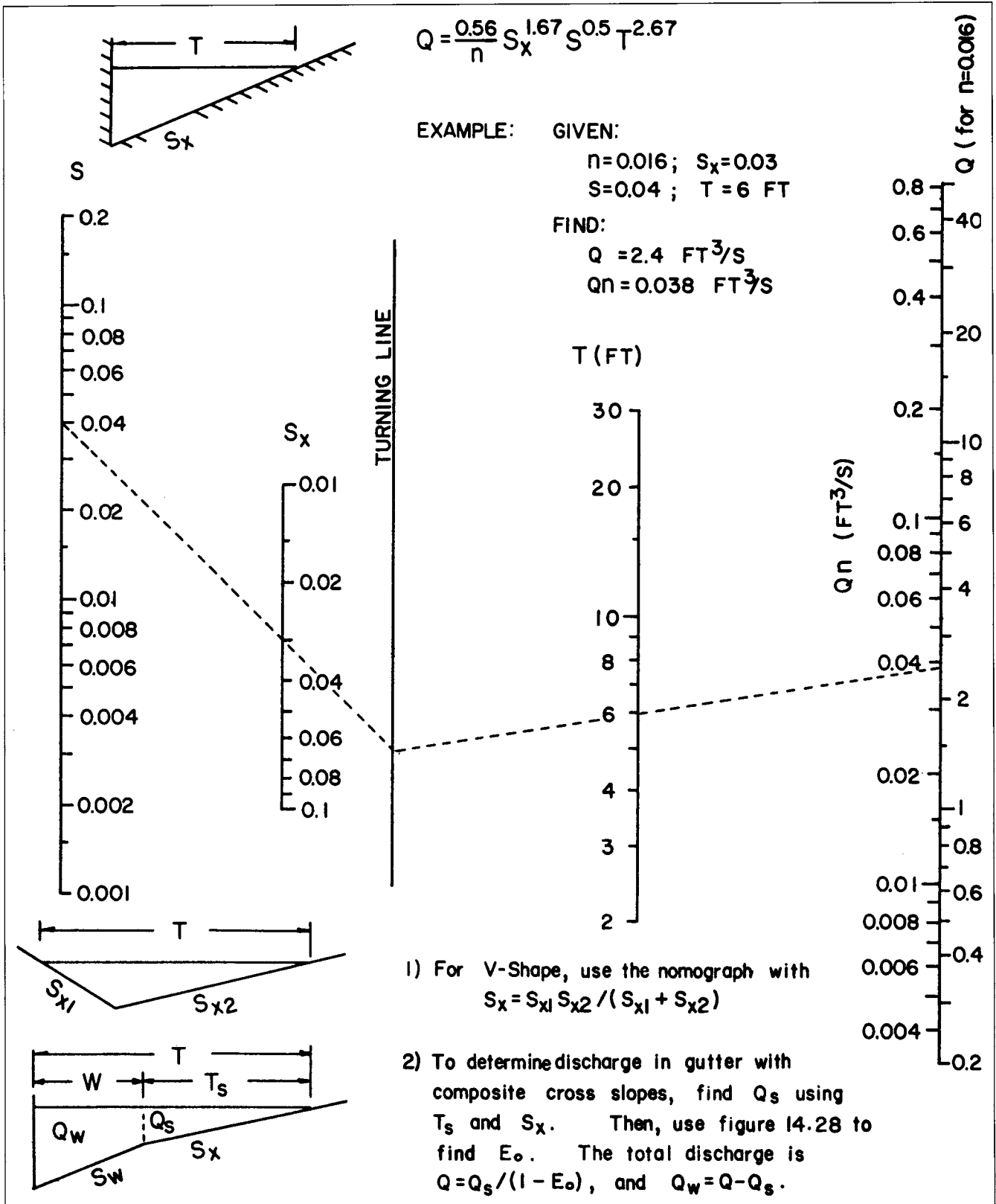


FIGURE 21.24 Nomograph for flow in triangular gutter sections. Reference: USDOT, FHWA, *Drainage of Highway Pavements*, HEC-12 (1984).

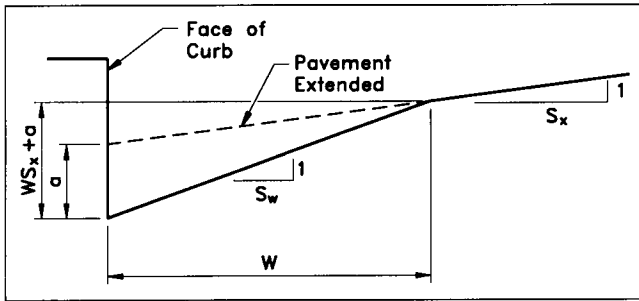


FIGURE 21.25 Composite gutter section nomenclature.

**EXAMPLE 5:** Find the spread of flow for a street section with a gutter width ( $W$ ) of 2 ft, curb height = 6 in.,  $S_w = 1$  in./ft, pavement cross slope of  $\frac{1}{4}$  in./ft ( $S_x = 0.0208$ ), longitudinal slope of 3%, and a design discharge of 3.5 cfs (assume  $n = .015$  for both pavement and gutter).

- Equations (21.49) and (21.50) can be used simultaneously to solve for the two unknowns  $T$  and  $Q_w$  (recall that  $Q = Q_w + Q_s$ ). Because a direct solution is difficult due to the exponential form of the equations, the nomograph of Figure 21.29 can aid the computations.
- Extend a horizontal line from 0.053 [ $(= 3.5)(0.015)$ ] on the  $Qn$  to intersect the  $S = 0.03$  line. From this point draw a vertical line to intersect the  $W = 2$  ft line. Continue horizontally and intersect the  $S_x = 0.02$  at the bottom left quadrant of the nomograph. Extend a vertical line to  $S_w/S_x = 0.083/0.020 = 4.1$ . A horizontal line drawn from this point to the  $T/W$  scale gives a value of 4.0. This translates to a spread of  $T = (2)(4.0) = 8.0$  ft.
- Check the depth at the curb:

$$d_1 = TS_x + W(S_w - S_x) \quad (21.51)$$

$$= (8.0)(0.02) + 2(0.083 - 0.02)$$

$$= 0.29 \text{ ft}$$

Since  $0.29 \text{ ft} < 0.5 \text{ ft}$ , the design flow will not overtop the curb.

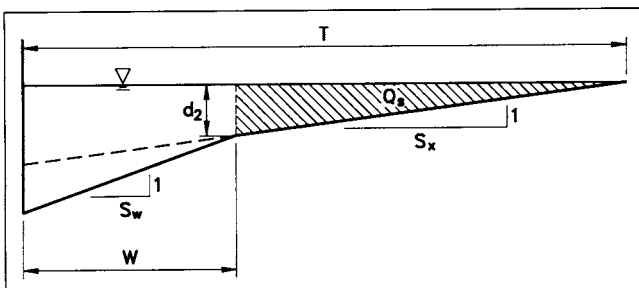


FIGURE 21.26 Flow in pavement section.

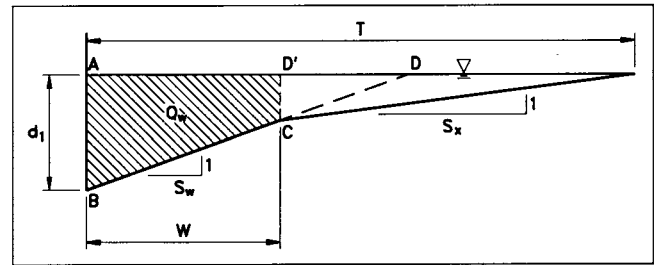


FIGURE 21.27 Flow in gutter section.

- Often, the values for  $S_x$ ,  $S_w$ , and  $S$  must be interpolated on the nomograph, and this may contribute to some error in reading the  $T/W$  scale. In these situations, the estimated value for  $T$  can be used as a beginning trial value to substitute into Eqs. (21.49) and (21.50). The value for  $T$  is adjusted to get  $Q_s + Q_w = Q$ .

**Inlets**

Inlets intercept surface runoff and transfer it to the pipe network. The two basic types of inlets used with pavement drainage are the curb opening inlet and the gutter (or grate) inlet. The curb opening inlet collects runoff through a vertical opening, along the face of curb. These inlets consist of a manhole structure with a throat and concrete slab fitting on the top. An opening in the slab allows access to the structure. A gutter (or grate) inlet lies flat in the pavement or gutter and allows runoff to fall directly down and into the system. Another type of inlet is a combination of the curb and grate inlet. Figure 21.30 shows these various inlets.

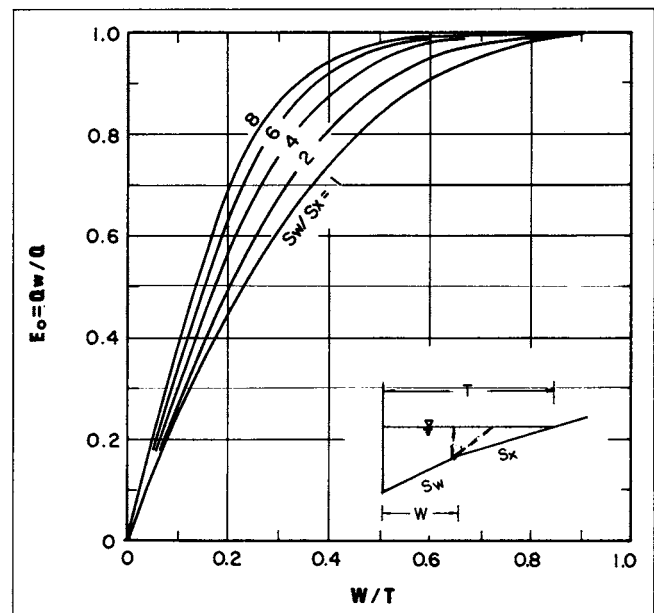
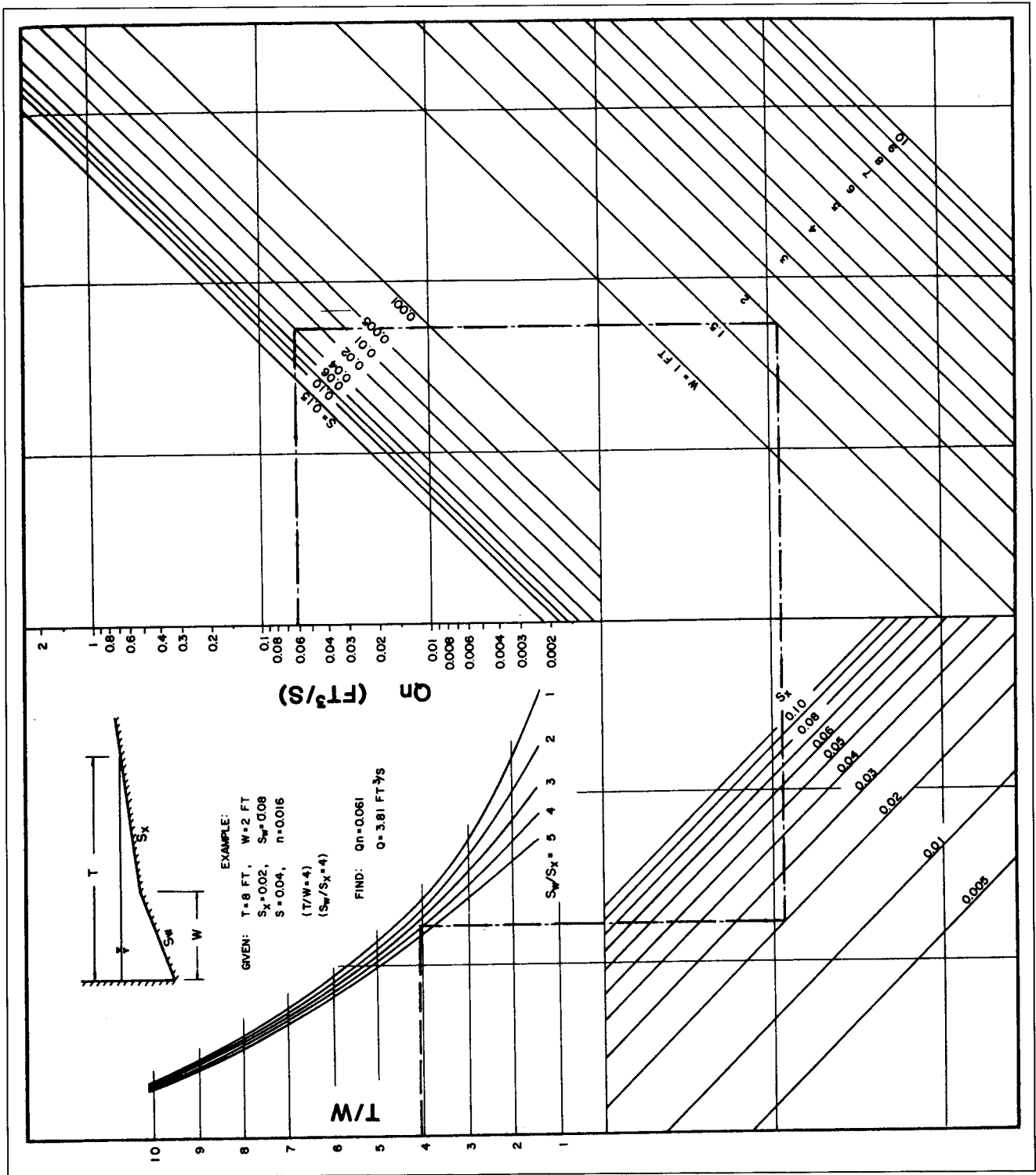


FIGURE 21.28 Ratio of frontal flow to total gutter flow. Reference: USDOT, FHWA, *Drainage of Highway Pavements*, HEC-12 (1984).



**FIGURE 21.29** Flow in composite gutter sections. Reference: USDOT, FHWA, *Drainage of Highway Pavements*, HEC-12 (1984).

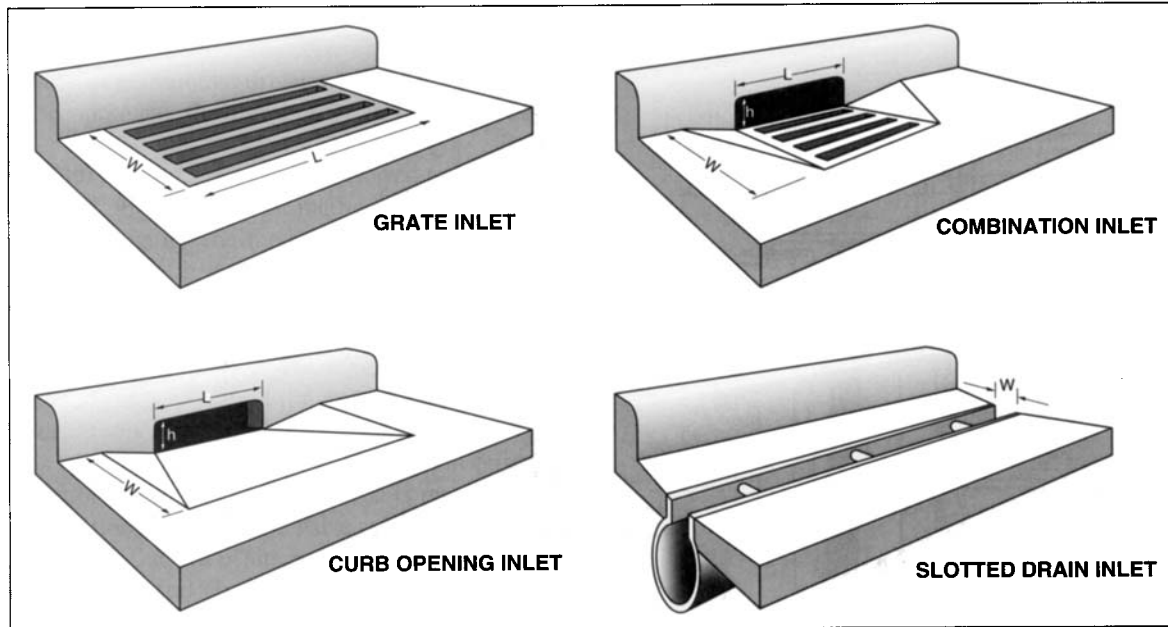


FIGURE 21.30 Various types of curb inlets and gutter inlets.

An inlet's flow capacity is the maximum amount of runoff that it can intercept as determined by the combined effects of longitudinal and transverse slopes of the road, roughness of the pavement, depth of flow at the inlet, and the configuration of the inlet opening. Flow through an inlet is either orifice flow or weir flow, depending on the flow depth at the opening. In either case, the capacity of the inlet is a function of the depth of flow forcing the water through the opening. For a given street geometry and flow conditions (i.e., flow depth), a given inlet has a specific interception capacity. When the flow exceeds the capacity of the inlet, a portion of the flow is not intercepted. The ratio of the intercepted flow to the total gutter flow,  $Q_i/Q_T$ , is the efficiency of the inlet. Any flow not intercepted by the inlet is termed *carryover* or *bypass* flow. This bypass flow must be added to the runoff computed for the next downstream inlet in the analysis. Obviously there can be no carryover from inlets located in sump (low-point) areas. Additionally, the designer should limit the carryover flows in pavement areas where the cross slope of the road changes direction. This is to avoid icing over the entire pavement width.

The physical characteristics of the street and inlet affect the flow velocity at the inlet. As gutter flow velocities increase, the depth at the curb decreases and the amount of by-pass flow increases. This is equivalent to a decrease in both interception capacity and efficiency for curb opening type inlets. The effective flow depth at a curb opening inlet is increased by depressing the gutter along the throat length. Most inlets have a 2- to 3-in. depression to improve interception capacity. Everything else being equal, increasing depth at the curb (by increasing pavement cross slope) increases both the interception capacity and efficiency. Al-

though grate inlets may be more effective on higher longitudinal slopes, they present more of an obstruction to pedestrians and bicycle traffic and are susceptible to clogging.

The type of inlet used is governed by the standards of the local jurisdiction. The department of transportation and the local department of public works can be very particular about the types of inlets allowed in right-of-ways.

### Inlet Capacity

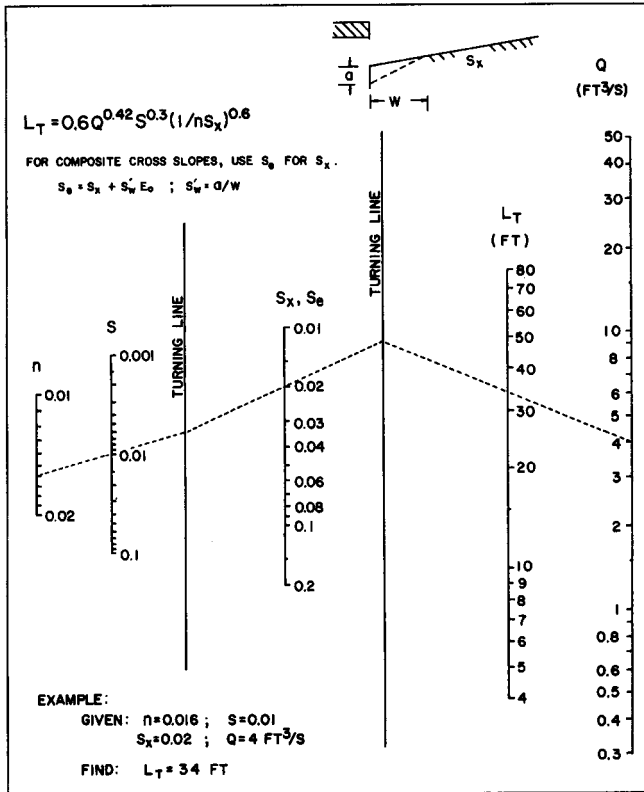
Typically, the design standards for a locality include capacity charts, nomographs, or equations for the inlets accepted by the review agencies. Manufacturers of inlets often perform their own tests on grate inlets and provide the capacity information upon request.

**Curb-Opening Inlets on Grade.** For curb-opening-type inlets, the length of curb opening necessary for 100% efficiency in a street section with undepressed gutter, and constant cross slope is

$$L_T = 0.6 Q^{0.42} S^{0.3} \left( \frac{1}{nS_x} \right)^{0.6} \quad (21.52)$$

where  $L_T$  = curb opening length required for 100% interception (ft),  $Q$  = gutter flow (cfs),  $S$  = longitudinal street slope (ft/ft),  $S_x$  = cross slope (ft/ft), and  $n$  = Manning's  $n$  coefficient for the pavement. The nomograph of Figure 21.31 can be used in lieu of the equation.

For a depressed curb-opening inlet the length of opening is the same as Eq. (21.52), with the  $S_x$  term replaced by the equivalent cross slope term,  $S_e$ . This term is used to account for the gutter depression. The equivalent cross slope is given as



**FIGURE 21.31** Curb opening and slotted-drain inlet length for total interception. Reference: USDOT, FHWA, *Drainage of Highway Pavements*, HEC-12 (1984).

$$S_e = S_x + S'_w E_0 \tag{21.53}$$

The efficiency  $E$  of a curb opening inlet is given as

$$E = \frac{Q_i}{Q} = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \tag{21.54}$$

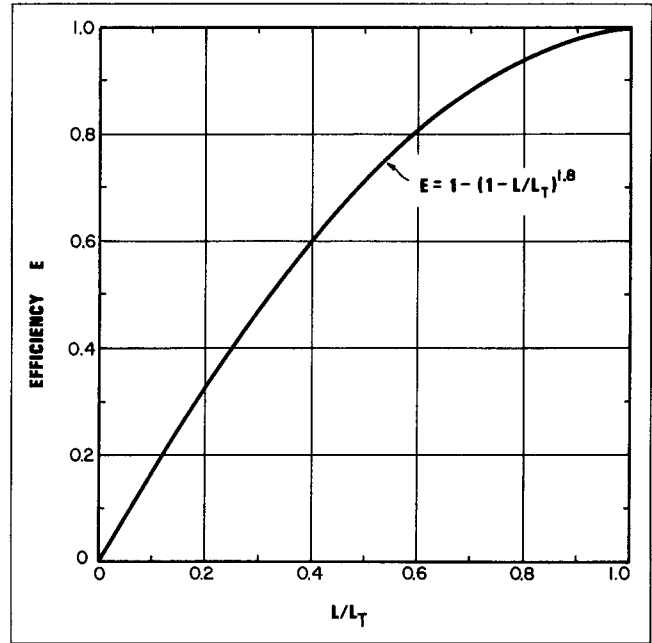
where  $L$  is the actual length of the curb opening. This is shown graphically in Figure 21.32.

**EXAMPLE 6:** Determine the throat length required for 100% interception for the values in the preceding example

1. Since this is a composite gutter section, the equivalent cross slope,  $S_e$ , is used as calculated in 21.53. Given  $S_w/S_x = 4.1$  and  $W/T = 0.25$ , Figure 21.28 reads a value of  $E_0 \approx 0.70$ .  $S'_w$  is still required to solve Equation 21.53.

$$S'_w = \frac{a}{W} = \frac{0.125}{2} = 0.063 \tag{21.55}$$

$$\begin{aligned} \text{where } a &= w(S_w - S_x) & (21.56) \\ &= 2(0.0833 - 0.0208) \\ &= 0.125 \end{aligned}$$



**FIGURE 21.32** Curb opening and slotted-drain inlet interception efficiency. Reference: USDOT, FHWA, *Drainage of Highway Pavements*, HEC-12 (1984).

2. The equivalent cross slope is:

$$\begin{aligned} S_e &= 0.0208 + 0.063(0.70) & (21.57) \\ &= 0.065 \end{aligned}$$

3. Substituting the values into Eq. (21.52):

$$\begin{aligned} L_T &= 0.6(3.5)^{0.42}(0.03)^{0.3} \left(\frac{1}{(0.015)(0.065)}\right)^{0.6} & (21.58) \\ &= 22.7 \text{ ft} \end{aligned}$$

Since most throat lengths are available in 2-ft increments, a 24-ft throat length would be specified for this situation.

Most manufacturers do not make inlets with throats longer than 20 feet. Therefore the designer would have to consider the overflow from this location to be included at the next downstream location. Or, if 100% interception is required, moving the inlet upstream or adding another inlet upstream to reduce the flow to this structure should be considered.

Note that once  $S_e$  was determined, the required throat length could have been found using Figure 21.31. Also, the answer does not include the effects of additional inlet or local depression which is standard for some state departments of transportation.

**Curb-Opening Inlets In Sag Locations** These inlets act as weirs for gutter flow depths less than or equal to the height of the opening. At depths greater than 1.4 times the

height of the opening, the inlet operates as an orifice. Between these depths the flow is in transition. The interception capacity for a depressed inlet as given by the weir equation is

$$Q_i = C_w(L + 1.8W)d^{1.5} \tag{21.59}$$

where  $C_w$  is the weir coefficient ( $\approx 3.0$ ),  $L$  is the length of the opening (e.g., throat length),  $W$  is the lateral width of the depression, and  $d$  is the depth of flow at the curb as measured from the normal cross slope gutter flow line. For an undepressed inlet,  $W$  is equal to 0.

When flow in the gutter exceeds the height of the curb opening inlet by a factor of 1.4, the opening acts as an orifice with the capacity given as

$$Q_i = C_o h L \sqrt{2gd_o} = C_o A \sqrt{2g\left(d_i - \frac{h}{2}\right)} \tag{21.60}$$

where  $C_o$  is the orifice coefficient (usually 0.67),  $h$  is the height of the opening (ft),  $L$  is the length of the opening (ft),  $d_o$  is the effective head on the center of the orifice throat,  $A$  is the area of the opening (ft<sup>2</sup>), and  $d_i$  is the depth at the lip of curb opening (ft). Typical opening configurations for curb inlets are shown in Figure 21.33. The computed capacity for the inlet depends on proper interpretation of the “height” of the opening, since the height depends on the configuration of the throat opening.

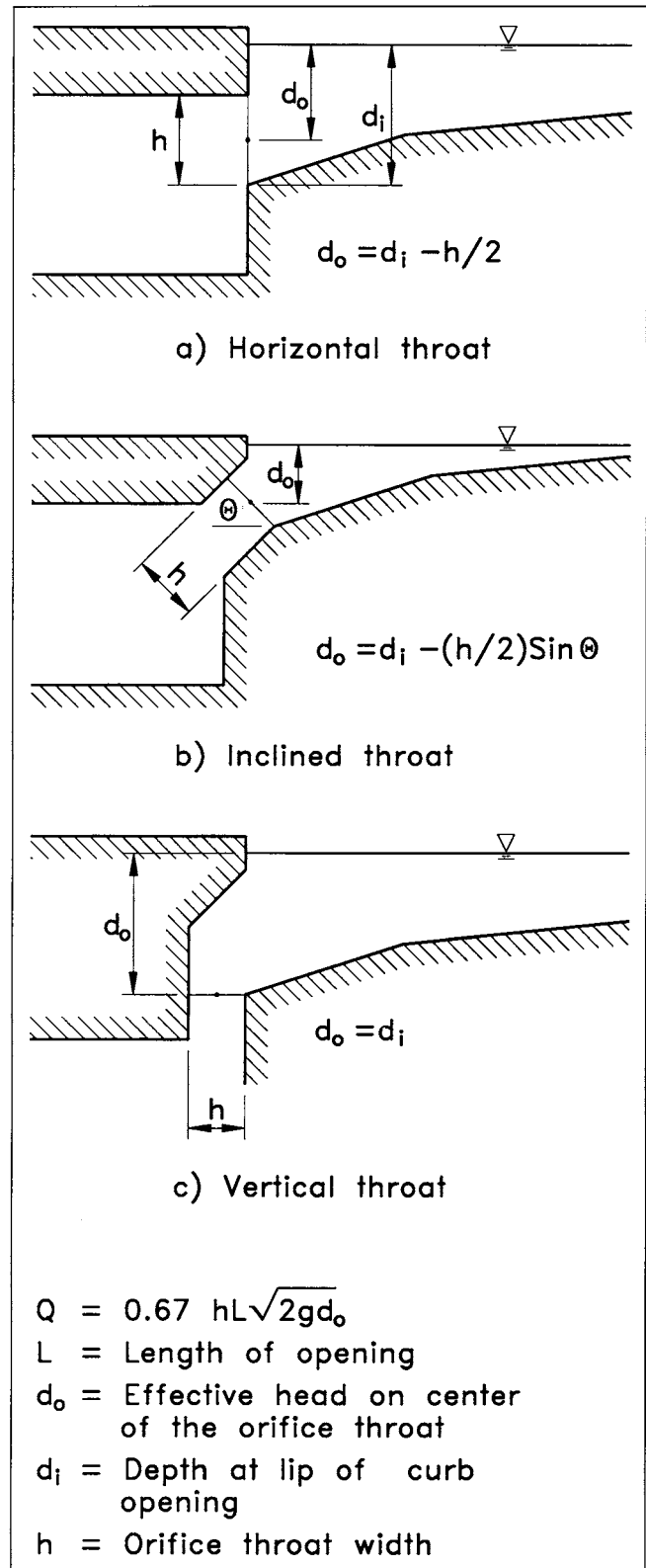
**Grate Inlets.** The capacity of a grate inlet depends on pavement geometry, amount of flow and the configuration of the grate. Flow through the grate is affected by the size and spacing of the bars as well as their shape and orientation with respect to flow direction.

Any jurisdiction’s design standards identify preferred types of grates. Each type of grate has its own hydraulic capacity chart. The Federal Highway Administration (FHWA) identifies several types of grates and their hydraulic characteristics in HEC-12. For those occasions where a standard grate will not suffice, the reader is referred to manufacturers of grates for dimensions and hydraulic characteristics.

**Grate Inlets in Sag Locations.** Flow through grates located at the low point of a sag vertical curve operate as weir flow for relatively shallow depths. As the depth increases, the flow through the inlet transitions to orifice flow. There is no definitive depth where weir flow stops and orifice flow begins. The range of depth for which the transition between weir and orifice flow occurs depends on geometric and hydraulic characteristics of the grate. The weir equation for grate inlets is

$$Q = C_w P d^{3/2} \tag{21.61}$$

where  $C_w$  is a weir coefficient dependent on geometry of the opening ( $C_w \approx 3.0$ ),  $P$  is the effective length of opening where weir flow occurs (grates located along the face of curb have only three sides where discharge passes through as weir flow), and  $d$  is the depth of flow.



**FIGURE 21.33** Throat configuration of curb opening inlets. Reference: USDOT, FHWA, *Drainage of Highway Pavements*, HEC-12 (1984).



The orifice equation for grate inlets is

$$Q = C_d A \sqrt{2gd} \quad (21.62)$$

where  $C_d$  is the coefficient of discharge ( $\approx 0.67$ ),  $A$  is the clear opening area of the grate ( $\text{ft}^2$ ),  $g = 32.17 \text{ ft/sec}^2$  and  $d$  is the depth of flow. Orifice discharge through the grate depends on the clear open area, which does not include space occupied by bars, vanes and other obstructions. Since sag locations are highly susceptible to clogging by debris and trash, many localities require an allowance of 50% for clogging.

## CULVERTS

A culvert is a relatively short length of conduit, typically less than 250 ft long, used to transport water through (or under) an embankment. A culvert, which acts as an enclosed channel through the embankment, serves as a continuation of the open stream. However, flow through culverts depends on entrance geometry and depth of flow at the downstream end. Consequently, flow computations for culverts are more complex than the open channel flow analysis associated with pipes and ditches. Culverts through roadway and railway embankments are designed to pass the design discharge without overtopping the embankment or causing extensive ponding, or inundation at the upstream end. Local requirements may allow nominal depths over the embankments for lesser frequency storm (greater recurrence interval) events.

### Components

Major components of a culvert include the barrel, end treatments such as headwalls, endwalls, and wingwalls, outlet protection, inlet improvements, and debris control structures. Except for the barrel, these components are used as the specific situation warrants.

*Barrels* are available in various sizes, shapes, and materials. Figure 21.34 shows the commonly used culvert shapes as well as applications of the various shapes. Selection of shape depends on construction limitations, embankment height, environmental issues, hydraulic performance, and cost. The most commonly used culvert materials are corrugated steel, corrugated aluminum and precast or cast-in-place concrete. Factors such as corrosion, abrasion, and structural strength determine the selection of material. In cases where the culvert is located in a highly visible area, the selection of shape and material may be based on aesthetics, as well as the functional aspects.

*End treatments*, such as headwalls and wingwalls, protect the embankment from erosion, serve as retaining walls to stabilize the bank and add weight to counter any buoyancy effects. Ideally, the centerline of the culvert should follow the alignment and grade of the natural channel. In many cases this cannot be done, and skewing headwalls and wingwalls helps transition the natural stream alignment to

the culvert alignment. Figure 21.35 shows four types of inlet entrances.

*Debris barriers* are sometimes constructed on the upstream end to prevent material from entering and clogging the culvert. The barriers are placed far enough away from the entrance so that accumulated debris does not clog the entrance.

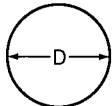
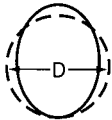
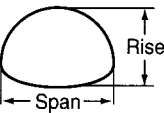
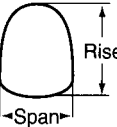
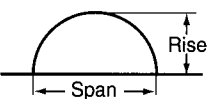
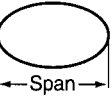

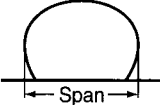
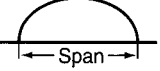
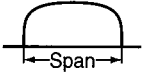
At the inlet and outlet ends of the culvert, endwalls and wingwalls serve as retaining walls and erosion protection for the embankment and help to inhibit piping along the outside surface of the culvert. Downstream wingwalls provide a smooth transition between the culvert and the natural stream banks.

All culverts that cross underneath roadway embankments inherently create potential stability problems for the embankments. The same can be said of storm sewer pipe installations. This is a function of the construction of the culvert system. Compaction around a pipe, whether it is circular or rectangular, is more difficult than standard compaction of fill slopes. The contractor must take care in compacting around culvert systems, usually requiring several "lifts" to compact the surrounding fill material to specification. The designer and contractor should always consider that storm runoff will seep into roadway embankments, or that groundwater tables will rise during flood events. If the compaction is inadequate, water can run along the voids in the soil of the "uncompacted" portion of the culvert. When water continues to flow along a pipe or culvert, the uncompacted material slowly washes away from the surrounding pipe, and failure of the embankment can occur. This is one of the most common causes of embankment failures at culvert installations and roadway pavement failures at storm sewer installations. This type of failure is referred to as piping failure.

Culverts and storm sewers are typically constructed with a stone base to provide support and flexibility. For most installations, this stone base is porous and allows any water that is inside the embankment to travel downstream, along the path of the culvert. Typically, this aggregate base aids in reducing piping failures, in that it concentrates the water flowing through the embankment. However, should there be excessive ponding of water behind an embankment, the standard bedding provided at culvert installations will not be adequate to pass the seepage flow safely along the conduit, and piping failures could occur. The designer should consult with a geotechnical engineer to determine appropriate countermeasures, such as concrete cradles, use of impervious materials, or embankment draining devices such as toe-drains to minimize the potential for piping failures.

### Culvert Hydraulics

In open-channel flow, either of three turbulent flow regimes exists: subcritical flow, supercritical flow or critical flow. The discharge energy, relative to the channel bed, is referred to as the specific energy and is mathematically defined as

Shape	Range of Sizes	Common Uses
Round 	6" – 26'	Culverts, subdrains, sewers, service tunnels, etc. All plates same radius. For medium and high fills (or trenches).
Vertically-elongated (ellipse) 5% is common 	4' – 21' nominal; before elongating	Culverts, sewers, service tunnels, recovery tunnels. Plates of varying radii; shop fabrication. For appearance and where backfill compaction is only moderate.
Pipe-arch 	Span x Rise 18" x 11" to 20' 7" x 13' 2"	Where headroom is limited. Has hydraulic advantages at low flows. Corner plate radius, 18 inches or 31 inches for structural plate.
Underpass* 	Span x Rise 5' 8" x 5' 9" to 20' 4" x 17' 9"	For pedestrians, livestock or vehicles (structural plate).
Arch 	Span x Rise 6' x 1' 9 1/2" to 25' x 12' 6"	For low clearance large waterway opening, and aesthetics (structural plate).
Horizontal Ellipse 	Span 20' – 40'	Culverts, grade separations, storm sewers, tunnels.
Pear 	Span 25' – 30'	Grade separations, culverts, storm sewers, tunnels.
High Profile Arch 	Span 20' – 45'	Culverts, grade separations, storm sewers, tunnels, Ammo ammunition magazines, earth covered storage.
Low Profile Arch 	Span 20' – 50'	Low-Wide waterway enclosures, culverts, storm sewers.
Box Culverts 	Span 3' – 20'	Low-Wide waterway enclosures, culverts, storm sewers.
Specials	Various	For lining old structures or other special purposes. Special fabrication.

\* For equal area or clearance, the round shape is generally more economical and simpler to assemble.

**FIGURE 21.34** Shapes and uses of corrugated conduits. Reprinted with permission from American Iron and Steel Institute, *Handbook of Steel Drainage and Highway Construction Products*, 3rd ed, American Iron and Steel Institute, Washington, D.C. 1983.

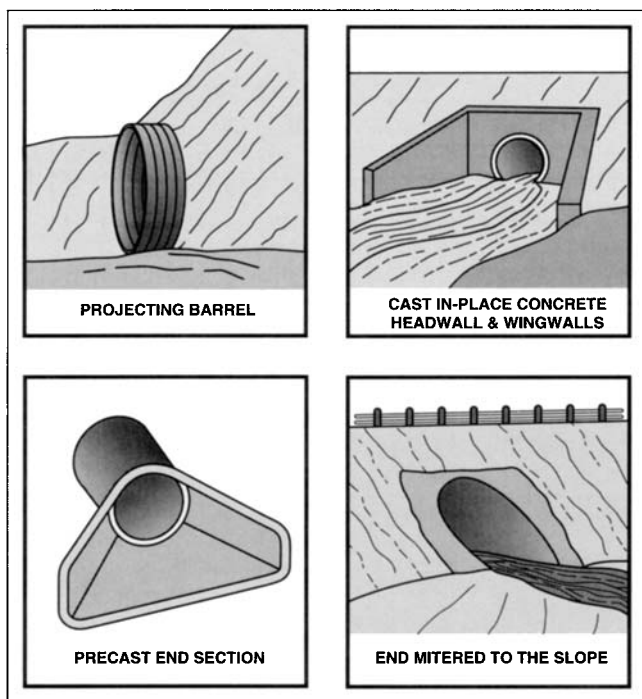


FIGURE 21.35 Four standard end inlet treatments.

$$E = y + \frac{V^2}{2g} \quad (21.63)$$

where  $y$  is the depth of flow,  $V$  is the average cross-sectional velocity, and  $E$  is the specific energy.

For one particular combination of depth and velocity, for a specified discharge, the specific energy is a minimum. This particular discharge is the critical flow. Alternatively, critical flow is the condition of flow when, for a given energy content of the water, the discharge is a maximum. For the same discharge, flows above this minimum energy may exist as either high depth–low velocity or low depth–high velocity. In the former case, flow is subcritical, the latter is supercritical flow. The parameter that distinguishes flow regimes, the Froude number, is a dimensionless number. The Froude number, as seen in Eq. (21.64), is the ratio of inertial forces to gravitational forces of the flow:

$$F_R = \frac{V}{\sqrt{gd_h}} \quad (21.64)$$

In this equation,  $d_h$  is the hydraulic depth (the cross-sectional area of flow divided by the width of the channel at the water surface),  $V$  is the velocity, and  $g$  is gravitational acceleration. At critical flow,  $F_R = 1$ ; for subcritical flow,  $F_R < 1$ ; and at supercritical flow,  $F_R > 1$ . The location in the channel section where flow changes from subcritical to supercritical flow (i.e.,  $F_R = 1$ ) is defined as a control section. At a control section, there exists a unique relation between depth and discharge, given as

$$Q = \sqrt{g \frac{A^3}{B}} \quad (21.65)$$

where  $A$  is the cross-sectional area of flow and  $B$  is the width of the channel at the free surface of the water. Hydraulically, a control section restricts the transmission of the effect of changes in flow conditions, either in the upstream or downstream direction, depending on the state of flow.

The amount of water entering a culvert is determined by the location of the control section. If the control section is at the inlet (i.e., inlet control), the amount of flow into the culvert is restricted by entrance conditions and the flow into the culvert is less than what the culvert might actually carry. If the control section is at the downstream end (i.e., outlet control), the amount of flow through the barrel is controlled by the combination of conditions on the downstream side of the culvert, inlet configuration, and hydraulic properties of the barrel itself.

### Inlet Control

When the control section is near the inlet, only the headwater depth and inlet configuration determines the amount of water entering the culvert. In most cases, the amount of water entering the culvert is less than what the barrel is capable of carrying. Conceivably, for a given inlet configuration, the culvert can flow full if the headwater depth is high enough to force enough water through the inlet. The headwater depth necessary to achieve full flow in the barrel of the culvert is very large and is usually not achievable given practical site constraints. Figure 21.36 illustrates the various types of inlet control.

For flows that do not submerge the inlet, as in A and B of Figure 21.36, the flow into the culvert is modeled as weir flow. For submerged conditions, the flow into the culvert is modeled as orifice flow. Equations (21.66), (21.67), and (21.68) were determined from experimental data on culverts by the National Bureau of Standards (NBS). Equations (21.66) and (21.67) are for unsubmerged conditions and Eq. (21.68) is for submerged (orifice) conditions. (Equation (21.67) is another form of Eq. (21.66).)

$$\frac{HW_i}{D} = \frac{H_c}{D} + K \left[ \frac{Q}{AD^{0.5}} \right]^M + C_s \quad (21.66)$$

(for  $Q/AD^{0.5} \leq 3.5$ )

$$\frac{HW_i}{D} = K \left[ \frac{Q}{AD^{0.5}} \right]^M \quad (21.67)$$

(for  $Q/AD^{0.5} \leq 3.5$ )

$$\frac{HW_i}{D} = c \left[ \frac{Q}{AD^{0.5}} \right]^2 + Y + C_s \quad (21.68)$$

(for  $Q/AD^{0.5} \geq 4.0$ )

where  $HW_i$  = headwater depth above inlet control section invert (ft),  $D$  = interior height of culvert barrel (ft),  $H_c$  =

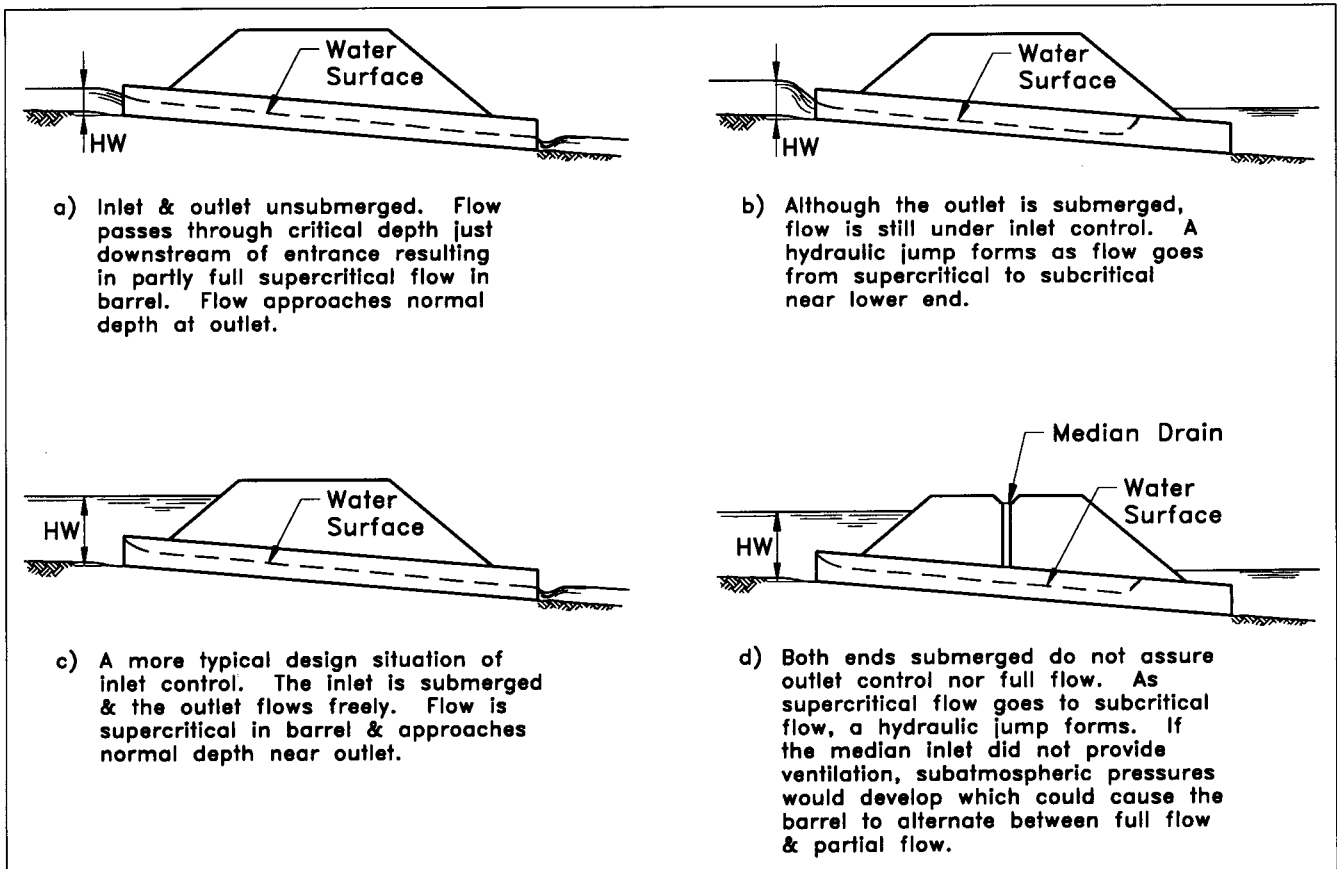


FIGURE 21.36 Types of inlet control.

specific head at critical depth ( $d_c + V^2/2g$ ) (ft),  $Q$  = discharge (cfs),  $A$  = full cross-sectional area of culvert barrel ( $\text{ft}^2$ ),  $C_s$  = slope correction factor (= 0.75 for mitered inlets and  $C_s = -0.5S$  for all other inlets where  $S$  = culvert barrel slope (ft/ft)), and  $K$ ,  $M$ ,  $c$ ,  $Y$  are constants from Table 21.17.

The Federal Highway Administration has developed numerous nomographs for various culvert shapes for inlet control. Figures 21.37, 21.38, and 21.39 illustrate several of these nomographs. Others are available in the *Hydraulic Design of Highway Culverts* (FHWA, 1985), available through National Technical Information Service (NTIS), Springfield, Virginia.

### Outlet Control

When the control section is at the downstream side of the culvert, flow through the culvert is either subcritical or pressure flow. Additionally, the amount of flow through the culvert is governed by hydraulic characteristics of both the culvert and tailwater conditions. In culverts under outlet control, the barrel is not capable of carrying all of the water that passes through the inlet. Figure 21.40 shows the various types of outlet control conditions.

Flow through the culvert is a balance between the energy available to pass the water through and the energy con-

sumed by friction and minor losses. The energy balance is given by

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + \Sigma H_L \quad (21.69)$$

where  $HW_o$  = headwater depth above the outlet invert,  $V_u$  = approach velocity of the water at the inlet,  $TW$  = tailwater depth above the outlet invert,  $V_d$  = discharge velocity of the water at the outlet, and  $\Sigma H_L$  represents the summation of all energy losses through the barrel.

As with pipe flow, energy losses for culverts are attributed to barrel roughness (friction), minor losses at the inlet and outlet, as well as bends and other appurtenances. Similarly, minor losses are computed as a fraction of the velocity head (kinetic energy) as given in the equation

$$h = k \left[ \frac{V^2}{2g} \right] \quad (21.70)$$

where the factor  $k$  is empirically determined and depends on geometry and hydraulic characteristics. Table 21.18 provides values of  $k$  for various entrance conditions. In most cases the  $k$  value for exit loss is 1.0.

**TABLE 21.17 Constants for Inlet Control Design Equations**

CHART No.	SHAPE AND MATERIAL	MONOGRAPH SCALE	INLET EDGE DESCRIPTION	EQUATION FORM	UNSUBMERGED			SUBMERGED		
					K	M	C	Y	C	Y
1	Circular concrete	1	Square edge w/headwall	1	0.0098	2.0	0.0398	0.67	(56)	(57)
		2	Groove end w/head wall		.0078	2.0	.0292	.74	(56)	(57)
		3	Groove end projecting		.0045	2.0	.0317	.69	(56)	(57)
2	Circular CMP	1	Headwall	1	.0078	2.0	0.379	.69	(56)	(57)
		2	Mitered to slope		.0210	1.33	0.463	.75	(57)	
		3	Projecting		.0340	1.50	0.553	.54	(57)	
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74	(57)	
		B	Beveled ring, 33.7° bevels		.0018	2.50	.0243	.83	(57)	
8	Rectangular box	1	30-75% wingwall flares	—	.026	1.0	.0385	.81	(56)	
		2	90° and 15° wingwall flares	1	.061	.75	.0400	.80	(56)	
		3	0° wingwall flares		.061	.75	.0423	.82	(8)	
9	Rectangular box	1	45° wingwall flare $d = .043D$	2	.510	.667	.0309	.80	(8)	
		2	18-33.7° wingwall flare $d = .083D$		.486	.667	.0249	.83	(8)	
10	Rectangular box	1	90° headwall w/3/4° chamfers	2	.515	.667	.0375	.79	(8)	
		2	90° headwall w/45° bevels		.495	.667	.0314	.82	(8)	
		3	90° headwall w/33.7° bevels		.486	.667	.0252	.865	(8)	
11	Rectangular box	1	3/4° chamfers; 45° skewed headwall	2	.522	.667	.0402	.73	(8)	
		2	3/4° chamfers; 30° skewed headwall		.533	.667	.0425	.705	(8)	
		3	3/4° chamfers; 15° skewed headwall		.545	.667	.04505	.68	(8)	
		4	45° bevels; 10-45° skewed headwall		.498	.667	.0327	.75	(8)	
12	Rectangular box	1	45° nonoffset wingwall flares	2	.497	.667	.0339	.803	(8)	
		2	18.4° nonoffset wingwall flares		.493	.667	.0361	.806	(8)	
		3	18.4° nonoffset wingwall flares		.495	.667	.0386	.71	(8)	
13	Rectangular box	1	45° wingwall flares, offset	2	.497	.667	.0302	.835	(8)	
		2	33.7° wingwall flares, offset		.495	.667	.0252	.881	(8)	
		3	18.4° wingwall flares, offset		.493	.667	.0227	.887	(8)	
16-19	CM boxes	1	90° headwall	1	.0083	2.0	.0379	.69	(57)	
		2	Thick wall projecting		.0145	1.75	.0419	.64	(57)	
		3	Thin wall projecting		.0340	1.5	.0496	.57	(57)	

29	Horizontal ellipse concrete	1 2 3	Square edge with headwall Groove end with headwall Groove end projecting	1	0.0100 .0018 .0045	2.0 2.5 2.0	0.0398 .0292 .0317	0.67 .74 .69	(57) (57) (57)
30	Vertical ellipse concrete	1 2 3	Square edge with headwall Groove end with headwall Groove end projecting	1	.0100 .0018 .0095	2.0 2.5 2.0	.0398 .0292 .0317	.67 .74 .69	(57) (57) (57)
34	Pipe arch 18" corner Radius CM	1 2 3	90° headwall Mitered to slope Projecting	1	.0083 .0300 .0340	2.0 1.0 1.5	.0496 .0463 .0496	.57 .75 .53	(57) (57) (57)
35	Pipe arch 18" corner Radius CM	1 2 3	Projecting No bevels 33.7° bevels	1	.0296 .0087 .0030	1.5 2.0 2.0	.0487 .0361 .0264	.55 .66 .75	(56) (56) (56)
36	Pipe arch 31" corner Radius CM	1	Projecting	1	.0296	1.5	.0487	.55	(56)
40-42	Arch CM	1 2 3	90° headwall Mitered to slope Thin wall projecting	1	.0083 .0300 .0340	2.0 2.0 1.5	.0379 .0463 .0496	.69 .75 .57	(57) (57) (57)
54	Circular	1 2	Smooth tapered inlet throat Rough tapered inlet throat	2	.534 .519	.555 .64	.0196 .0289	.89 .90	(3) (3)
55	Elliptical Inlet face	1 2 3	Tapered inlet, beveled edges Tapered inlet, square edges Tapered inlet, thin edge projecting	2	.536 5035 .547	.622 .719 .80	.0368 .0478 .0598	.83 .80 .75	(3) (3) (3)
56	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	(3)
57	Rectangular concrete	1 2	Side tapered, less favorable edges Side tapered, more favorable edges	2	.56 .56	.667 .667	.0466 .0378	.85 .87	(3) (3)
58	Rectangular concrete	1	Slope tapered, less favorable edges Slope tapered, more favorable edges	2	.50 .50	.667 .667	.0466 .0378	.65 .71	(3) (3)

Reference: FHWA, *Hydraulic Design of Highway Culverts*, HDS-5, 1985.

Friction loss  $H_f$  in the barrel of the culvert depends on culvert material and geometry. Using the Manning equation, the friction loss is given as

$$H_f = \left[ \frac{29n^2L}{R^{1.33}} \right] \frac{V^2}{2g} \quad (21.71)$$

where  $n$  = Manning's roughness coefficient,  $L$  = barrel length, and  $V$  = average discharge velocity in the barrel. The variable  $R$  is the hydraulic radius =  $A/WP$ , where  $A$  = cross-sectional area of the culvert and  $WP$  = wetted perimeter.

The head loss through a straight culvert of uniform cross section is

$$H = \left( 1 + k_e + \frac{29n^2L}{R^{4/3}} \right) \frac{V^2}{2g} \quad (21.72)$$

where  $k_e$  is the entrance loss coefficient. For culverts with bends, sudden expansions, and other features that create energy losses, additional terms are added to Eq. (21.72).

### Procedure for Culvert Design

1. Determine the culvert alignment. The most desirable alignment follows the natural stream bed. However, factors such as economics, environmental issues, maintenance considerations, and other site constraints contribute to the alignment selection. Setting the alignment also determines the length and slope of the culvert.
2. Select the design storm, which is frequently dictated by state or local criteria. For culverts crossing roadways, the type of roadway affects the design storm. For secondary roads, the recurrence interval may range from 10- to 25-year storm. On major highways and interstates, the recurrence interval may be 50 years or greater. Other criteria affecting design may be the freeboard distance or depth of flow allowed on the roadway for a storm other than the design storm.
3. Utilizing land use maps, topographic maps, and whatever hydrologic technique is best suited for the area (e.g., rational method, TR-55 methodology, regional equations), determine the design discharge for the culvert.
4. Select the culvert material and shape that best conforms to the constraints of the site. Certain materials and shapes may not be readily available from suppliers. For example, low embankment height may warrant the use of elliptical pipe, or high embankments may warrant structurally enhanced material. Aesthetics may also affect the selection of material and shape.
5. Size the culvert either using the equations for inlet and outlet control or the culvert nomographs, which-

ever gives the greatest controlling elevation. The FHWA culvert design form of Figure 21.41 provides a guide to the selection of a culvert. For the design discharge, the headwater depth is computed assuming inlet control and outlet control. The type of control that produces the highest headwater depth is the governing control for the design.

6. After sizing the culvert for the design discharge, check to see that other requirements as dictated by applicable design standards are met. Such requirements might include freeboard height or roadway overtopping for lower recurrence interval storms, effects and impacts of upstream ponding, the need for erosion control for excessive outlet velocities, and structural stability.

### Culvert Design Example

Size a concrete ( $n = 0.013$ ,  $k_e = 0.5$ ) box culvert for the following constraints: 25-yr design discharge = 461 cfs with tailwater depth = 2.6 ft; 100-yr design discharge = 690 cfs with tailwater depth = 4.3 ft. At least 1.5 ft of freeboard is required for the 25-year design discharge, and the maximum allowable flow depth over the embankment for the 100-year design discharge is 12 in. The upstream invert elevation = 314.2 ft, the downstream invert elevation 313.8 ft, and the top of the roadway elevation = 324.5 ft. The culvert is 140 ft long. Assume flared wingwalls with angle of flare between 30 and 75° and square-edge conditions. The minimum allowable cover over the culvert is 3.0 ft. From the given data, the constraints for the 25-year discharge are

$$\begin{aligned} \text{Maximum headwater depth} & \quad (21.73) \\ & = 324.5 - 1.5 - 314.2 \\ & = 8.8 \text{ ft} \\ \text{Maximum culvert height} & = 324.5 - 3 - 314.2 \\ & = 7.3 \text{ ft} \\ \text{Slope} & = \frac{314.2 - 313.8}{140} \\ & = 0.003 \text{ ft/ft} \end{aligned}$$

Sizing a culvert is a trial and error procedure. Estimating a "first try" size becomes more intuitive with experience. In this example, select a trial  $HW/D$  based on the maximum headwater depth and culvert height,  $HW/D = 8.8/7.3 = 1.2$  to obtain a first approximation. (Note: When finalizing the culvert design, the engineer should consider the physical characteristics of an installation and determine that the culvert will fit safely underneath the roadway pavement box. This should be done by drawing a profile of the culvert with relation to the proposed ground above the culvert. The thickness of the culvert should also be considered when profiling the system.) For inlet control conditions, use the

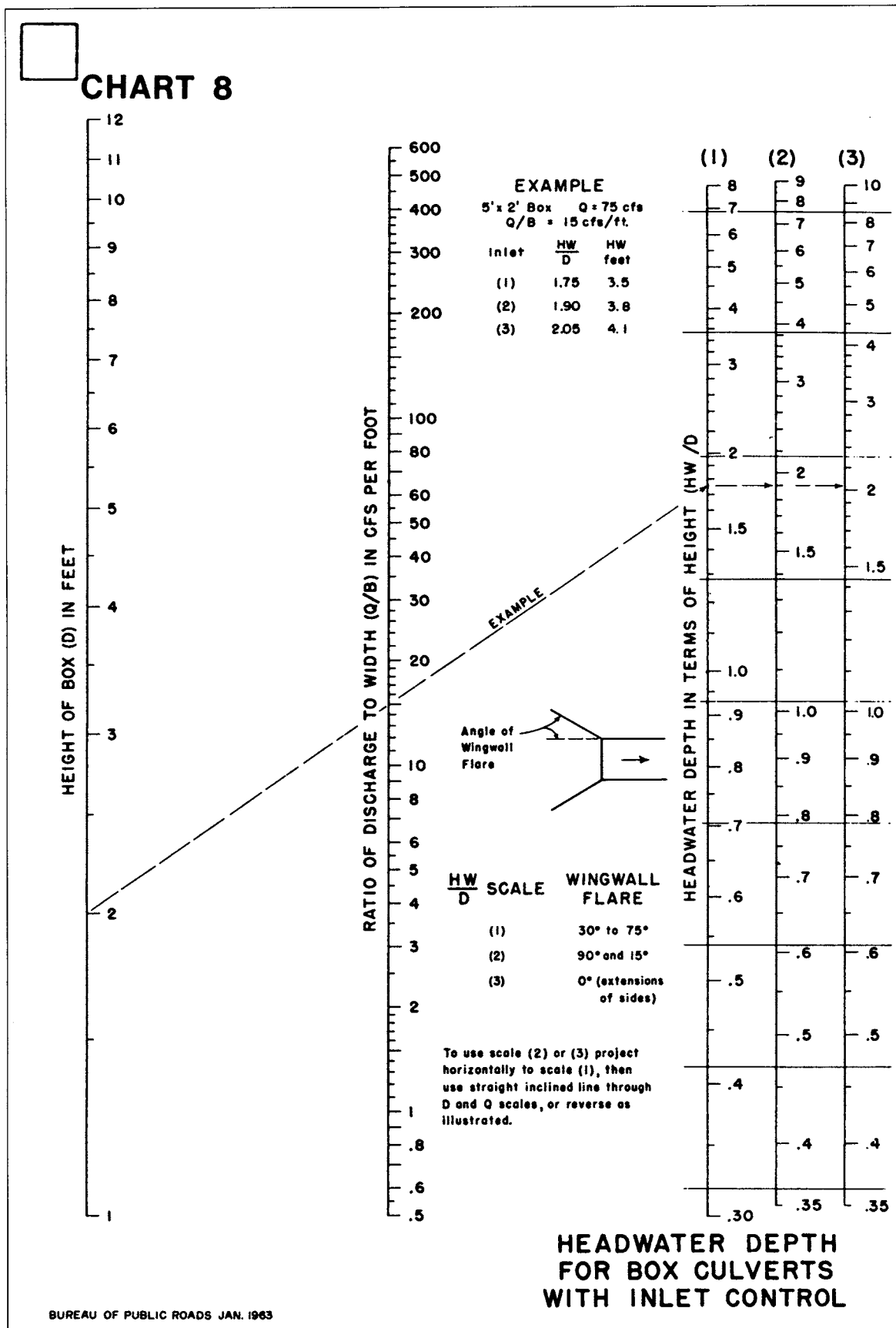


FIGURE 21.37 HDS-5 Chart 8, Nomograph for headwater depth for box culvert with inlet control.



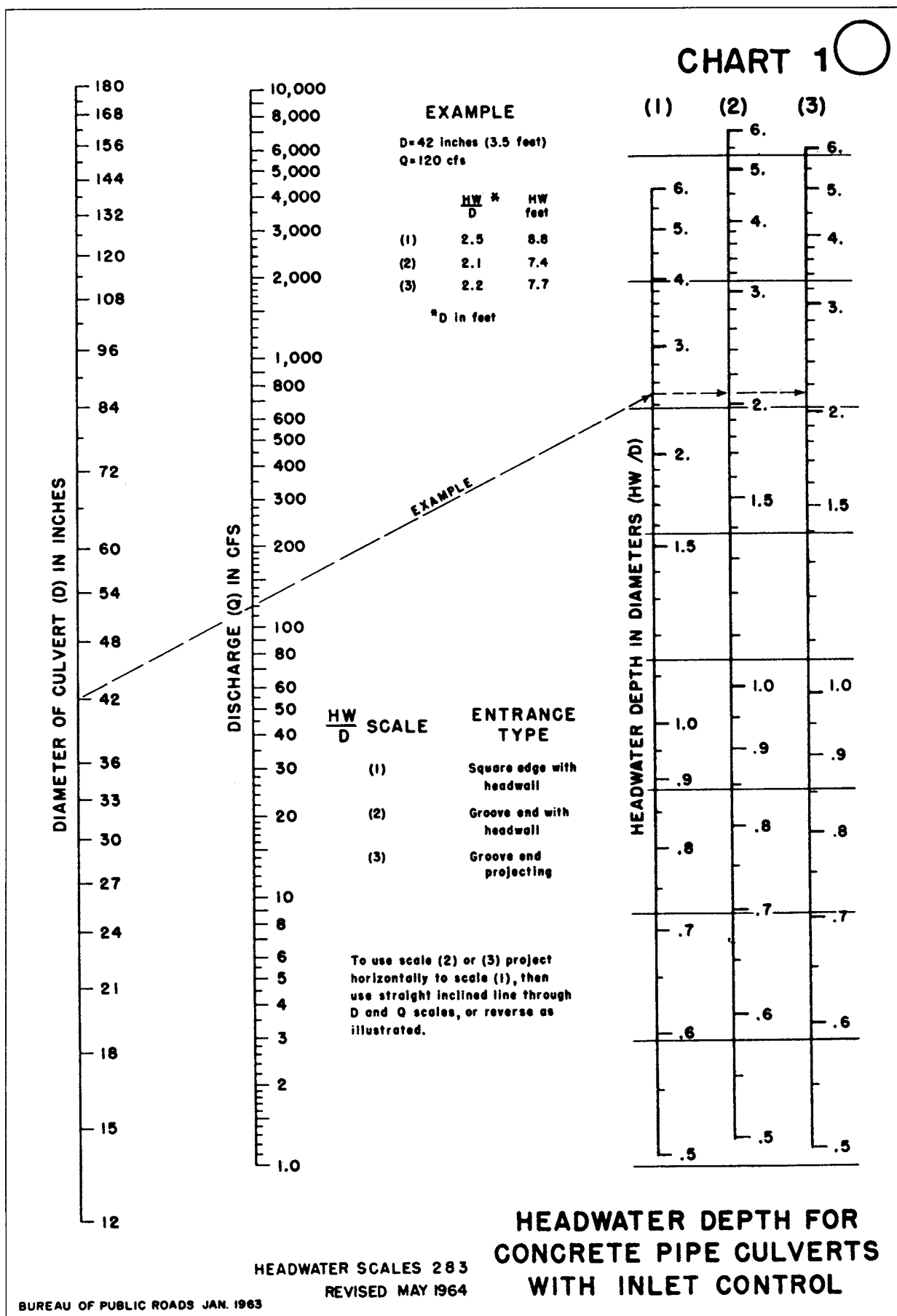


FIGURE 21.38 HDS-5 Chart 1, Nomograph for headwater depth for concrete pipe culvert with inlet control.

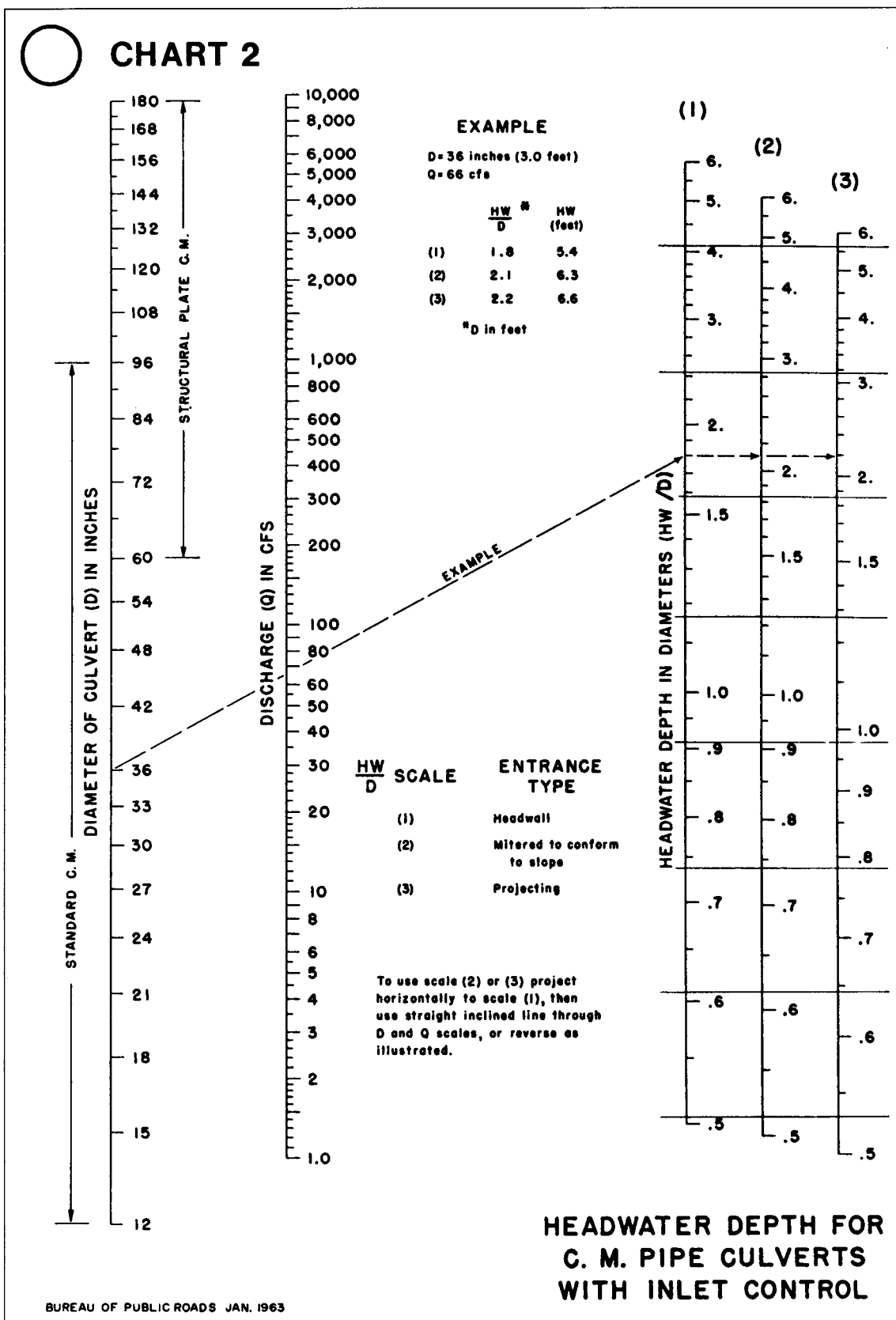


FIGURE 21.39 HDS-5 Chart 2, Nomograph for headwater depth for corrugated metal pipe culvert with inlet control.

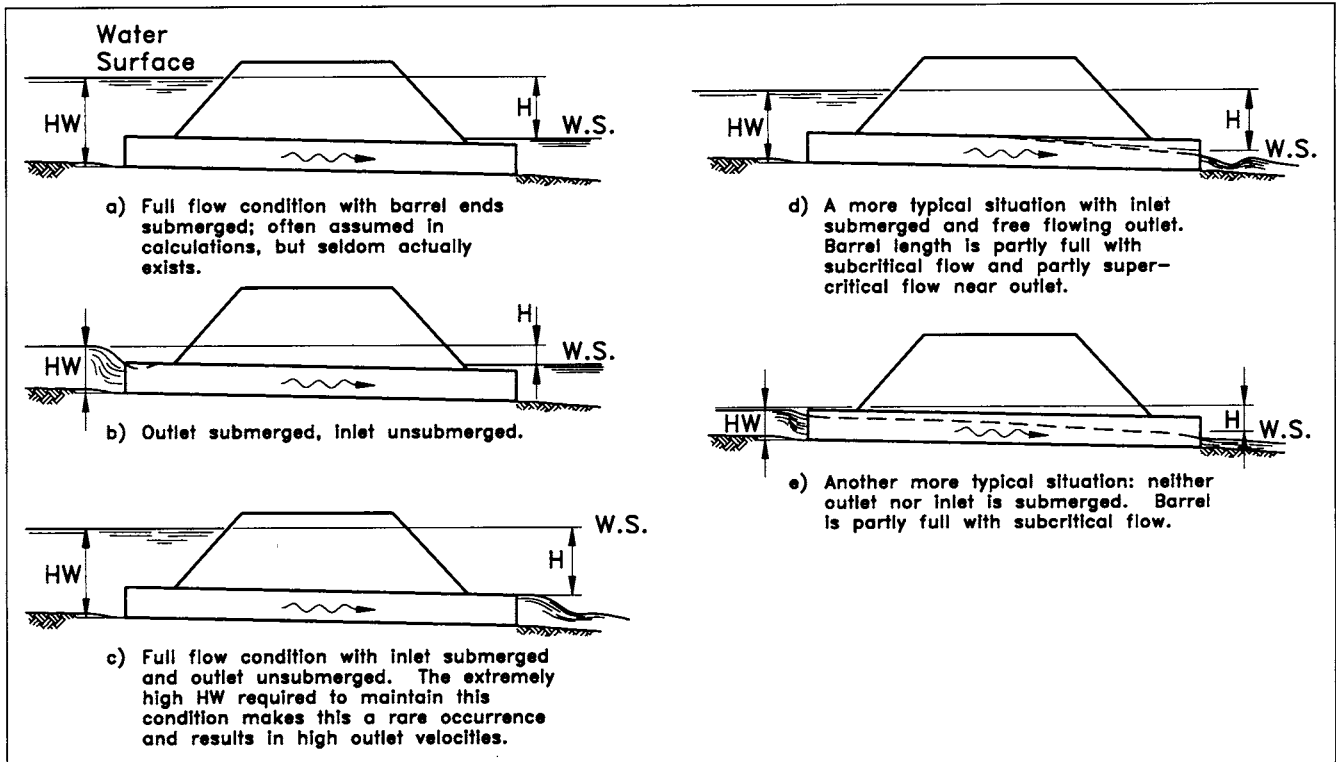


FIGURE 21.40 Types of outlet control.

nomograph in Figure 21.37. On the scale at the right, for  $HW/D = 1.2$ , extend a straight line to the left scale (height of box) value of 7 and read  $Q/B = 68$  cfs/ft on the middle scale. Therefore, a discharge of 461 cfs requires a  $461/68 = 6.8$ -foot-wide box culvert. The first trial size is  $7' \times 7'$ . The corresponding headwater depth is 8.3 ft.

Now check the  $HW$  depth for outlet control conditions. From Table 21.18 for the given headwall/wingwall configuration, the entrance loss coefficient  $k_e$  is 0.5. Substituting the values into Eq. (21.72) gives

$$H = \left( 1 + 0.5 + \frac{(29)(0.013)^2(140)}{(49/28)^{4/3}} \right) \left( \frac{(461/49)^2}{2g} \right) = 2.5 \text{ ft} \quad (21.74)$$

The critical depth  $d_c$  for the box culvert using the give data is

$$d_c = 0.315 \sqrt[3]{\left(\frac{Q}{B}\right)^2} = 0.315 \sqrt[3]{\left(\frac{461}{7}\right)^2} = 5.1 \text{ ft} \quad (21.75)$$

and  $(d_c + D)/2 = (5.1 + 7)/2 = 6.1$  ft. The variable  $h_0$  is the greater of either the tailwater elevation or  $(d_c + D)/2$ . Here  $(d_c + D)/2 = 6.1$  ft  $>$   $TW = 2.6$  ft.

Finally, the headwater depth for outlet control is

$$HW = H + h_0 - LS_0 = 2.5 + 6.1 - 140(0.003) = 8.1 \text{ ft} \quad (21.76)$$

Since the headwater depth for inlet control ( $= 8.3$  ft) and outlet control ( $= 8.1$  ft) are both less than or equal to the maximum allowable headwater depth ( $= 8.8$  ft), a  $7' \times 7'$  box culvert is an acceptable size. Also, since the headwater depth for inlet control is greater than the headwater depth for outlet control, the culvert operates as inlet control.

The second part of the design is to check if the flow depth over the road embankment for the 100-year discharge is less than 12 in. From the nomograph in Figure 21.38, extend a straight line from the left scale (height of box  $= 7$  ft) through the  $Q/B = 690/7 = 98$  cfs/ft and read  $HW/D = 1.8$  on scale 1 at the left. The resulting headwater depth  $= (7 \times 1.8) = 12.6$  ft.

Now determine the headwater depth for outlet control conditions. From Eq. (21.72)

$$H = \left( 1 + 0.5 + \frac{(29)(0.013)^2(140)}{(49/28)^{4/3}} \right) \left( \frac{(690/49)^2}{2g} \right) = 5.6 \text{ ft} \quad (21.77)$$

This same value can be obtained using the outlet control nomograph for concrete box culvert of Figure 21.42. First draw a straight line from the dimension of square box scale ( $7 \text{ ft} \times 7 \text{ ft}$ ) to the length scale value of 140 ft for  $k_e = 0.5$ . Next draw a straight line from the discharge scale

**TABLE 21.18 Entrance Loss Coefficients**

OUTLET CONTROL, FULL OR PARTLY FULL ENTRANCE HEAD LOSS

$$H_e = k_e \left( \frac{V^2}{2g} \right)$$

TYPE OF STRUCTURE AND DESIGN OF ENTRANCE	COEFFICIENT $k_e$
<i>Pipe, Concrete</i>	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded (radius = $\frac{1}{12} D$ )	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope*	0.5
Beveled edges, 33.7 or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
.....	
<i>Pipe or Pipe-Arch, Corrugated Metal</i>	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End section conforming to fill slope*	0.5
Beveled edges, 33.7 or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
.....	
<i>Box, Reinforced Concrete</i>	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of $\frac{1}{12}$ barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30–75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of $\frac{1}{12}$ barrel dimension, or beveled top edge	0.2
Wingwall at 10°–25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

\*Note: "End section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both *inlet* and *outlet* control. Some end sections, incorporating a *closed* taper in their design have a superior hydraulic performance. These latter sections can be

Reference: FHWA, *Hydraulic Design of Highway Culverts*, HDS-5, 1985.

PROJECT : _____		STATION : _____		<b>CULVERT DESIGN FORM</b>								
SHEET _____ OF _____		DESIGNER / DATE : _____ / _____		REVIEWER / DATE : _____ / _____								
<p style="text-align: center;"><u>HYDROLOGICAL DATA</u></p> <p>SEE ADD'L SHTS. <input type="checkbox"/> METHOD : _____</p> <p><input type="checkbox"/> DRAINAGE AREA : _____ <input type="checkbox"/> STREAM SLOPE : _____</p> <p><input type="checkbox"/> CHANNEL SHAPE : _____</p> <p><input type="checkbox"/> ROUTING : _____ <input type="checkbox"/> OTHER : _____</p> <p style="text-align: center;"><u>DESIGN FLOWS/TAIWATER</u></p> <table style="width:100%; border-collapse: collapse;"> <tr> <td style="width:33%; border-bottom: 1px solid black;">R. I. (YEARS)</td> <td style="width:33%; border-bottom: 1px solid black;">FLOW (cfs)</td> <td style="width:33%; border-bottom: 1px solid black;">TW (ft)</td> </tr> <tr> <td style="border-bottom: 1px solid black;"> </td> <td style="border-bottom: 1px solid black;"> </td> <td style="border-bottom: 1px solid black;"> </td> </tr> </table>		R. I. (YEARS)	FLOW (cfs)	TW (ft)				<p>ROADWAY ELEVATION : _____ (ft)</p> <p style="text-align: right;">S = S<sub>0</sub> - FALL / L<sub>0</sub></p> <p style="text-align: right;">S = _____</p> <p style="text-align: right;">L<sub>0</sub> = _____</p>				
R. I. (YEARS)	FLOW (cfs)	TW (ft)										
<p><u>CULVERT DESCRIPTION:</u></p> <p>MATERIAL - SHAPE - SIZE - ENTRANCE</p>		<p>TOTAL FLOW (cfs)</p> <p>FLOW PER BARREL (1)</p>		<u>HEADWATER CALCULATIONS</u>						<p>CONTROL HEADWATER ELEVATION</p> <p>OUTLET VELOCITY</p>		<p>COMMENTS</p>
<p><u>TECHNICAL FOOTNOTES:</u></p> <p>(1) USE Q/NB FOR BOX CULVERTS</p> <p>(2) HW<sub>i</sub>/D = HW<sub>1</sub>/D OR HW<sub>1</sub>/D FROM DESIGN CHARTS</p> <p>(3) FALL = HW<sub>i</sub> - (EL<sub>hd</sub> - EL<sub>st</sub>); FALL IS ZERO FOR CULVERTS ON GRADE</p>		<p>(4) EL<sub>hi</sub> = HW<sub>i</sub> + EL<sub>i</sub> (INVERT OF INLET CONTROL SECTION)</p> <p>(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.</p>		<p>(6) h<sub>0</sub> = TW or (d<sub>c</sub> + D/2) (WHICHEVER IS GREATER)</p> <p>(7) H = [1 + k<sub>e</sub> + (29n<sup>2</sup>L) / R<sup>1.33</sup>] v<sup>2</sup> / 2g</p> <p>(8) EL<sub>ho</sub> = EL<sub>o</sub> + H + h<sub>0</sub></p>								
<p><u>SUBSCRIPT DEFINITIONS:</u></p> <p>a. APPROXIMATE</p> <p>i. CULVERT FACE</p> <p>hd. DESIGN HEADWATER</p> <p>hi. HEADWATER IN INLET CONTROL</p> <p>ho. HEADWATER IN OUTLET CONTROL</p> <p>i. INLET CONTROL SECTION</p> <p>o. OUTLET</p> <p>st. STREAMBED AT CULVERT FACE</p> <p>tw. TAILWATER</p>		<p><u>COMMENTS / DISCUSSION:</u></p>						<p><u>CULVERT BARREL SELECTED:</u></p> <p>SIZE : _____</p> <p>SHAPE : _____</p> <p>MATERIAL : _____</p> <p>ENTRANCE : _____</p>				

FIGURE 21.41 Culvert design form. Reference: FHWA, Hydraulic Design of Highway Culverts HDS-5 (1984).

(= 690 cfs) through the point where the previously drawn line intersects the turn line to the head (H) scale on the left. Read a value of H = 5.6 ft.

The critical depth  $d_c$  is

$$d_c = 0.315 \sqrt[3]{\left(\frac{Q}{B}\right)^2} \tag{21.78}$$

$$= 0.315 \sqrt[3]{\left(\frac{690}{7}\right)^2}$$

$$= 6.5 \text{ ft}$$

whereby  $(d_c + D)/2 = (6.5 + 7)/2 = 6.8$  ft. Hence, the variable  $h_0 = 6.8$  ft. The headwater depth for outlet control is

$$HW = H + h_0 - LS_0 \tag{21.79}$$

$$= 5.6 + 6.8 - 140(0.003) = 12.0 \text{ ft}$$

For the 100-year discharge the culvert operates under inlet control (i.e., 12.6 > 12.0).

Based on the assumption of no roadway overtopping, the headwater elevation for the 100-year discharge is 326.8,

which is 2.3 ft above the road. Finding the actual depth over the road is another trial-and-error procedure. For a given headwater (above elevation 324.5), the total flow is the amount that the given headwater depth pushes through the culvert plus the amount of flow going over the embankment. The objective is to find the headwater depth where the combined flows equal the specific design discharge.

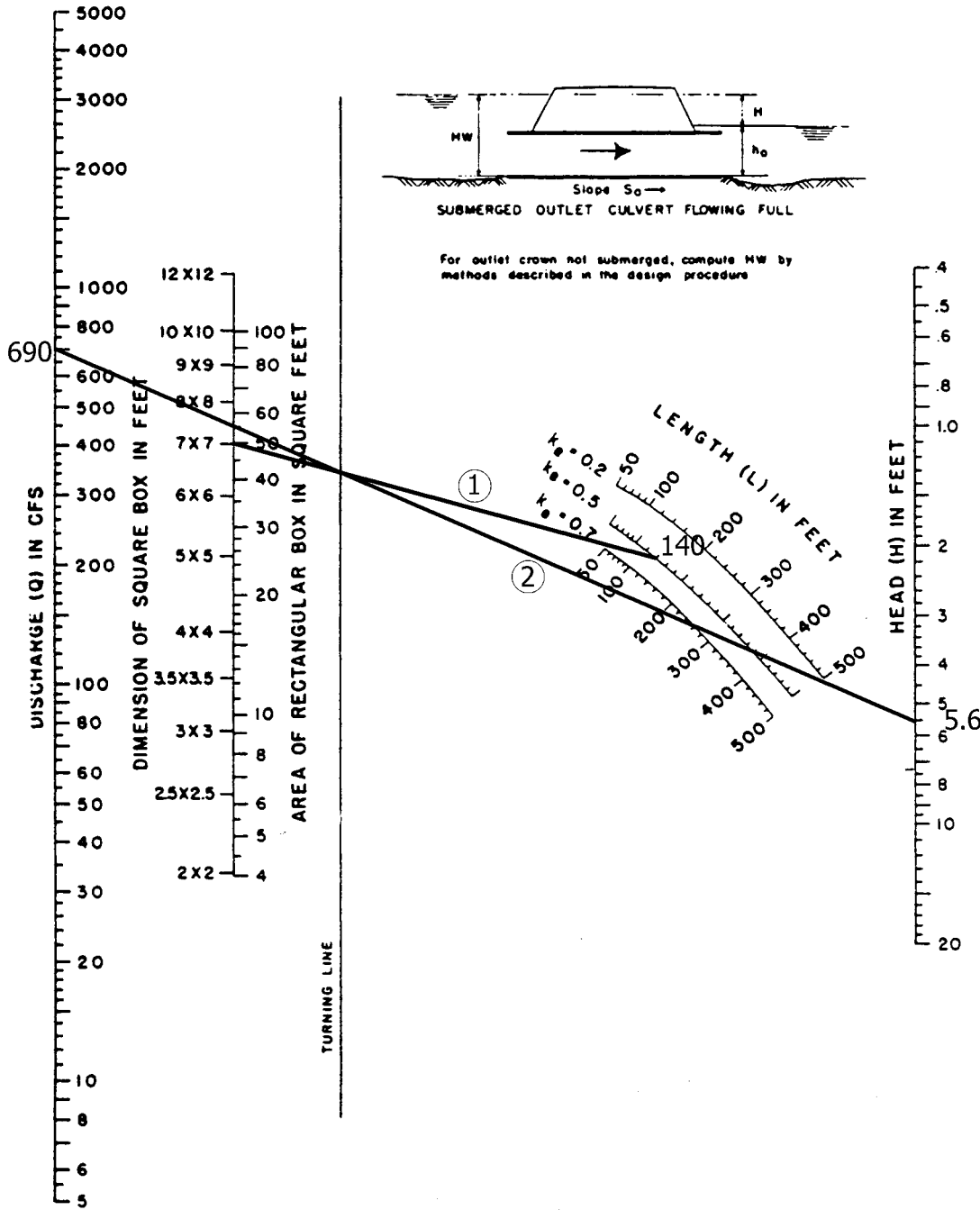
Flow over the embankment is typically modeled as a weir. The length of the weir and depth of water over the weir are difficult to determine when the road profile is a sag vertical curve or when the embankment surface has a very irregular shape. There are various numerical methods to approximate each of these values. Sometimes a simple approximation of a broadcrested rectangular weir can be used. For more complex situations, many culvert analysis programs will allow for the input of the roadway profile above the culvert and calculate the weir flow incrementally over the road until the resulting discharge balances. Weir flow is

$$Q_w = C_w LH^{3/2} \tag{21.80}$$

where  $C_w$  is a weir coefficient that depends on the shape



# CHART 15



HEAD FOR  
CONCRETE BOX CULVERTS  
FLOWING FULL  
 $n = 0.012$

AU OF PUBLIC ROADS JAN. 1963

FIGURE 21.42 Outlet control nomograph for example problem.

and depth of water above the weir relative to the depth of water below the weir,  $L$  is the length of the weir, and  $H$  is the depth of water above the weir. For a broadcrested trapezoidal weir,  $C_w$  ranges between 2.5 to 3.1. Refer to hydraulic handbooks, such as Brater and King, for various other values for different types of weirs.

To determine if the roadway overtops less than 1 ft, as was required in the example problem, we will assume that a 50-ft long rectangular weir approximates the overtopping portion of the roadway. Using Eq. (21.80), a  $C_w = 2.6$ ,  $L = 50$  ft and  $H = 1$  ft (allowable overtopping) yields a weir discharge of 130 cfs. Next we determine whether the combination of weir flow and culvert flow, at the allowable elevation of 325.5 (1 ft above the road elevation), is sufficient to pass the 100-year discharge.

Using an elevation of 325.5, the corresponding  $HW/D$  ratio of  $(325.5 - 314.2)/7 = 1.61$ . From the box culvert with inlet control nomograph (Figure 21.37), construct a straight line from scale  $HW/D$  scale 1 for  $HW/D = 1.61$  to the left scale for  $D = 7$  ft. Read  $Q/B = 90$  and find  $Q = 7(90) = 630$  cfs. The total flow of culvert and weir flow is  $630 + 270 = 760$  cfs. Our 100-year design discharge is equal to 690 cfs. Therefore, there is sufficient capacity within the culvert and the one ft of allowable overtopping of the road to pass the design discharge. A trial-and-error process using the same procedures could calculate the actual 100-year water surface elevation.

Recognize, however, that the  $7' \times 7'$  box culvert is the minimum size necessary for the given flows. Other issues may require a larger culvert. For instance, upstream flood stages may be increased above what might be acceptable. Besides the local design criteria, another consideration is the amount of upstream land inundated because of the increased flood elevation. In flat areas, with significant land value, a 0.5-ft rise in flood stage may mean acres of land impacted by flooding, resulting in reduced development potential or increased development costs for grading. Therefore, the cost to increase the culvert size in order to reduce upstream water surface elevations could be warranted.

Another consideration in culvert sizing includes providing for the increase in runoff from proposed upstream development. Often, communities will require the designer to consider the potential for upstream development, usually based on an approved comprehensive or zoning plan. For many culvert installations, the required design storm could be the 25-year storm or even 50-year storm. Most localities do not regulate stormwater management (SWM) controls to mitigate for the 25- or 50-year storm events. Many localities only consider the 10-year and smaller events. Therefore a culvert located in the lower part of a watershed, especially where development potential is high, should be sized to accommodate the increase in peak discharges over time.

This example uses the culvert nomograph for inlet control to calculate the headwater depth. Equations (21.66),

(21.67), and (21.68) could have also been used for the same purpose. The nomograph is convenient and quick; however, the equations can be incorporated into computer programs or hand-held calculators. A variety of computer programs are available to assist and automate the culvert analysis process. There are also numerous inlet and outlet nomographs for various culvert shapes that can be found in *Hydraulic Design of Highway Culverts* by the FHWA (1985), available from National Technical Information Service, Springfield, Virginia.

## STORM SEWER AND INLET LAYOUT

The storm sewer design, like most design processes, is an iterative process. The results from one iteration provide the details necessary for further refinement of the design in the next iteration. Each iteration attempts to optimize the design in order to provide adequate drainage control, without excessive number of inlets or length of pipe. In most cases, design of the storm sewer system requires between two and four iterations. The design of the minor system involves:

- Locating the inlets
- Hydrologic analysis to size the inlets
- Determining the pipe network
- Hydrologic and hydraulic analysis to size the pipes
- Checking to see that all constraints are met; all constraints are those dealing with hydrology and hydraulics, compliance with local criteria and standards, and any impacts to other components of the project.

Requisite information to begin design of the storm drain system includes on- and off-site topographic maps, current and projected land use maps, soil identification maps, floodplain delineation maps, floodplain reports, existing site plans of surrounding property, stream and channel outfall information and the plans of the proposed conditions of the project. Worksheets of the proposed project in plan view, at a scale of  $1'' = 50'$  or larger are preferable. The worksheets should show proposed and existing topography, layout of the proposed streets, buildings and parking areas, profiles of the road alignment, horizontal and vertical location of all existing utilities, and any proposed utilities. Much of this existing information should already be in project files, having been accumulated during the course of the project.

## Location of Inlets in Streets

Inlets are placed at all lowpoints in sag vertical curves. In sag locations, where there is a higher propensity for clogging of grates, the spread of ponded water presents a traffic hazard. Flanking inlets should be considered, if they are not required by code, in the sump area of a sag vertical curve. The addition of flanking inlets, which limit the

spread and ponding at the lowpoint, act as relief inlets when the sump inlet is clogged. However, the cost of construction and maintenance of such inlets, especially on tertiary streets, may not be warranted.

Table 21.19 is the Federal Highway Administration's recommendation for spacing of the flanking inlets. This recommendation is based on the desired depth at the curb and the vertical curve length defined by  $K = L/A$ , where  $L$  is the length of the vertical curve and  $A = g_2 - g_1$  is the algebraic difference of the tangent slopes of the vertical curve.

For an example, what is the recommended distance to the flanking inlets for a 300-ft long vertical curve with longitudinal slopes of  $g_1 = -3\%$  and  $g_2 = +3\%$  and the depth of flow at the curb is 0.4 ft? Solving for  $K$ ,

$$K = \frac{L}{g_2 - g_1} = \frac{300}{(+3) - (-3)} = 50 \quad (21.81)$$

Entering Table 21.19 for depth at curb value = 0.4 ft and  $K = 50$  gives a distance of 63 ft.

On normal crown street sections, two inlets are needed, one on each side of the street at the sump. Two inlets are also needed if the lowpoint (sump) of a sag vertical curve occurs on a section of road that is superelevated, since the standard gutter on the superelevated side forces the water against the face of curb, although some localities have a "spill" gutter that dispenses the water away from the curb. In these cases, only one inlet is needed at the lowpoint of the vertical curve on the low side of the superelevation.

Be aware, however, that in certain situations the location of a lowpoint on the outer edge of pavement does not al-

ways occur at the same location as the lowpoint on the profile grade line (PGL). For example, the lowpoint locations may not coincide when a superelevation transition<sup>8</sup> occurs at or near the lowpoint of the PGL. Other potential situations in which this might occur typically involve a superelevation transition and/or pavement widening, as when adding a right turn lane. To properly locate the drainage inlet, the design engineer should check the elevations of the outer edge of pavement along the superelevation transition segment to determine the location of any "false lowpoints."<sup>9</sup> Most urban streets have curb or curb and gutter sections. In those situations, where the urban street is required to have superelevation through horizontal curves, certain conditions create the situation where a lowpoint occurs and is not in an obvious sump. To attain full superelevation or to go from full superelevation to normal crown (or reverse superelevation in the case of compound curves) requires a length of transition. If the rate of superelevation is greater than the longitudinal rate of grade change, a lowpoint in the gutter area results. Although superelevation transitions through horizontal curves are not considered good practice, this situation, in some instances, cannot be avoided due to other constraints. The engineer should be aware of the conditions that present this predicament and locate the not-so-obvious lowpoint.

<sup>8</sup> A superelevation transition is a segment of the road used to gradually change the cross slope of the pavement from a normal crown section (straight section of road) to a superelevation section (curved section of road, like a racetrack curve that is banked) and vice versa.

<sup>9</sup> The term *false lowpoint* is used to indicate a lowpoint location difference from the lowpoint location given on the PGL.

**TABLE 21.19 Distance to Flanking Inlets in Sag Vertical Curve Locations Using Depth at Curb Criteria**

<b>SPEED (MPH):</b> <b>K(= L/A):</b>	<b>20</b>	<b>25</b>	<b>30</b>	<b>35</b>	<b>40</b>	<b>45</b>	<b>50</b>	<b>55</b>	<b>60</b>	<b>65</b>	<b>70</b>	
	<b>20</b>	<b>30</b>	<b>40</b>	<b>50</b>	<b>70</b>	<b>90</b>	<b>110</b>	<b>130</b>	<b>160</b>	<b>167</b>	<b>180</b>	<b>220</b>
Depth @ curb												
0.1	20	24	28	32	37	42	47	51	57	58	60	66
0.2	28	35	40	45	53	60	66	72	80	82	85	94
0.3	35	42	49	55	65	73	81	88	98	100	104	115
0.4	40	49	57	63	75	85	94	102	113	116	120	133
0.5	45	55	63	71	84	95	105	114	126	129	134	148
0.6	49	60	69	77	92	104	115	125	139	142	147	162
0.7	53	65	75	84	99	112	124	135	150	153	159	176
0.8	57	69	80	89	106	120	133	144	160	163	170	188

Reference: USDOT FHWA, *Drainage of Highway Pavements*, HEC-12, 1984.



Inlet placement in an intersection requires detailed analysis of the intersecting streets' profiles and cross sections. Intersections have the potential for creating unusual drainage patterns and collection networks. The design of an intersection involves matching the cross slope of the intersecting street with the longitudinal profile of the through street. Curb returns (the horizontal curve portion of the curb used to join the curb of the two intersecting streets) and the cross sections of the intersecting streets, coupled with any horizontal and vertical curves, can create sump areas in the most inconspicuous and least favorable locations.

Drainage patterns can be further complicated by deviations from the cross slopes shown on the typical section, which may be relaxed through the intersection to fit the pavement from the intersecting street to the pavement of the through street. Additionally, the profile of the curb return itself may not follow a known mathematical function (e.g., a parabolic curve) and typically does not exactly coincide with the street profile. The intersection's contour grading plan usually provides clues to the drainage pattern and the potential location of sump areas. An experienced engineer will design the intersection with drainage as a priority consideration. For example, locating an intersection within the low area of a vertical curve creates drainage design difficulties. Avoiding a very flat cross slope at or near the lowpoint of a sag vertical curve will also facilitate the drainage design in the intersection area.

Often local design criteria prescribe an upper limit to the amount of flow that can bypass an intersecting street. Typically, the amount of flow crossing the intersection depends on the functional street classification. The higher the functional classification, the lower the amount of discharge allowed to pass the intersection. Occasionally design criteria may even prescribe that no flow can pass the intersection regardless of the street's functional classification; therefore, the inlet should be located at the ends of the curb returns. Depending on the longitudinal street slope and the gutter flow, an inlet just upstream of this location may also be necessary. Locating inlets at the ends of curb returns also reduces flows around the curb returns, and this may be a consideration if the longitudinal gradient of either street is very steep, or if the area is a high pedestrian traffic area.

It is difficult to install curb inlets in curb returns with short radii. Therefore, the engineer may decide to show the profile of the curb return to ensure no sump area exists and to guide the contractor who is installing the curb return. Inlets with relatively short throats or short grates may be installed on curb returns with larger radii without disrupting the continuity of the curve. Occasionally curb returns of large radii (e.g., 100 ft or more), that are longer than normal, may be designed to intentionally have a sump area to keep runoff from spilling into the travel area of the intersection.

After inlets have been placed at the required locations, the location of any and all remaining inlets is dictated by

the limitations on spread of flow into the street. Frequently the inlet is intentionally sized to intercept less than the total design flow approaching the inlet, thus allowing some flow to carry over and be added into the design discharge for the next downstream inlet. One study concludes that an inlet's capacity is maximized if some water is permitted to bypass the inlet. Therefore, the size and location of street inlets on grade depends on how the design discharge in the street is divided among the inlets.

In parking areas, the grading plan determines the drainage pattern and inlet location. In commercial building sites, pedestrian traffic is an important consideration for grading and drainage. Sump areas should be avoided in pedestrian travelways such as crosswalks or near entrances to the building. Additionally, sump areas should be avoided in areas where passengers discharge from vehicles, such as bus stops or walkways. Although there is typically no limitation for the spread of flow into privately maintained travel lanes and parking areas, good judgment is needed to limit the spread in cold regions, since the formation of ice sheets is a concern. Handicap ramps and parking spaces also need to be considered when locating drainage inlets and sump areas.

### Location of Inlets Outside of Paved Areas

The drainage system should be carefully integrated into all projects. Ideally, the drainage system should accommodate the site or roadway engineer's plan and not be the driving factor behind the remaining design components. In single-family detached projects, drainage patterns favor the rear and side of the houses. Swales and inconspicuous ditches convey the water across the rear of the lots. The rate of flow being conveyed across a lot is limited to nonerosive velocities. No more than 2 to 4 cfs should concentrate and flow across the lots without being intercepted or conveyed in a well-defined and stabilized channel. Maintenance and ownership of swales and ditches is a factor in drainage design in suburban developments. Drainage easements may be necessary. The desirability of a lot is reduced if a drainage easement runs through the middle of a rear yard. In most cases, the drainage system should be kept within the public right of way.

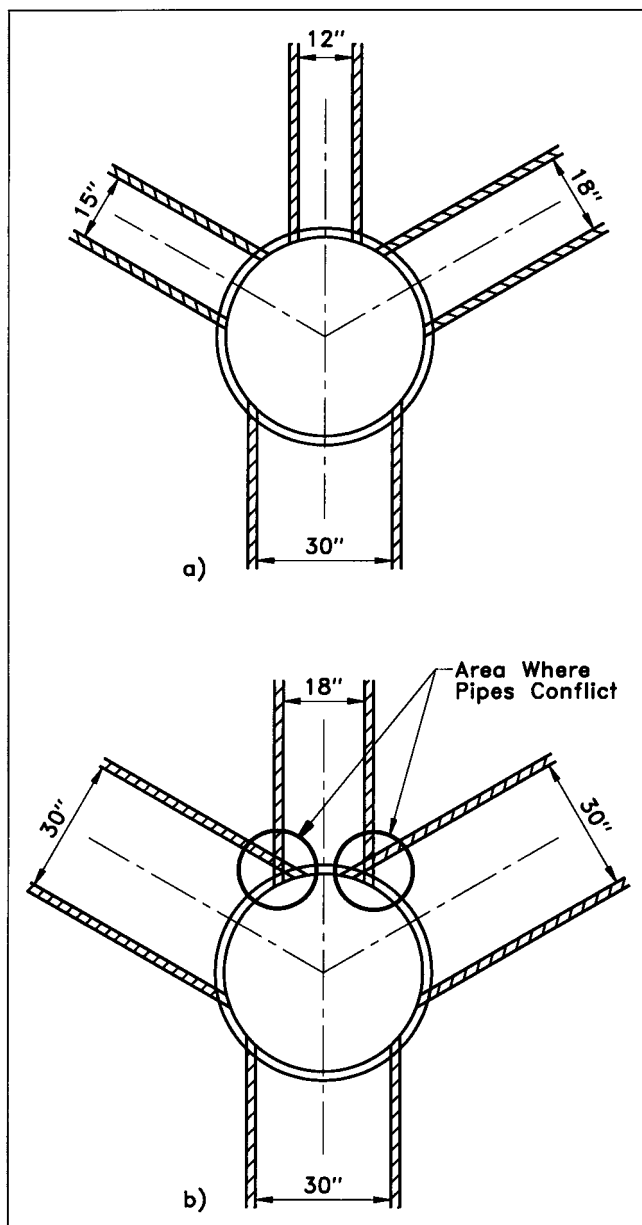
### Manholes

Manholes are typically precast circular concrete barrel sections, in 3 to 4-ft lengths, which stack on top of each other. The elevation of the top of the manhole is adjusted to meet grade with spacer rings. The top is covered with an iron ring fitted with an iron cover. Many types of inlets use the manhole barrel sections but have a precast throat or grate that fits to the top instead of the frame and cover. Manholes are used to change horizontal and/or vertical direction of pipes, while also acting as a junction, allowing the convergence of several incoming pipes.

It is common practice to match crowns of an incoming and outgoing pipe. When several pipes converge at a man-

hole, matching crowns may not be feasible due to the width of the manhole, the diameters of the incoming pipes, and the angles of approach. The diameter of the manhole depends on the size of the pipes connecting to it. Standard manhole diameters are approximately 4 ft.

In Figure 21.43(a), the size and angles of the pipes are such that they can be connected to the manhole without interfering with each other. Compare this to Figure 21.43(b), where the pipe sizes and angles cannot fit into the manhole at the same elevations (or nearly same elevations). In the second case the elevations of the pipe should be staggered enough to provide for the necessary clearance. In each case the thickness of the pipe walls should be con-

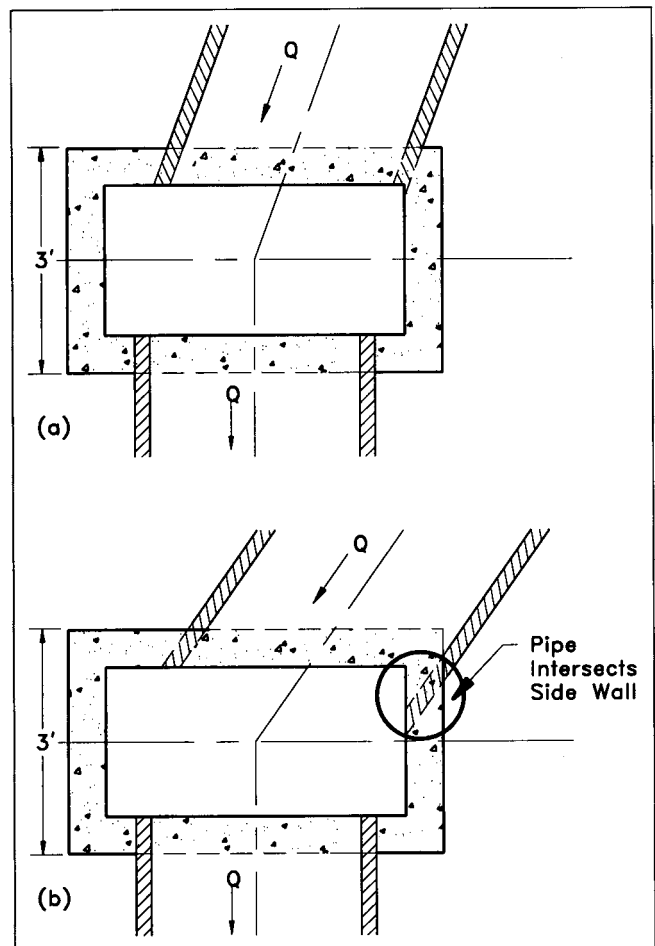


**FIGURE 21.43** Pipes connecting at manhole at same elevation.

sidered when determining the necessary manhole size. Note: A rule of thumb for the wall thickness of a standard reinforced concrete pipe is as follows. The wall thickness (in inches) is equal to the diameter of the pipe (in feet) plus one. For example, a 48-in. pipe will have a thickness of 5 in. (4 ft diameter + 1).

Similar to the preceding discussion, where several pipe connections to the manhole must be checked for proper fit, the skew angle of the pipe connection at a rectangular structure must be checked for fit. Figure 21.44(a) shows an incoming 36-in. diameter pipe properly fitted to a 3 ft by 5 ft (outside dimensions) structure. In Figure 21.44(b), the pipes are skewed and do not properly connect to the structure. In this example, the center of the structure is aligned with the centerline of the pipe, and the skew angle causes part of the incoming pipe to overlap onto the 3 ft side of the structure.

In most moderate-density developments, manholes and inlet structures will be spaced less than several hundred feet apart by necessity. Most localities will set design criteria limiting the maximum distances between manholes, and this often is a function of the pipe size. Table 21.20 is the



**FIGURE 21.44** Skew angle for pipes connecting to rectangular structure.

**TABLE 21.20 Recommended Manholes Spacing**

PIPE DIAMETER	DISTANCE (ft)
12–24	350
27–36	400
42–54	500
≥60	1000

recommended spacing provided by AASHTO. This distance is usually determined by maintenance and accessibility concerns.

### Stormwater Management Considerations

Often the storm sewer and pipe design of a roadway drainage system will be governed by the need to convey water to a stormwater management (SWM) facility. SWM facilities can include large ponds, basins, infiltration areas, grassed swales, or other devices meant to reduce peak flow runoff or to treat runoff for water quality purposes. Much of the focus on today's SWM design, for roadway and land development projects, concerns water quality. Therefore, even if the designer can provide the necessary detention or prove that detention requirements are not warranted, more than likely, water quality requirements will still be required. Given the site constraints, or the SWM needs required for the development, most storm sewer systems and inlets will be driven by the need to collect runoff and discharge it into a SWM facility.

With the growing demand for redevelopment and the emphasis on water quality, numerous man-made structures are being used to provide water quality. There are a variety of water quality inlets that help to remove oil, grit, sediment, and pollutants. Many of these devices are underground structures varying in size (from regular-sized manholes to large vault boxes) that are connected directly to a storm sewer system. Given the amount of flow or drainage area accumulated in a storm sewer system, the engineer may decide to design a storm sewer system that diverts a portion of the runoff in the storm sewer system to a water quality inlet. This will divert the "first flush" of the stormwater (in theory, the runoff with the most pollutants) into the water quality inlet, while allowing the remainder of the storm water to flow through the pipe system. When storm water is diverted within a system, the water quality inlet is said to function "off-line." When the water quality inlet is placed within the storm sewer system itself (usually on small catchment areas only), the system is considered to be "on-line."

### Pipe Design

After inlet locations and sizes are established, the pipe conveyance system is determined. The pipe network typically converges to the outfall point. Occasionally, due to site conditions, the network is separated into several systems, each discharging at different outfall points or connecting to another storm sewer network. Outfall points can be natural channels of adequate capacity, retention/detention areas, lakes, and rivers. The size, slope, and depth of the pipe in the network are controlled by the elevation of the outfall point. This is more of a problem in flat terrain than in rolling and hilly terrain. In flat terrain, the engineer may have design problems trying to match outfall elevations; however, in steep terrain, high velocity and high energy losses may create a potential problem. In general, the slope and size of the pipe are kept to the minimum required to carry the design flows at near full capacity, and this helps ensure proper cleanout of the storm sewer systems. Recognize, however, that many public agencies have a requirement for a minimum allowable pipe size. For storm sewer systems in steep terrain, the minimum pipe size may have excess capacity.

The pipe network, wherever possible, should be located within the public right of way. Most storm sewer systems are maintained by a government agency. Therefore, any part of the conveyance system on private property requires an easement to allow access. Additionally, it is desirable to locate the pipe parallel to a property line, offset to one side. This allows for maximum use of the property (i.e., no encumbrances through the middle of the lot) and also allows an owner to build a fence without fear of having to dismantle it later if the pipe needs to be repaired.

Common practice is to set the pipe at minimum depth (i.e., minimum cover or minimum inlet depth) or at the shallowest depth possible (to ease in constructability) and select the pipe size and slope that convey the design discharge at near full capacity. The pipe network is typically designed such that the HGL is at or below the crown of the pipe (i.e., nonpressure flow). Under such conditions, the pipe network is surcharged for storms that generate runoff greater than the design storm. The HGL should be checked against this secondary, or "check" storm—the local design standards may even require it. The designer should determine the overland relief path for the larger secondary storm surcharges for safety or property damage concerns. Site grading or alignment, which would result in these surcharges damaging private property, should be re-thought.

Discharge velocity in a pipe should be kept within a nominal range to prevent scour and eliminate sediment buildup. Although efforts are made to keep sediment from entering the system, all sediment cannot be kept out. Sediment transported by very high velocities causes abrasion damage to the pipe. Low velocities cause sediment to settle out and reduce the pipe capacity. A recommended velocity range is 2 to 12 fps.

Avoiding conflicts with other underground utilities is a major objective in the design of a storm sewer system. Since storm sewer flow is driven by gravity, any conflicts with other proposed nongravity-dependent utilities are usually resolved by redirecting those utilities. Attempts are made to locate the proposed storm sewer system around any existing gravity or nongravity dependent-utilities. It can be costly to redirect existing utilities, and only in the unavoidable case is this done.

### Pipe Materials

Pipes can be manufactured from many different materials, including concrete, reinforced concrete, aluminum, steel, and various other synthetic materials. Given the locality, the preference of contractor, the proximity to manufacturing plants, and the cost of the project, the engineer must be careful to consider the pipe material when designing a storm sewer system. The various different pipes available on the market can have an enormous range of  $n$  values (from as little as 0.007 to as high as 0.033). Since  $n$  value plays an important role in pipe capacity and hydraulic grade line calculations, the selection of the pipe material could dramatically affect the design.

Some localities will provide guidance as to the types of materials preferred. Many other jurisdictions will allow the contractor to choose the pipe material. When the contractor is given the option of pipe materials, it will either be up to the contractor or the engineer to determine whether the material selected will still meet the design requirements of the construction documents. Therefore, it is important that the engineer understand the requirements of the local industry before determining pipe materials to be used for design and analysis.

### PROCEDURE FOR STORM SEWER DESIGN

One of the more frustrating predicaments encountered by designers, after painstakingly designing the system, is to find that the design pipe invert elevation at the outfall point is too low. This can be avoided if preliminary pipe sizes are set based on rough estimates of the discharge and the first iteration of design is only a rough estimate. The general procedure for designing the storm sewer system is summarized as follows.

**Data Collection.** Obtain a plan of the project area showing existing and proposed features along with the existing topography and proposed grading. Profiles of roads, water and sanitary lines, invert elevations of the outfall points, topographic maps showing areas contributing off-site runoff and discharges from pipes, and typical sections of stream channels are all necessary information when designing the storm sewer system.

**Evaluate Sump Inlet Locations.** Inlets are set at all sump areas in streets, parking areas, and in unpaved areas. Check all curb returns for lowpoints. Using topographic information, establish the drainage areas to each inlet and compute the runoff. Include runoff and discharge from any areas out-

side of the project area. Use the appropriate capacity charts for the selected inlet to determine the ponding depth and/or spread at the inlet.

**Supplementary Sump (Flanking) Inlets.** If the inlets at the sump areas are inadequate, either increase the size of the inlet or add another inlet upstream from the sump. If dual flanking inlets are not required, as recommended by FHWA, consider if only one flanking inlet is necessary to reduce the ponding and spread in the sump. Similarly, if the drainage area on one side of the sump contributes most of the runoff, then consider only one flanking inlet. In residential areas, inlets should be located where they will not interfere with driveway entrances. In many cases, final house and driveway locations are not known during the design of the storm sewer system. Usually, houses are centered on the lot and the driveway favors one side of the house. If the length of the flanking inlet is short, its location can be either the center of the lot or at the lot line extended. For longer length inlets, the engineer must exercise judgment in anticipating the driveway location to locate the inlet. Inlets are considered in residential yards when the discharge velocity in the swale approaches erosive velocities, or if flow depths are excessive. Few residents use the yard during a rainstorm; however, in poorly drained soils, ponding and residual soggy areas are a nuisance.

**Other Inlet Locations.** Other critical areas that warrant consideration are street intersections and curb returns. Many localities limit the amount of discharge that may cross through the entrance of an intersecting street. The amount of discharge may vary according to the functional designation of the street. Too much discharge flowing around a curb return causes the water to spread out as it moves around the curb. In lieu of any local restrictions, the engineer should consider adding inlets at curb returns to reduce the flow across the intersection when flows approach three to four cfs on lower category streets and when flows are greater than two cfs on higher category streets.

Placement of other inlets is then dictated by the spread limitations identified by design standards. Studies have shown that the efficiency of an inlet is increased if it is sized such that 5% to 10% of the flow is allowed to by-pass. Therefore, consideration should be given for sizing the throat lengths so that they are about 90% efficient. In the design, the by-pass flow must be included in the runoff discharge for the next downstream inlet.

**Pipe Layout.** The inlets are then connected to show how the pipes convey the water to the outfall points. The pipe network usually directs the water in the prevailing direction of the slope of the proposed grading. However, there may be some instances where judgment supersedes rule-of-thumb guidelines. Pipe runs of excessive length (e.g., greater than the maximum allowable manhole spacing) require intermittent manholes. Additionally, any changes in horizontal direction or vertical slope of the pipe (e.g., to go around a foundation or to minimize easements on lots) also require additional manholes. When additional manholes are

necessary, the engineer should check if additional inlets would be beneficial and substitute accordingly. This is not always possible or desirable, but the cost differential between the two is not significant.

**Profile the System.** Profile this tentative layout on a profile worksheet. Show the existing and proposed grades and mark the locations of the proposed inlets and manholes, as well as the existing (and proposed if available) sanitary sewer and water main crossings. The purpose of this profile is to ensure that the pipes have sufficient cover to prevent excessive excavation and to avoid utility crossing conflicts. Utility conflicts need to be identified during the first phase of design so that they do not cause a major redesign of the system in the later design phases. A more accurate profile can be drawn later after the pipe design is complete. This step is highly recommended. Although some construction plans or documents do not require storm sewer profiles, the simple task of putting the design on paper can eliminate countless redesigns of a system.

**Tentative Pipe Sizes.** Estimate the size of each pipe by adding the flows from each inlet above the inlet at the design point. *Recognize this is not the proper way to calculate flows, as pointed out in the section discussing the rational method.* However, this method, although it overestimates the flow, is adequate for tentatively estimating the pipe size. For small- to moderate-sized sites (less than 50 acres) and pipe runs of 800 ft and less, the relative error in the discharge using this method versus the correct method is tolerable for tentatively estimating pipe sizes. Using a hydraulic calculator, or other means, select the smallest pipe size, which conveys the estimated discharge at full

flow, at a slope that approximates the ground slope above the pipe. As the pipes are selected, write this information on the profile worksheet between the appropriate inlets. Draw a line between inlets that represents the invert of the pipe. The crown of the outgoing pipe is set at the same elevation, or higher, than the crown elevation of the lowest incoming pipe. When large junction losses are anticipated, the crown of the incoming pipe should be slightly higher. This increases the difference in elevation in inverts to allow for the large junction losses. Note: These last three statements affect where the invert line of the pipe is drawn.

**Final Assessment.** Check to see that the invert of the last pipe is above the invert of the outfall point. Confirm all existing and proposed utility crossings. If less than minimum clearance is provided where the storm sewer crosses a utility, concrete piers should be considered to support the storm sewer pipe at the utility crossing. If there are no utility conflicts, proper connections to outfall points are possible, and if the engineer is comfortable with the cost-effectiveness of the design, perform a more detailed design. That is, correctly compute the discharge through the pipes, fill out the pipe design and hydraulic grade line charts, and accurately draw the profiles.

Hydraulic grade line (HGL) computations should be the last step in the design. Although the pipe slopes, if properly designed, were set for gravity (nonpressure) flow, head losses through structures and other appurtenances may cause pressure flow in the system. In most cases, where pipe velocities are less than 5fps, energy losses through manholes are less than 0.5 ft. However, on systems with numerous manholes, or in those pipes where velocities are

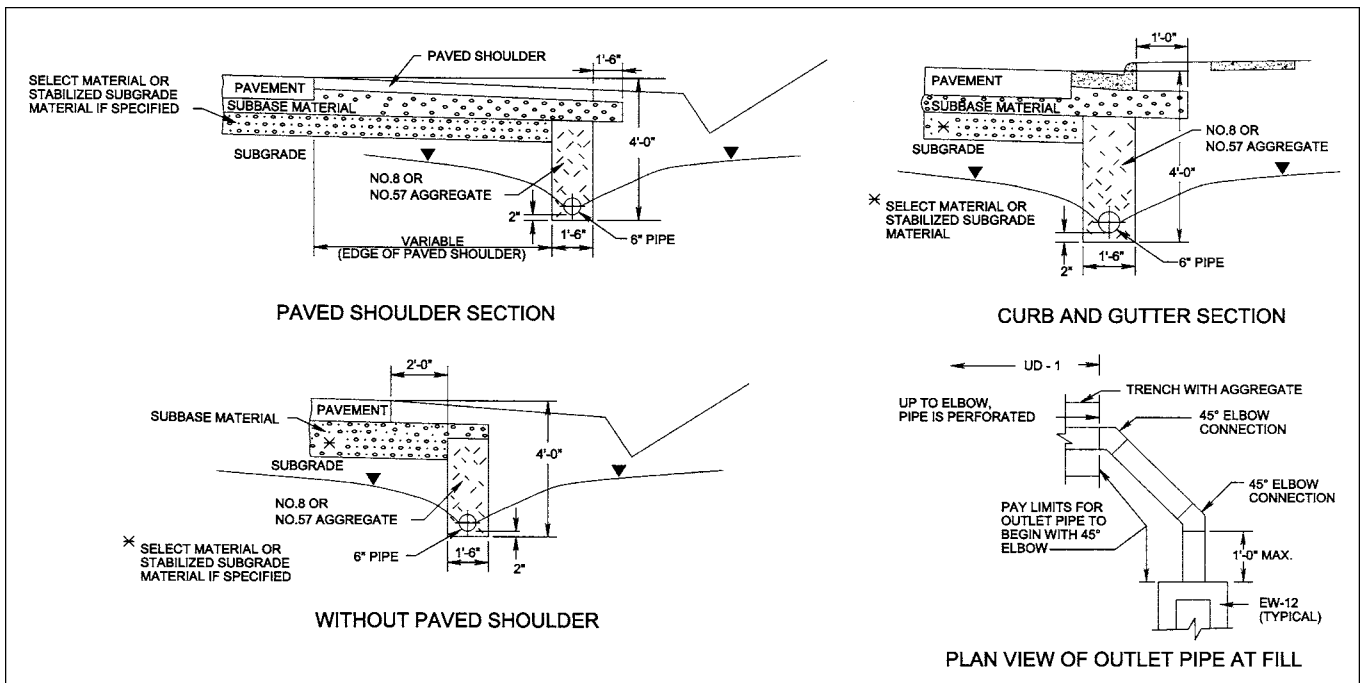


FIGURE 21.45 Typical underdrain.

10 fps and higher, the head losses accrue very quickly. For systems with these characteristics, the potential for pressurizing the system becomes very likely. Even though a specified design storm is used to design the system, local criteria may require the HGL be checked for other storms. The engineer uses his best judgment and may want to see the impact on the HGL for other design storms.

## UNDERDRAIN DESIGN

The inlet and storm sewer design has focussed on drainage pavements, which have surface water runoff. However, there are many instances where water collects underneath the pavement section, either in locations of cut/fill transitions, or just by infiltration. Water will usually collect in the subgrade portion of the pavement, especially in areas where aggregate or stone is used in the design. This collection of water underneath the pavement can be extremely problematic, leading to loss of stability and strength of the pavement structure, fatigue based on freeze/thaw conditions, or loss of cohesion and bearing pressures of the surrounding soil.

Given certain situations, this excess water underneath the pavement will require that a secondary drainage system be constructed to reduce the water collected underneath roadways. This is usually done by placing a combination of small pipes (sometimes perforated or wrapped in geotextile fabric) along the aggregates section of the pavement box and discharging the water either into a ditch or a manhole associated with the roadway drainage system. These devices are called underdrains.

In most urban situations, underdrains are placed along the edge of the pavement, below the curb and gutter section. Typical roadway design requires that the pavement be sloped to the edge to facilitate drainage. The same applies to pavement design as well, such that all layers of the pavement box (top surface coarse, base coarse, subgrade, etc.) drain to the outside. Curb and gutter sections typically are constructed on stone or aggregate bases, meaning that water can flow easily along the corridor. Underdrains placed longitudinally below the curbline, within the aggregate, can capture the majority of water trapped beneath the pavement and discharge it directly into the roadway drainage system. Figure 21.45 shows a typical underdrain.

## REFERENCES

American Iron and Steel Institute, *Handbook of Steel Drainage and Highway Construction Products*, 3d ed., Washington, D.C., 1983.

American Concrete Pipe Association, *Concrete Pipe Design Manual*, Arlington, Va., 1987.

American Association of State Highway and Transportation Officials, *Highway Drainage Guidelines*, AASHTO, Washington, D.C., 1982.

American Public Works Association, *Urban Stormwater Management*, Special Report no. 49, Chicago, Ill., 1981.

ASCE and Water Pollution Control Federation, *Design and Construction of Sanitary and Storm Sewers*, 1972.

Balmer, Peter, Per-Arne Malmqvist, and Anders Sjoberg, eds., *Proceedings of the Third International Conference on Urban Storm Drainage*, Vols. 1 and 2, June 4–8, 1984.

Brater, E.F., and H.W. King, *Handbook of Hydraulics for the Solution of Hydraulic Engineering Problems*, 6 ed. McGraw-Hill, New York, 1976.

Chow, V.T., David R. Maidment, and Larry W. Mays, *Applied Hydrology*, McGraw-Hill, New York, 1988.

Chow, V.T., *Open Channel Hydraulics*, McGraw-Hill, New York, 1959.

Clark, Robert Emmet, ed., *Waters and Water Rights*, Vol. 5, Allen Smith Co., Indianapolis, Ind., 1972.

Federal Highway Administration, *Drainage of Highway Pavements*, FHWA, Washington, D.C., 1984.

Goldfarb, William, *Water Law*, Butterworth Press, Boston, Mass., 1984.

Hendrickson, John G., Jr., *Hydraulics of Culverts*, American Concrete Pipe Association, Chicago, Ill., 1964.

U.S. Dept. of Transportation, Federal Highway Administration, *Hydraulic Design of Highway Culverts*, Washington, D.C., September 1985.

Jens, Stiffel W., and M.B. McPerson, Hydrology of urban areas, *Handbook of Applied Hydrology*, V.T. Chow, ed., McGraw-Hill, New York, 1964.

Kibler, David F., ed., *Urban Stormwater Hydrology Monograph 7*, American Geophysical Union (AGU), Washington, D.C., 1982.

McCuen, Richard H., *Hydrologic Analysis and Design*, Prentice Hall, Englewood Cliffs, N.J., 1989.

McPherson, M.B., *The Design Storm Concept*, Urban Runoff Control Planning Miscellaneous Report Series, U.S. Environmental Protection Agency, GPO, Washington, D.C., 1978.

National Weather Service, *Five to 60-Minute Precipitation Frequency for the Eastern and Central United States*, (NWS Hydro-35), Washington, D.C., 1977.

Ponce, Victor Miguel, *Engineering Hydrology*, Prentice Hall, Englewood Cliffs, N.J., 1989.

Trelease, Frank, J., Water law, *Handbook of Applied Hydrology*, V.T. Chow, ed., McGraw-Hill, New York, 1964.

U.S. Department of Agriculture, *Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years* (Technical Paper No. 40), Washington, D.C., 1963.

Yen, Ben Chie, and Ven Te Chow, Design hyetographs for small drainage structures, *Journal of the Hydraulics Division*, HY-6 V.106, American Society of Civil Engineers, 1980.



## CHAPTER 22

# DESIGN OF STORMWATER MANAGEMENT FACILITIES

*Dave Parker, P.E. / Kathleen Schaefer, P.E.*

*Updated/Revised: Philip C. Champagne, P.E.*

### INTRODUCTION

Changing land use from vacant rural land to residential/commercial use inevitably results in a decrease in pervious surface and an increase in impervious surface, resulting in a change in the hydrologic and hydraulic characteristics of the watershed. This change in basin imperviousness results in increased flow rates, increased runoff volumes, increase in the frequency of flooding and degradation of surface water quality. In response to the increased runoff, systems consisting of curb and gutter, storm sewer, ditches and paved channels are developed to convey the runoff safely through the basin. These systems directly increase flow velocity and consequently decrease the basin time of concentration, resulting in higher peak flow rates. In a typical, moderately-developed watershed, the increase in peak discharge could be from two to five times higher than pre-developed conditions. The volume of runoff can be increased by as much as 50%, and the time of concentration may be decreased by an even greater percentage (see Figure 22.1). Although such effects from a small individual site may seem inconsequential, the collective effects of numerous sites throughout the watershed can have substantial impact on the environment, especially in the lower reaches of a watershed.

Along with the increase in water quantity, urbanization results in the increase in non-point pollutants. Trace metals from galvanized downspouts, flashing and roofing materials, and pipes are washed into natural channels. Other pollutants such as tire particulate, hydrocarbon products from pavement and fuels, and mechanical part flakes can all end up in the surrounding streams, lakes, and groundwater. The damaging effects of the many pollutants are not always immediately apparent. It may be years before fish and wildlife

are affected by land use changes or before these effects become noticed. Other pollutants such as phosphorus and nitrogen, when washed into lakes, promote algae growth. If left unchecked, eutrophication and water quality degradation are imminent.

Stormwater management (SWM) is the mechanism for controlling stormwater runoff for the purposes of reducing downstream erosion and flooding, and mitigating the negative effects resulting from urbanization. Many localities have ordinances that require specific action to mitigate such potential damage. Within the broad scope of the Clean Water Act, local authorities are compelled to implement measures such as erosion and sediment control, storm water retention and detention ponds, and Best Management Practices (BMPs) to diminish the effects of land use changes. BMPs are those techniques used to control non-point source pollution.

One of the most common methods for reducing downstream flooding is the use of retention/detention facilities to temporarily impound the runoff and then release it at a lower rate to reduce the peak discharge downstream of the facility. Just how much the peak is reduced depends on the type of hydraulic outlet structure and the size of the facility. Retention and detention facilities may also double as a BMP facility. Improved water quality can be achieved by retaining runoff long enough for sediment, and the attached pollutants to settle out.

This chapter discusses the basic concepts for hydrologic analysis, locating and sizing a stormwater management pond and discussion of the various types of Best Management Practices that can be incorporated into a land development program. To clarify, in this chapter a detention pond is a dry pond, one that holds water for a short du-



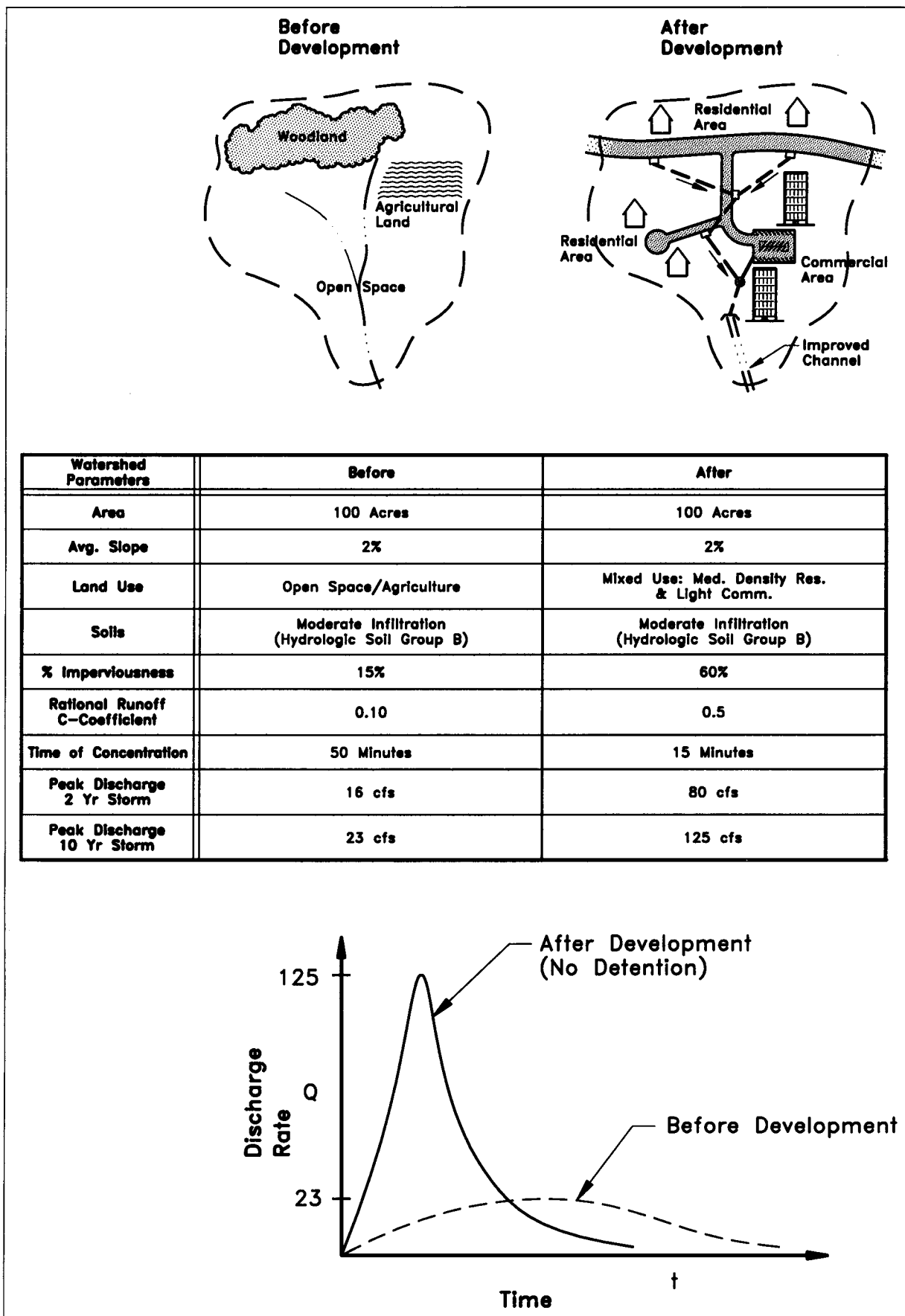


FIGURE 22.1 Hydraulic and hydrologic effects of urbanization.

ration (i.e., <72 hrs) after a storm event. After the water is released, the detention pond remains dry until the next storm. A retention pond has a permanent pool of water.

### SCS METHODOLOGY FOR COMPUTING RUNOFF

For relatively small catchments (<200 acres), the rational method (see Chapter 21) can be used to determine peak runoff discharge. However, for larger catchments, designers prefer to use more sophisticated rainfall-runoff models. Although a more sophisticated model does not necessarily provide greater accuracy and better results, there is greater flexibility for calibrating the model to local observations. One such hydrologic model, developed by the National Resource Conservation Service, formerly the Soil Conservation Service (SCS), is widely accepted, well documented, and available for use with microcomputers. Underlying fundamentals of this method are found in the *National Engineering Handbook*, Chapter 4, Hydrology (NEH-4) and the computer program documented in Technical Release 20 (TR-20). Recent documentation of SCS methods is found in Technical Release 55 (TR-55), *Urban Hydrology for Small Watersheds*. These documents are available from the Government Printing Office, Washington, D.C. Because the SCS method is routinely used for stormwater management design, a brief discussion of the primary components and principals are provided in the following sections.

#### Rainfall

Rainfall events are described in terms of depth, duration and frequency of occurrence. A plot of the rainfall distribution with time is a hyetograph. Most retention/detention (R/D) design is based on fabricated hyetographs of definite shapes, which define the design storm. The design storm is not an actual storm of occurrence. Rather it is a fabricated storm intended to represent the characteristics of an average storm. The specifics of the design storm, such as duration, depth, and whether the peak rainfall occurs near the beginning of the storm (early peaking), in the middle of the storm (center peaking), or at the end of the storm (late peaking), are detailed in design standards of the various government reviewing agencies.

For the design of larger SWM facilities, where downstream safety is a major concern in the event of dam failure, local agencies usually require a hydrologic analysis of large storm events such as the probable maximum precipitation (PMP), the probable maximum flood, or a percentage of the PMF. The PMF is the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMP is, theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographic location at a certain time of the year.

Precipitation data is available from various publications of the National Weather Service (NWS). Isopluvial maps

(see Figures 22.2 and 22.3), such as those found in Technical Paper 40 Rainfall Frequency Atlas and Hydro 35, provide adequate information for most designs. Rain gauge data and stream gauge data may be available, but for most small to moderately-sized R/D facilities, synthetic design hyetographs suffice.

The SCS has developed four synthetic rainfall distributions that are indicative of the rainfall intensities inherent to geographic regions of the U.S. These four standard rainfall distributions, labeled Type I, Type Ia, Type II, and Type III, have been developed from the Weather Bureau's Rainfall Frequency Atlases (e.g., TP-40). Since most rainfall data is reported on a 24-hr basis, the SCS used 24 hours as the duration for these distributions. The location of the peak rainfall intensity in each storm is intended to mimic the location of the peak intensity for the particular region of the U.S. For example, peak intensities for Type I and Ia storms occur around 8 hours, similar to the storms in the far western part of the U.S. Type II and III storms, however, have peak intensities occurring around the midpoint of the duration. Specific geographical areas are shown in Figure 22.4.

#### Runoff Volume Using Runoff Curve Numbers

The fundamental equation in the SCS hydrologic method for computing the volume of runoff from a catchment area is

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \quad (22.1)$$

where  $Q$  is the runoff in inches over the entire watershed,  $P$  is rainfall (in.),  $S$  is the potential maximum retention (i.e., rain not converted to runoff) after runoff begins (in.) and  $I_a$  is the initial abstraction that accounts for losses before runoff begins (losses mainly include interception, infiltration, and surface storage).

Values for  $I_a$  depend on characteristics of the soil, land use, and vegetation. Empirically,  $I_a$  is taken as equal to  $0.2S$  where  $S$  is given as

$$S = \frac{1000}{CN} - 10 \quad (22.2)$$

and  $CN$  is the curve number that relates the runoff to land characteristics. The curve number is analogous to the runoff coefficient used in the rational method. It converts the mass rainfall to runoff and is based on such factors as the hydrologic soil group (HSG), cover type, hydrologic conditions, and antecedent moisture conditions. Using the these relationships, the runoff can be expressed in terms of  $S$ :

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad (22.3)$$

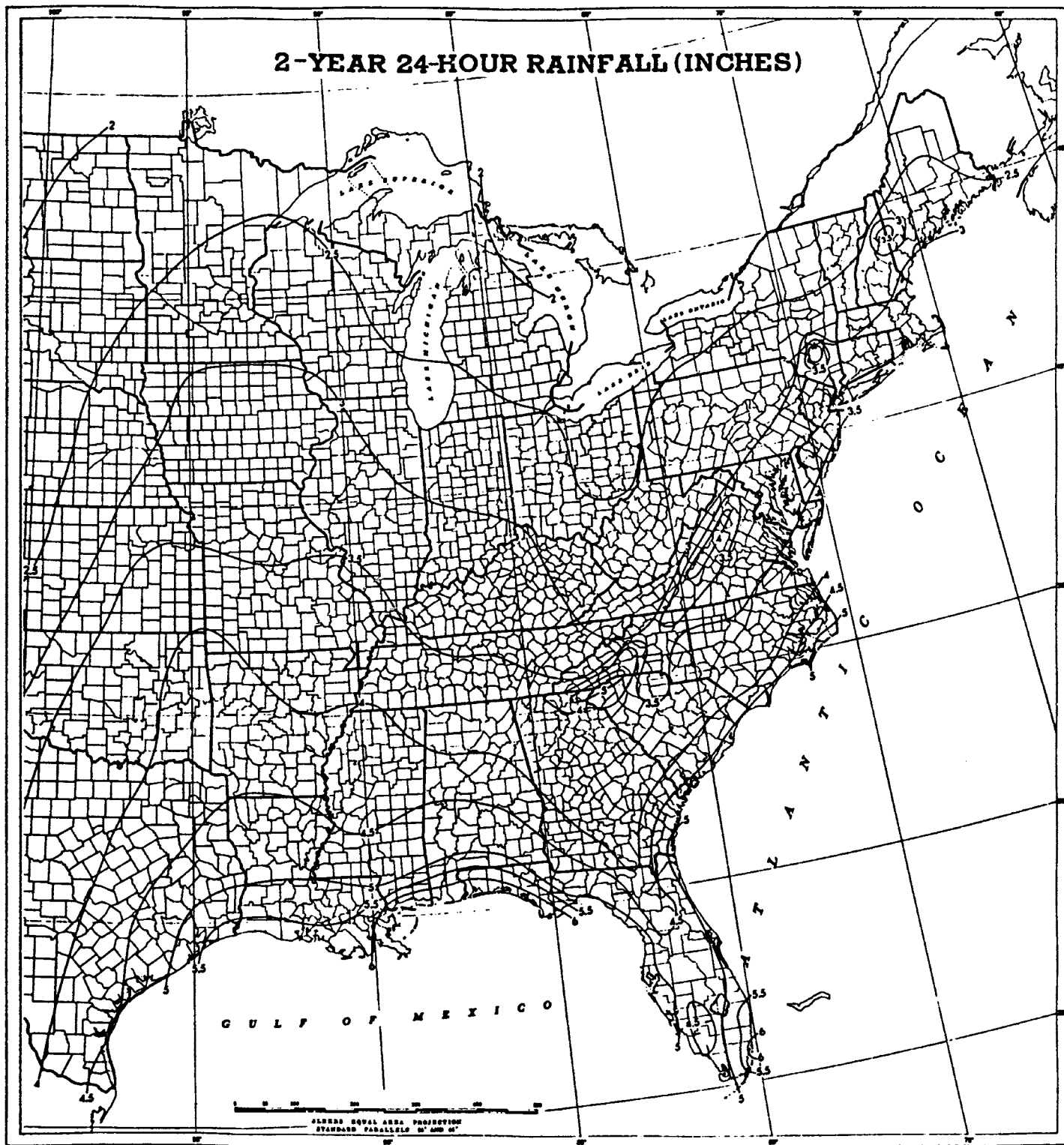


FIGURE 22.2 Two-year 24-hour rainfall map.

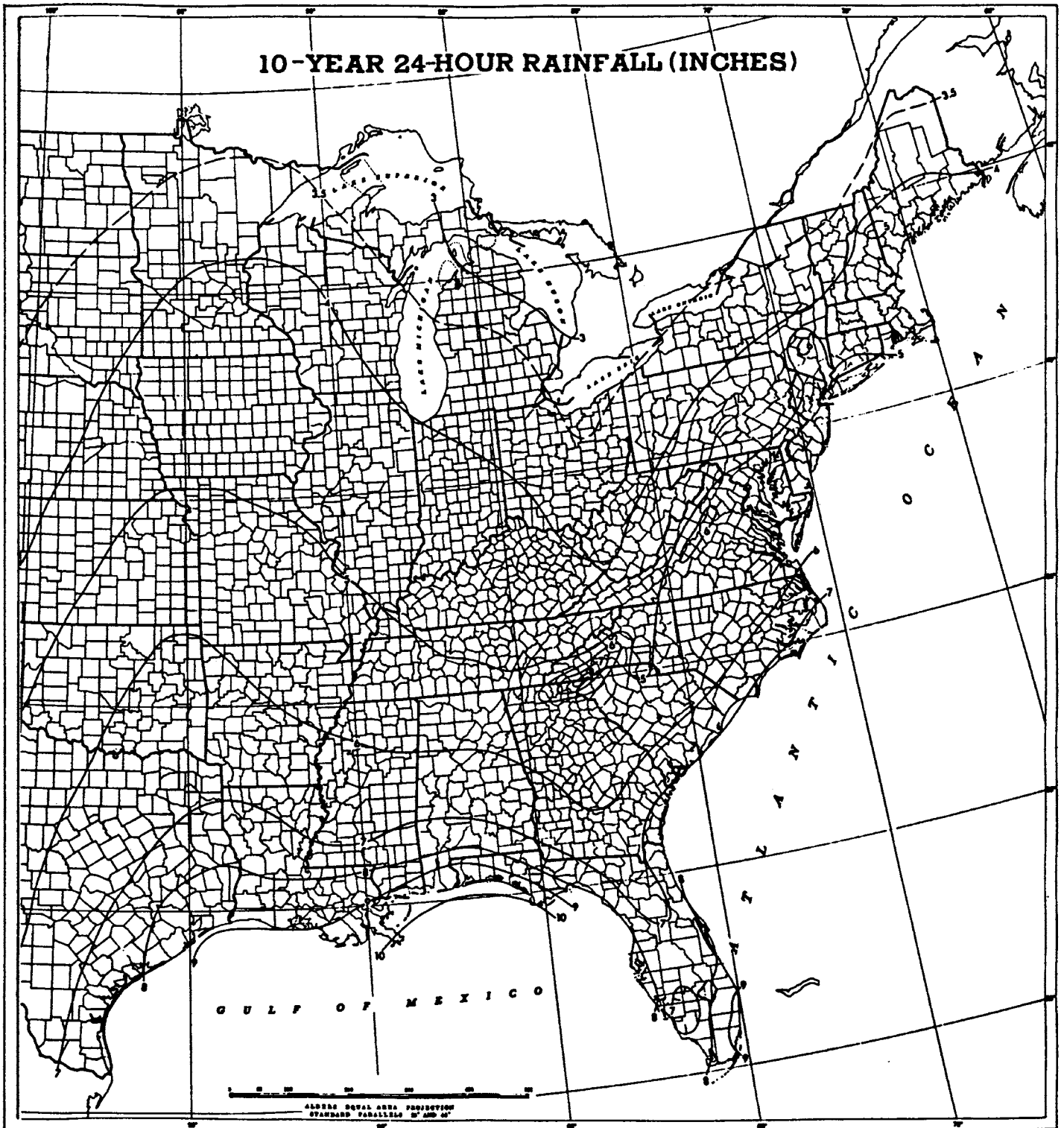
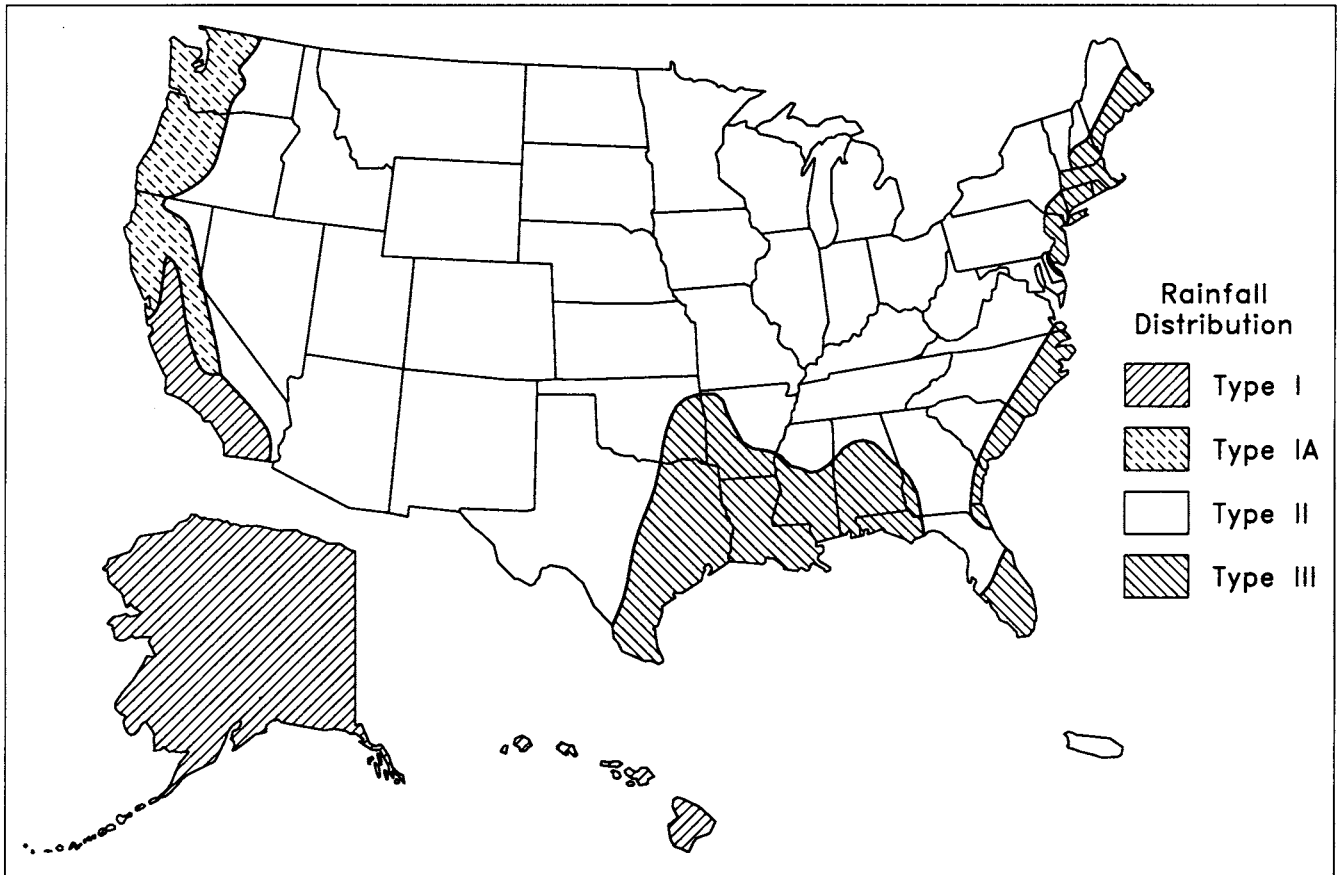


FIGURE 22.3 Ten-year 24-hour rainfall map.



**FIGURE 22.4** Approximate geographic boundaries for SCS rainfall distributions. (Source: USDA, TR-55.)

In part, the  $CN$  for a particular soil depends on the HSG classification. Soils are divided into 4 hydrologic soil groups, A, B, C, and D, according to their minimum infiltration rates. Soils classified in hydrologic group A generally have high infiltration rates, whereas the HSG D has the lowest infiltration rates. The cover type describes the surface of the catchment, such as type and denseness of vegetation and impervious or semi-impervious pavements. It is determined from field reconnaissance, aerial photographs, and land use maps. Hydrologic condition is a measure of the effects of the cover type on infiltration and runoff. Antecedent runoff condition is an index of the runoff potential of the soil before a storm event. It is an attempt to account for the variation in  $CN$  at the site from storm to storm.

Table 22.1 presents  $CN$  values for several types of soils and cover types. It should be noted that these  $CN$  values are based upon the an average antecedent runoff condition. The County Soil Survey, published by the SCS, contains soil classification information. The SCS guidance documents should be consulted if the runoff conditions are other than average.

Once the hydrologic soil group and the cover type and antecedent runoff condition have been determined, a weighted  $CN$  can be found by determining the areal cov-

erage of each set of conditions and consulting a SCS curve number table. Figure 22.5 shows the HSG groups overlain on a land use map. A soils map is used to identify the soil series which is then converted to a hydrologic soil group. This map was created by tracing the HSG map onto the land use map. The worksheet shown in Figure 22.6 is used to tabulate the data and determine the composite  $CN$ .

### Time of Concentration

The time of concentration as it applies to hydrology is the time for runoff to travel from the *hydraulically* most remote point of a watershed to the outlet. The SCS method for determining  $t_c$  consists of computing the travel times associated with runoff over three distinct types of flowpaths as described below.

1. Sheet flow is the initial phase of runoff characterized as flow over plane surfaces. The flow depth is very shallow ( $<0.1$  ft), and consequently the Manning's roughness coefficient is modified to reflect the increased effects of drag from surface irregularities. Sheet flow, no matter the type of surface, is assumed to occur for distances less than 300 feet. The travel time is given as

**TABLE 22.1-a Runoff Curve Numbers for Urban Areas<sup>1</sup>**

COVER DESCRIPTION	CURVE NUMBERS FOR HYDROLOGIC SOIL GROUP				
	AVERAGE % IMPERVIOUS AREA <sup>2</sup>	A	B	C	D
<i>Fully developed urban areas (vegetation established)</i>					
Open space (lawns, parks, golf courses, cemeteries, etc.) <sup>3</sup>					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
.....					
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of way)		98	98	98	98
Streets and roads:					
Paved: curbs and storm sewers (excluding right-of-way)		83	89	92	93
Paved: open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
.....					
Western desert urban areas:					
Natural desert landscaping (pervious areas only) <sup>4</sup>		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-in. sand or gravel mulch and basin borders)		96	96	96	96
.....					
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
.....					
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
.....					
<i>Developing urban areas</i>					
Newly graded areas (pervious areas only, no vegetation) <sup>5</sup>		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 22.1-c).					

<sup>1</sup>Average runoff condition and  $I_a = 0.2S$ <sup>2</sup>The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CNs for other combinations of conditions may be computed using Figures 22.3 or 22.4.<sup>3</sup>CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.<sup>4</sup>Composite CNs for natural desert landscaping should be computed using Figures 22.3 or 22.4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CNs are assumed equivalent to desert shrub in poor hydrologic condition.<sup>5</sup>Comparable CNs to use for the design of temporary measures during grading and construction should be computed using Figures 22.3 or 22.4, based on the degree of development (impervious area percentage) and the CNs for the newly graded pervious area.

Source: USDA, TR-55.

**TABLE 22.1-b Runoff Curve Numbers<sup>1</sup> (Continued)**

COVER TYPE	HYDROLOGIC CONDITION <sup>2</sup>	CURVE NUMBERS FOR HYDROLOGIC SOIL GROUP			
		A <sup>3</sup>	B	C	D
Pasture, grassland, or range-continuous forage for grazing <sup>2</sup>	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow-continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush-brush-weed-grass mixture with brush the major element. <sup>3</sup>	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30	48	65	73
Woods-grass combination (orchard or tree farm). <sup>5</sup>	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. <sup>6</sup>	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads-buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

<sup>1</sup>Average runoff condition, and  $I_a = 0.2 S$ .

<sup>2</sup>Poor: <50% ground cover or heavily grazed with no mulch.

Fair: 50 to 75% ground cover and not heavily grazed.

Good: >75% ground cover and lightly or only occasionally grazed.

<sup>3</sup>Poor: <50% ground cover.

Fair: 50 to 75% ground cover.

Good: >75% ground cover.

<sup>4</sup>Actual curve number is less than 30: use  $CN = 30$  for runoff computations.

<sup>5</sup> $CNs$  shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the  $CNs$  for woods and pasture.

<sup>6</sup>Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.

Fair: Woods are grazed but not burned, and some forest litter covers the soil.

Good: Woods are protected from grazing, and litter and brush adequately over the soil.

Source: USDA, TR-55.

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \quad (22.4)$$

which is the kinematic solution to Manning's equation. In Eq. (22.4)  $T_t$  represents the travel time in hours,  $n$  is the effective Manning's  $n$  as given in Table 22.2,  $L$  is the flow length in feet (<300 ft),  $P_2$  is the 2-yr 24-hr rainfall (in.) given in Figure 22.3, and  $s$  is the slope of the hydraulic grade line, which is assumed to be the same as the average land slope (ft/ft).

2. Shallow concentrated flow occurs after sheet flow. The travel time for shallow concentrated flow is

$$T = \frac{L}{3600 V} \quad (22.5)$$

where  $V =$

- 16.1345 (s)<sup>0.5</sup> for unpaved surfaces
- 20.3282 (s)<sup>0.5</sup> for paved surfaces

where  $V$  is the average runoff velocity (fps),  $L$  is the flow length (ft) and the travel time  $T$  is in hours.

3. Eventually runoff collects into defined open channels that are, according to the SCS, visible on aerial

**TABLE 22.1-c Runoff Curve Numbers<sup>1</sup> (Continued)**

COVER TYPE	COVER DESCRIPTION	HYDROLOGIC CONDITION <sup>2</sup>	CURVE NUMBERS FOR HYDROLOGIC SOIL GROUP			
			A <sup>3</sup>	B	C	D
Herbaceous-mixture of grass, weeds, and low-growing brush, with brush the minor element.		Poor		80	87	93
		Fair		71	81	89
		Good		62	74	85
Oak-aspen-mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.		Poor		66	74	79
		Fair		48	57	63
		Good		30	41	48
Pinyon-juniper-pinyon, juniper, or both; grass understory.		Poor		75	85	89
		Fair		58	73	80
		Good		41	61	71
Sagebrush with grass understory.		Poor		67	80	85
		Fair		51	63	70
		Good		35	47	55
Desert shrub-major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.		Poor	63	77	85	88
		Fair	55	72	81	86
		Good	49	68	79	84

<sup>1</sup>Average runoff condition, and  $I_a = 0.2 S$ . For range in humid regions, use Table 22.2-c.

<sup>2</sup>Poor: <30% ground cover (litter, grass, and brush overstory).

Fair: 30 to 70% ground cover.

Good: >70% ground cover.

<sup>3</sup>Curve numbers for group A have been developed only for desert shrub.

Source: USDA, TR-55.

photographs, appear as blue lines on USGS quadrangle sheets, or where surveyed cross-section information begins. Travel time in open channels is determined from the average velocity at bank-full flow. Manning's equation for open channel velocity is

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \quad (22.6)$$

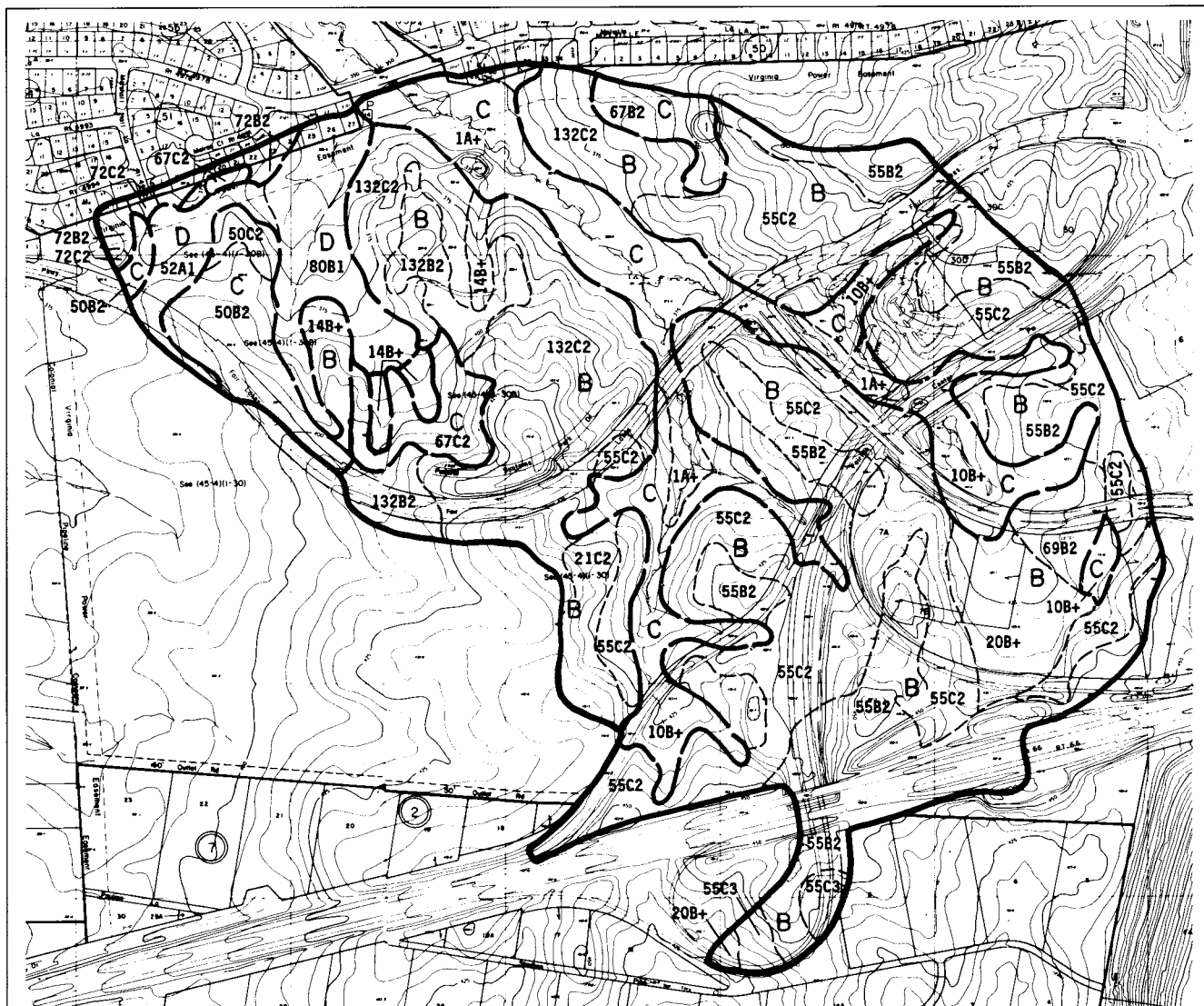
where  $V$  is the average velocity in fps,  $S$  is the slope of the hydraulic gradient (channel bed slope in the case of uniform flow) (ft/ft), and  $n$  is Manning's roughness coefficient (values are found in most hydraulic handbooks). The hydraulic radius,  $R$ , is defined as the cross-sectional flow area divided by the wetted perimeter. The travel time for open channel flow is determined from Eq. (22.5) using the velocity obtained from Manning's equation.

When computing the various components of the time of concentration, the values of the velocities must be carefully reviewed to determine if they are realistic. Many assumptions made about the land characteristics (e.g., uniform ground slope, vegetation height, and stream channel geometry) may give unrealistic values. For example, in natural open channels the critical velocity should be considered as the limiting velocity.

### Hydrographs

Runoff from a watershed is graphically shown by a hydrograph, which is a plot of the discharge as a function of time. In the most simplistic case, a hydrograph has a rising limb, which reflects rainfall characteristics, a crest segment, and a recession curve, which reflects watershed characteristics. A hydrograph may be classified as a natural hydrograph, which is derived from observed data from a stream flow gauge, or it may be classified as a synthetic hydrograph,



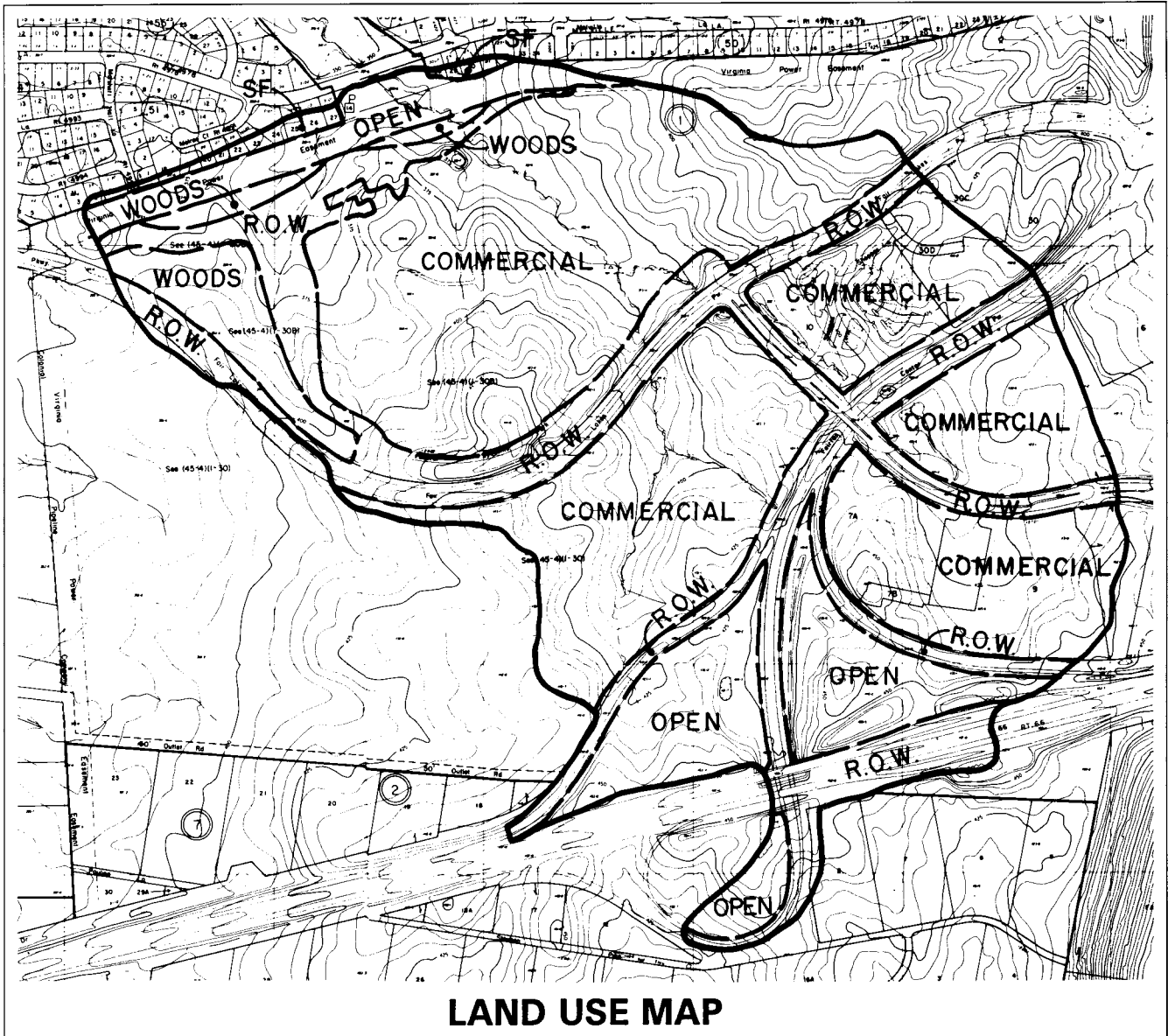


**HYDOLOGIC SOILS GROUPS**

**CONVERSION OF SOIL TYPES TO SOIL GROUPS**

SOIL TYPE	SOIL NAME	SOIL GROUP	SOIL TYPE	SOIL NAME	SOIL TYPE
1	Mixed Alluvial	C	52	Elbert	D
10	Glenville	C	55	Glenelg	B
14	Manassas	B	67	Penn	C
20	Meadowville	B	69	Enon	B
21	Manor	B	72	Bucks	B/C
32	Mayodan	B	80	Croton	D
50	Iredell-Mecklenburg	C/D			

FIGURE 22.5 Hydrologic soils designation on topographic map.



**FIGURE 22.5** (Continued) Land use designations on a topographic map.

which is derived from presumed characteristics related to the rainfall and the watershed. The area under a runoff hydrograph represents the volume of runoff from the watershed.

The following parameters define the timing aspects of the hydrograph (see Figure 22.7):

- *Time to peak* ( $t_p$ ); the time from beginning of runoff to the peak
- *Lag time* ( $t_l$ ); the time from center of mass of rainfall excess to the peak rate of runoff

- *Time of Concentration* ( $t_c$ ); the time of equilibrium of the watershed, where outflow is equal to net inflow. On a hydrograph,  $t_c$  is the time from the end of excessive rainfall to the inflection point on the recession limb.

- *Time base* ( $T_b$ ); total duration of the direct runoff hydrograph

As might be deduced from the hydrograph sketch and the above definitions, the hydrograph shape is affected by the intensity, duration, and distribution (both temporally

Worksheet 2: Runoff curve number and runoff

Project \_\_\_\_\_ By \_\_\_\_\_ Date \_\_\_\_\_

Location \_\_\_\_\_ Checked \_\_\_\_\_ Date \_\_\_\_\_

Circle one: Present Developed \_\_\_\_\_

**1. Runoff curve number (CN)**

Soil name and hydrologic group  (appendix A)	Cover description  (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN <sup>1</sup>			Area  <input type="checkbox"/> acres <input type="checkbox"/> mi <sup>2</sup> <input type="checkbox"/> %	Product of CN x area
		Table 15-1	Fig. 2-3 <sup>2</sup>	Fig. 2-4 <sup>2</sup>		
Totals =						

$$\text{CN (weighted)} = \frac{\text{total product}}{\text{total area}} = \frac{\quad}{\quad} = \quad ;$$

Use CN =

**2. Runoff**

Frequency..... yr  
 Rainfall, P (24-hour).....in  
 Runoff, Q .....in

Storm #1	Storm #2	Storm #3

<sup>1</sup>Use only one CN source per line

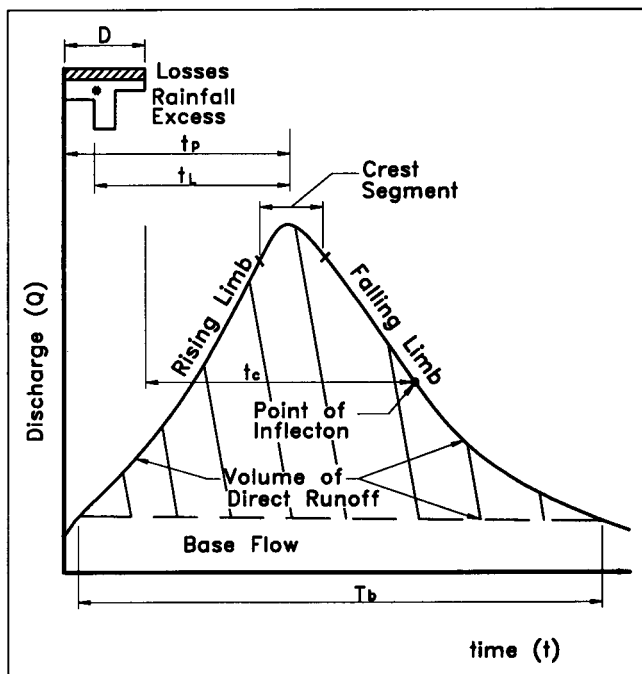
<sup>2</sup>Figure 2-3 and 2-4 found in the TR-55 manual relate to connected impervious areas. Refer to TR-55 manual for further explanation

**FIGURE 22.6** Runoff curve number worksheet. (Source: TR-55.)

**TABLE 22.2 Roughness Coefficient for Sheet Flow**

SURFACE DESCRIPTION	$n^1$
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.005
Cultivated soils:	
Residue over $\leq 20\%$	0.06
Residue cover $> 20\%$	0.17
Grass:	
Short grass prairie	0.15
Dense grasses <sup>2</sup>	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods <sup>3</sup>	
Light underbrush	0.40
Dense underbrush	0.80

<sup>1</sup>The  $n$  values are a composite of information compiled by Engman (1986).  
<sup>2</sup>Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.  
<sup>3</sup>When selecting  $n$ , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.



**FIGURE 22.7** Elements of a hydrograph.

and spatially) of rainfall, by the size and shape of the watershed, and by factors that influence the time of concentration (land slope, channel length, and land cover/use). A short time of concentration results in a higher peak discharge rate and a shorter time to peak, while a long time of concentration results in a lower peak discharge rate and longer time to peak.

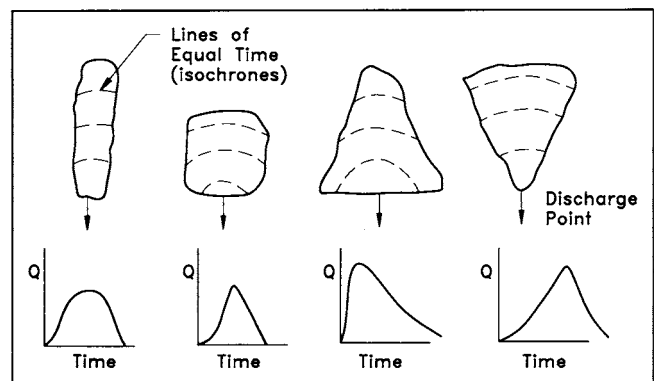
Generally, land development activities will decrease the time of concentration and increase the imperviousness for a basin. These changes increase the peak discharge, the time to peak, and the total runoff volume, and it is these increases that must be mitigated with storm water management methods. These concepts are illustrated in Figure 23.8.

**Unit Hydrographs**

A unit hydrograph (UH) is defined as a runoff hydrograph generated from a unit depth of rainfall excess occurring at a uniform rate over a specified duration of time, uniformly distributed over the watershed. The area under a unit hydrograph represents one unit of runoff depth over the entire watershed. Each unit hydrograph has a specific duration (i.e., time base), which represents the duration of the rainfall excess. Therefore, a  $D$ -hour unit hydrograph is defined as the hydrograph that results from a storm with a constant rainfall excess of 1 in. over a duration of  $D$  hours (see Figure 22.9).

The physical features of a watershed vary very little from storm to storm. Therefore, in unit hydrograph theory, a storm event of equal duration but different intensity produces a direct runoff hydrograph with an equal base length and similar shape as that of a unit hydrograph. If the ordinates of the direct runoff hydrograph are proportional to runoff volume, multiplying the ordinates of a unit hydrograph by the rainfall intensity generates a hydrograph corresponding to that intensity.

For more complex unit hydrographs, generate direct runoff hydrographs through multiplication-translation-



**FIGURE 22.8** Effect of basin shape and  $t_c$  on hydrograph shape. Courtesy of Martin P. Wanielista, *Hydrology and Water quality Control*, New York: John Wiley & Sons. Reprinted by permission of John Wiley & Sons, Inc.

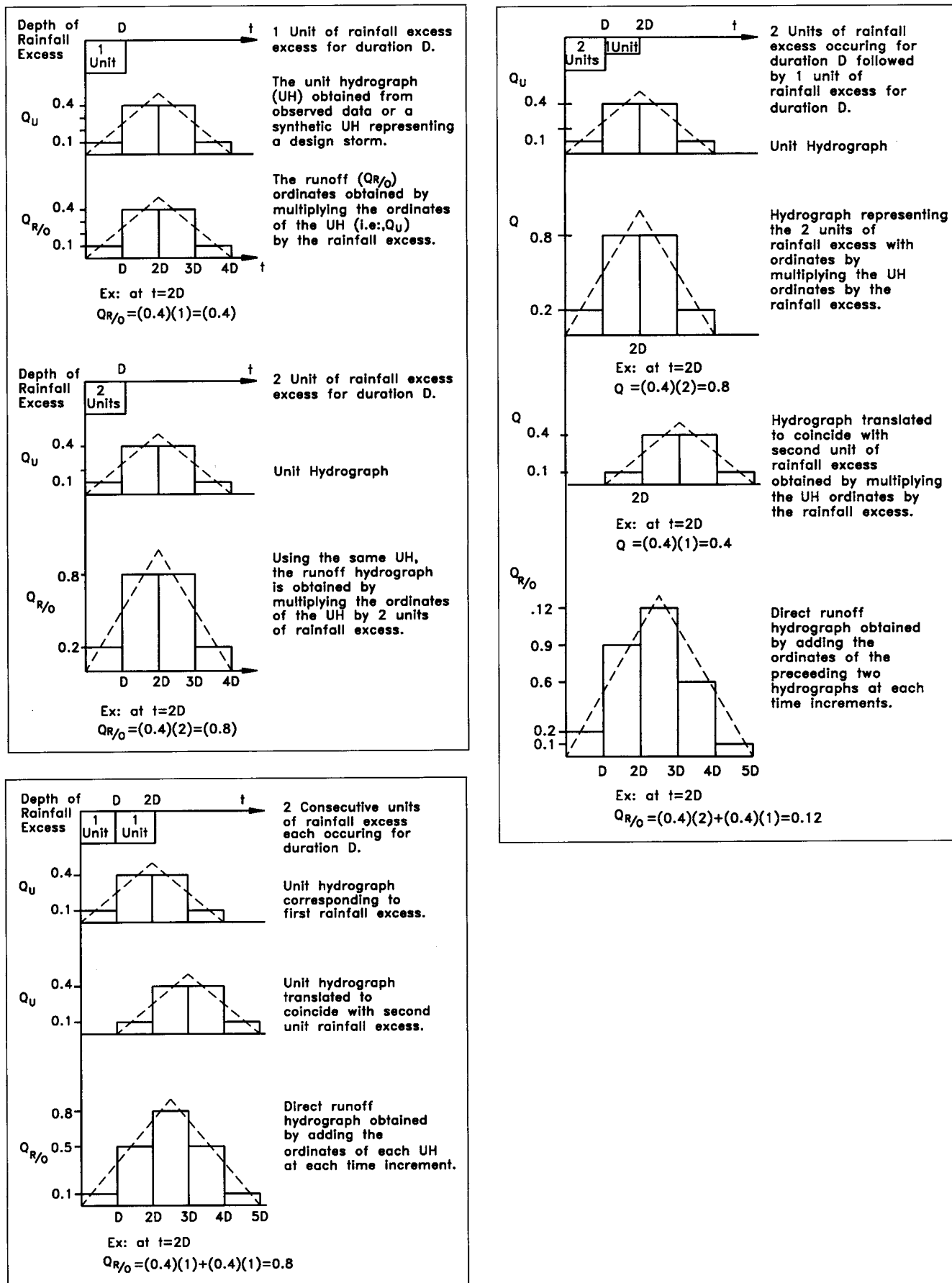


FIGURE 22.9 Direct runoff from a unit hydrograph.

addition procedures (convolution) to obtain the direct runoff hydrograph. This can only be done if the assumptions of linearity are valid—that is, the time base remains constant regardless of the runoff depth. Figure 22.9 shows how to obtain a direct runoff hydrograph using multiplication-addition procedure.

Mathematically, the ordinates for the direct runoff hydrograph for a complex storm pattern are found by

$$q_1 = \rho_1 u_1 \tag{22.7}$$

$$q_2 = \rho_2 u_1 + \rho_1 u_2$$

$$q_3 = \rho_3 u_1 + \rho_2 u_2 + \rho_1 u_3$$

$$q_4 = \rho_4 u_1 + \rho_3 u_2 + \rho_2 u_3 + \rho_1 u_4$$

$$q_5 = \rho_5 u_1 + \rho_4 u_2 + \rho_3 u_3 + \rho_2 u_4 + \rho_1 u_5$$

where  $q$  is the runoff hydrograph ordinate,  $p$  is the rainfall excess ordinate and  $u$  is the unit hydrograph ordinate. If  $n_p$  = the number of rainfall excess ordinates and  $n_u$  = the number of unit hydrograph ordinates, the number of direct runoff hydrograph ordinates  $n_r = n_p + n_u - 1$ .

In matrix format, Eq. (22.7) is written as

$$[Q] = [P][U] \tag{22.8}$$

$$\begin{bmatrix} q_1 \\ q_2 \\ q_3 \\ q_4 \\ q_5 \\ \vdots \\ \vdots \\ \vdots \end{bmatrix} = \begin{bmatrix} \rho_1 & 0 & 0 & 0 & 0 & 0 & \dots \\ \rho_2 & \rho_1 & 0 & 0 & 0 & 0 & \dots \\ \rho_3 & \rho_2 & \rho_1 & 0 & 0 & 0 & \dots \\ \rho_4 & \rho_3 & \rho_2 & \rho_1 & 0 & 0 & \dots \\ \rho_5 & \rho_4 & \rho_3 & \rho_2 & \rho_1 & 0 & \dots \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \end{bmatrix} \begin{bmatrix} u_1 \\ u_2 \\ u_3 \\ u_4 \\ u_5 \\ \vdots \\ \vdots \\ \vdots \end{bmatrix}$$

Generally, synthetic unit hydrographs are used to estimate the rainfall-runoff pattern for the design storm for a watershed. Synthetic unit hydrographs are often published in state or local design guides. If local design guides do not specify a hydrograph method, one of several synthetic hydrograph methods may be used. Popular methods for developing synthetic unit hydrographs include the SCS method and the Modified Rational formula, Snyder’s unit hydrograph, Clark’s unit hydrograph and numerous others.

**The SCS Dimensionless Unit Hydrograph**

The SCS has developed natural unit hydrographs from data collected from catchments of widely varied size and locations and then converted to an “average” synthetic curvilinear dimensionless unit hydrograph shown in Figure 22.10. To simplify calculations, the curvilinear UH may be approximated by a triangular UH with similar characteristics. The base of the triangular UH is 8/3 of the time-to-peak compared with 5 for the curvilinear hydrograph. However, the area under the rising limb of both hydrographs is iden-

tical. The unit hydrographs are normalized (made dimensionless) by expressing the discharge as a fraction of the peak discharge ( $q/q_p$ ), and the time values as a fraction of the time to peak ( $t/T_p$ ).

With respect to the triangular UH the total runoff volume,  $Q$  in inches, is

$$Q = \frac{1}{2} q_p(T_p + T_r) \tag{22.9}$$

where  $q_p$  is the peak discharge (in./hr),  $T_p$  is the time to peak (hr), and  $T_r$  is the recession time (hr), which can be re-written as

$$q_p = \frac{484AQ}{T_p} \tag{22.10}$$

where  $q_p$  is the peak discharge for the unit hydrograph,  $A$  is drainage area in square miles, and 484 is included to place 3/8 of the area under the hydrograph under the rising limb. Changing the rising side of the UH to reflect watershed topography requires a change in the 484 shape factor. The shape factor has been known to vary from 600 for mountainous regions to 300 in very flat swampy areas. The local SCS office should be contacted to determine if a regional shape factor exists.

From the relationship of the triangular hydrograph to the storm duration,  $T_p$  can be expressed in terms of duration of unit rainfall excess and the time of concentration as

$$T_p = \frac{D}{2} + 0.6t_c = \frac{2}{3} t_c \tag{22.11}$$

With proper substitution into Eq. (22.10),  $q_p$  can be expressed in terms of  $t_c$  as

$$q_p = \frac{726AQ}{t_c} \tag{22.12}$$

**EXAMPLE 1:** Determine the SCS synthetic UH for a predominantly commercial catchment with the following characteristics: area = 200 ac, average land slope = 4%,  $t_c$  = 20 min and HSG B.

1. From Table 22.1 the CN for commercial land use and HSG B is 92
2. From Eq. (23.12), the peak discharge for the UH is

$$\begin{aligned} q_p &= \frac{726(200 \text{ acre})(1 \text{ in.})}{(640 \text{ acre/mi}^2) \left( \frac{45 \text{ min}}{60 \text{ min/hr}} \right)} \tag{22.13} \\ &= 303 \text{ cfs} \end{aligned}$$

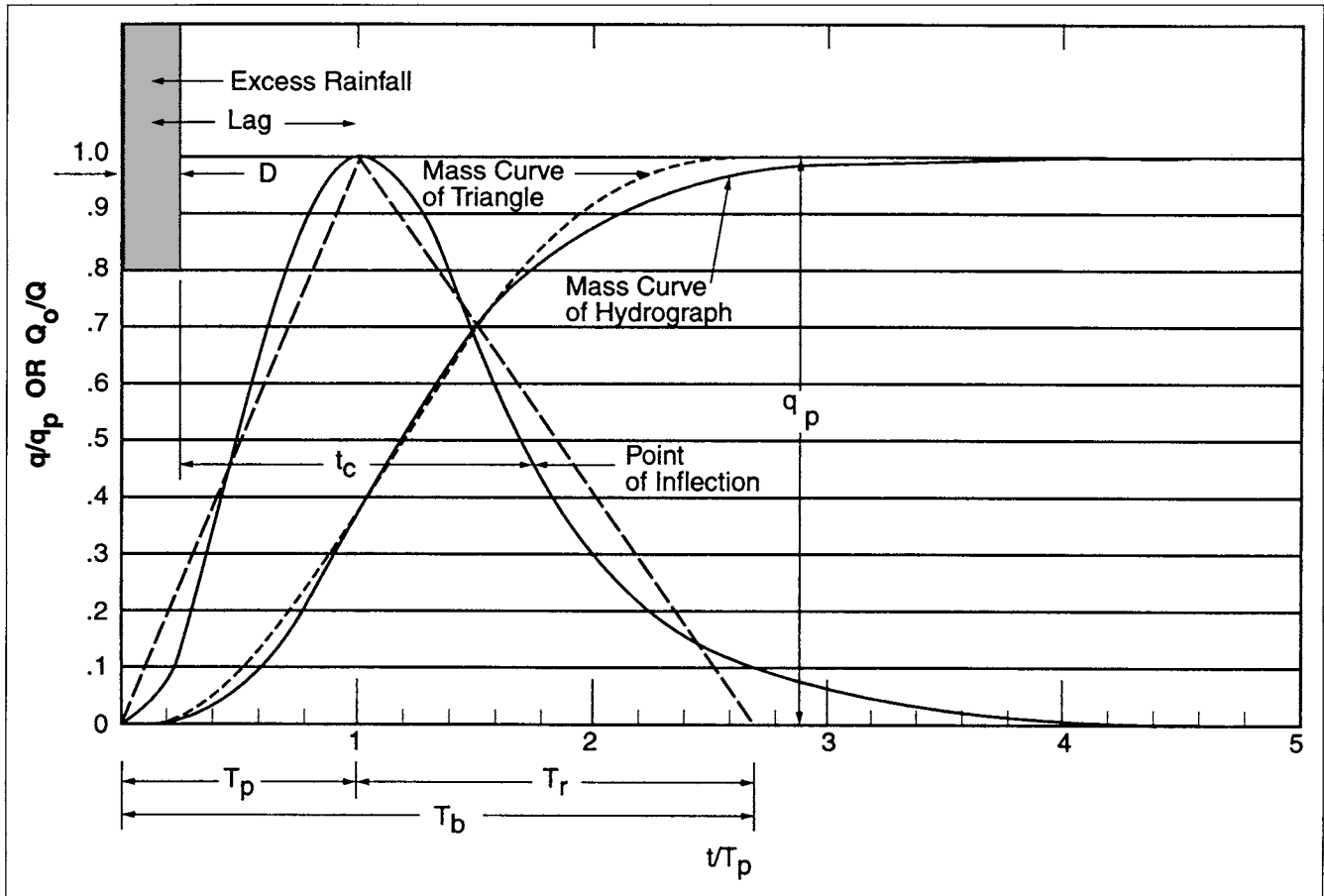


FIGURE 22.10 Dimensionless curvilinear UH with SCS's triangular UH. (Source: USDA, NEH-4.)

3. The time-to-peak is

$$t_p = \frac{2}{3} t_c = \frac{2}{3} (20 \text{ min}) = 13.3 \text{ min} \quad (22.14)$$

4. The time base is

$$t_b = \frac{8}{3} t_p = \frac{8}{3} (13.4 \text{ min}) = 35.7 \text{ min} \quad (22.15)$$

5. The triangular hydrograph is shown in Figure 22.11.

**Computer Models for Rainfall-Runoff Relationships**

The advent of microcomputers has made hydrologic modeling easier and quicker. Although many designers use these software programs as a “black box,” it is recommended that the user have some understanding of the hydrologic processes as well as a knowledge of the fundamental limitations of the programs. The reliability of the output from any computer program is only as good as the input data, the inherent assumptions associated with the model, as well as the numerical techniques used to simulate the model. Blind

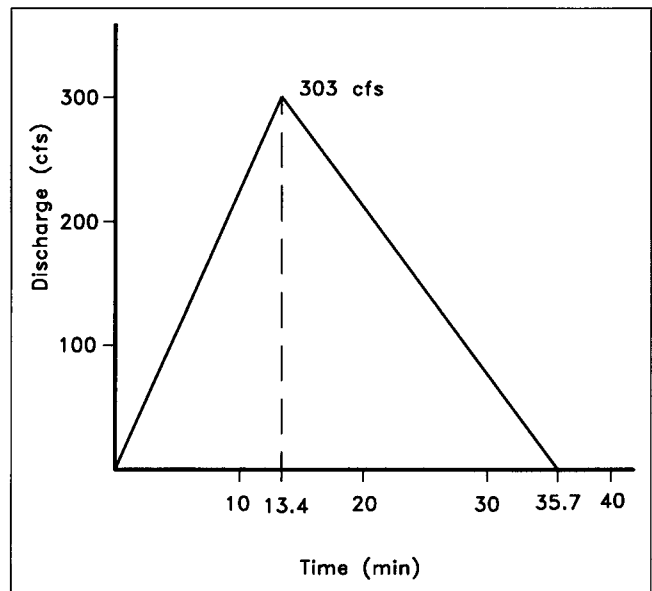


FIGURE 22.11 Synthetic triangular unit hydrograph for example 8.

application of the results can lead to poor design at best or jeopardize life and property at worst.

A number of computer models are available, well documented and supported, and powerful in terms of their ability to manipulate hydrographs. These models may be used to generate hydrographs from either synthetic or historical design storms, combine hydrographs together, and perform channel and pond routings. The type of computer model selected depends on the user, available data, and possibly any preferences by the reviewing agencies.

Some of the more popular models in use are listed in Table 22.3.

### SCS Graphical Peak Discharge

A peak discharge can be obtained from a direct runoff hydrograph. When only a peak discharge is needed, the SCS's graphical peak discharge method can be used instead of developing an entire hydrograph. SCS's graphical method for determining the peak discharge, appropriate for homogeneous (i.e., uniform  $CN$ ) watersheds and for  $0.1 \text{ hr} \leq t_c \leq 10 \text{ hrs}$ , is found by

$$q_p = q_u A_m Q F_p \quad (22.16)$$

where  $q_p$  = peak discharge (cfs),  $q_u$  = unit (hydrograph) peak discharge ( $\text{ft}^3/\text{mi}^2/\text{in.}$ ),  $A_m$  = drainage area ( $\text{mi}^2$ ),  $Q$  = runoff (in., from Eq. (22.1) and  $F_p$  = pond and swamp adjustment factor (from Table 22.4).

Before engaging this method, consider the following limitations.

- The watershed should be homogeneous with respect to the  $CN$ .
- The weighted  $CN$  should be greater than 40,  $0.1 \leq t_c \leq 10 \text{ hrs}$  and  $0.1 I_a/P \leq 0.5$ .
- The watershed should have only one main channel. If there is more than one, all channels must have nearly equal  $t_c$ s.
- The method cannot perform reservoir or channel routing.
- The  $F_p$  factor is applied only for ponds or swamps not in the  $t_c$  flowpath.

Compute the peak discharge for a watershed as follows:

1. From appropriate maps obtain the 24-hr rainfall,  $P$ , for the design storm.
2. Determine the  $CN$  for the watershed.
3. Determine  $I_a$  from

$$I_a = 0.2 \left( \frac{1000}{CN} \right) - 10 \quad (22.17)$$

4. The unit peak discharge ( $q_u$ ) in cubic feet per square mile per inch of runoff is determined from

$$\log(q_u) = C_0 + C_1 \log(t_c) + C_2 (\log(t_c))^2 \quad (22.18)$$

where coefficients for  $C_0$ ,  $C_1$ , and  $C_2$  are given in Table 22.5 for  $I_a/P$  ratios. Figure 22.12 shows the graphical representation of Eq. (23.18) for the SCS type II rainfall distribution.

5. Determine the pond factor,  $F_p$ , if applicable.
6. Use Eq. (22.16) to find the peak discharge.

## DESIGN FOR STORM WATER QUANTITY CONTROL

### Introduction

Many jurisdictions mandate that the peak, post-development runoff rate from a site not exceed the peak, pre-development runoff rate for a particular design storm. This requirement is commonly met through the use of storm water management ponds, which detain surface water runoff and release it at a slow rate, thereby reducing flooding potential, minimizing erosion and enhancing water quality. The majority of the discussion in this section will be directed to design of surface storm water management facilities, but the concepts can easily be applied to other types of detention facilities (e.g., underground SWM facilities).

Stormwater management ponds can take the form of a detention pond or a retention pond. A detention pond, or "dry pond," is used for the temporary impoundment of surface water runoff. A retention pond, or "wet pond," maintains a permanent pool of water, with additional storage volume above the permanent pool for detaining runoff. A storm water management pond usually consists of an impoundment area, a dam embankment, a principal spillway structure, and an emergency spillway. The impoundment area may be a natural stream valley and dam embankment, a completely excavated impoundment, or some combination of the two. The principal spillway usually consists of an outlet conduit through the embankment, draining to the outfall channel. Often the principal spillway incorporates a spillway "riser" structure, with a combination of orifice openings and weirs to regulate the amount of flow passing through it. The emergency spillway may be a channel cut around the dam embankment, or an overflow section built into the embankment.

The basic steps for designing a stormwater management pond for water quantity control purposes are:

1. Determine pond location and pond type (e.g., wet or dry pond).
2. Develop inflow hydrographs for the design storms. Localities may mandate that the pond must attenuate peak discharges for several design storms.
3. Estimate the amount of storage required for each of the design storms.
4. Grade the pond to accommodate the necessary storage and develop stage-storage-discharge curves.



**TABLE 2.2.3 Summary of Selected Computer Runoff Models**

<b>ATTRIBUTE (1)</b>	<b>MODEL: DR3M-QUAL (2)</b>	<b>HSPF (3)</b>	<b>ILLUDAS (4)</b>	<b>PENN STATE (5)</b>	<b>STATISTICAL (6)</b>	<b>STORM (7)</b>	<b>SWMM (8)</b>	<b>TR55 (9)</b>	<b>HEC-1 (10)</b>
Sponsoring agency	USGS	EPA	Ill. State Water Survey	OWRT and City of Phil. <sup>a</sup>	EPA	HEC	EPA	SCS	HEC
Simulation type <sup>b</sup>	C, SE	C, SE	SE	SE	N/A	C	C, SE	SE	Se
No pollutants	4	10	None <sup>c</sup>	NONE	Any	6	10	None	None
Rainfall/runoff analysis	Y	Y	Y	Y	N	Y	Y	Y	Y
Sewer system flow routing	Y	Y	Y	N	N/A	N	Y	Y	Y
Full, dynamic flow routing equations	N	N	N	N	N/A	N	Y <sup>d</sup>	N	N
Surcharge	Y <sup>c</sup>	N	Y <sup>c</sup>	N	N/A	N	Y <sup>d</sup>	N	N
Regulators, overflow structures, e.g., weirs, orifices, etc.	N	N	N	N	N/A	Y	Y	N	N

Special solids routines	Y	Y	N/A	N/A	N	N	Y	N/A	N/A
Storage analysis	Y	Y	Y	Y	Y <sup>i</sup>	Y	Y	Y	Y
Treatment analysis	Y	Y	N/A	N/A	Y <sup>i</sup>	Y	Y	N/A	N/A
Suitable for planning (P), design (D) <sup>a</sup>	P, D	P, D	D	D	P	P	P, D	D	D
Available on microcomputer	N	Y	Y	Y	Y <sup>h</sup>	N	Y	Y	Y
Data and personnel requirements	Medium	High	Low	Low	Medium	Low	High	Medium	Medium
Overall model complexity <sup>i</sup>	Medium	High	Low	Low	Medium	Medium	High	Low	High

Reset with permission from Water Environment Federation and American Society of Civil Engineers, Design and Construction of Urban Stormwater Management Systems, 1992.

<sup>a</sup>Currently supported by Penn State University.

<sup>b</sup>Y = yes, N = no, N/A = not applicable, C = continuous simulation, SE = single event simulation.

<sup>c</sup>Undocumented quality routines added during applications.

<sup>d</sup>Full dynamic equations and surcharge calculations only in Extran Block of SWMM.

<sup>e</sup>Surcharge simulated by storing excess inflow at upstream end of pipe. Pressure flow not simulated.

<sup>f</sup>Storage and treatment analyzed analytically. See references in Section 3.7.9.

<sup>g</sup>See Section 3.3.

<sup>h</sup>See Driscoll et al. 1989.

<sup>i</sup>General requirements for model installation, familiarization, data requirements, etc. To be interpreted only very generally.

<sup>j</sup>Reflection of general size and overall model capabilities. Note that complex models may still be used to simulate very simple systems with attendant minimal data requirements. Surcharge row, model column = Y<sup>e</sup>, illudus column = Y<sup>e</sup>, available column = Y<sup>i</sup>.

**TABLE 2.4 Adjustment Factor ( $F_p$ ) for Pond and Swamp Areas that are Spread Throughout the Watershed**

PERCENTAGE OF POND AND SWAMP AREAS	$F_p$
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

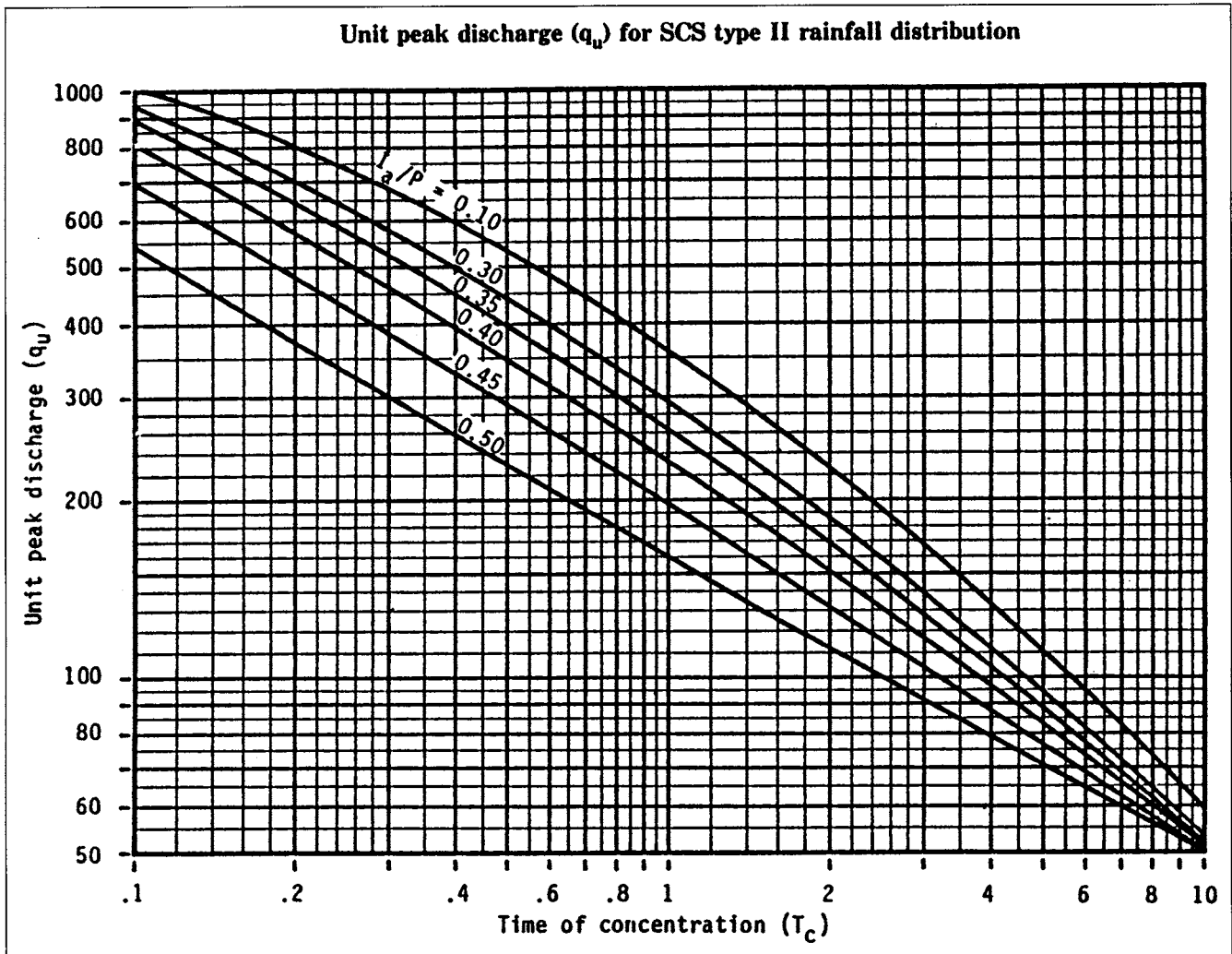
Source: USDA, TR-55.

5. Determine maximum release rate based on local design standards, including outfall adequacy.
6. Estimate elevations for water surface for design storms and top of dam and determine feasibility with grading design.
7. Select preliminary size and configuration of principal riser structure.
8. Perform hydrologic routing computations to verify preliminary design features.
9. Based on results obtained in step 8, modify grading and outlet riser structure.
10. Determine other design requirements (e.g., trash racks, concrete cradles, impervious copes, drainage blankets) and the need for emergency overflow relief and design accordingly.

**TABLE 2.5 Coefficients for Determining  $q_u$**

RAINFALL TYPE	$I_2/P$	$C_0$	$C_1$	$C_2$
I	0.10	2.30550	-0.51429	-0.11750
	0.20	2.23537	-0.50387	-0.08929
	0.25	2.18219	-0.48488	-0.06589
	0.30	2.10624	-0.45695	-0.02835
	0.35	2.00303	-0.40769	0.01983
	0.40	1.87733	-0.32274	0.05754
	0.45	1.76312	-0.15644	0.00453
	0.50	1.67889	-0.06930	0.0
IA	0.10	2.03250	-0.31583	-0.13748
	0.20	1.91978	-0.28215	-0.07020
	0.25	1.83842	-0.25543	-0.02597
	0.30	1.72657	-0.19826	0.02633
	0.50	1.63417	-0.09100	0.0
II	0.10	2.55323	-0.61512	-0.16403
	0.30	2.46532	-0.62257	-0.11657
	0.35	2.41896	-0.61594	-0.08820
	0.40	2.36409	-0.59857	-0.05621
	0.45	2.29238	-0.57006	-0.02281
	0.50	2.20282	-0.51599	-0.01259
III	0.10	2.47317	-0.51848	-0.17083
	0.30	2.39628	-0.51202	-0.13245
	0.35	2.35477	-0.49735	-0.11985
	0.40	2.30726	-0.46541	-0.11094
	0.45	2.24876	-0.41314	-0.1150°
	0.50	2.17772	-0.36803	-0.09525

Source: USDA, TR-55.



**FIGURE 22.12** Unit peak discharge ( $q_u$ ) for SCS type II rainfall distribution. (Source: USDA, TR-55.)

### Site Location

Small retention/detention (R/D) ponds are used to attenuate peak flows and improve water quality. That is, very little recreational use and very little amenity value can be expected. The designer still needs to incorporate as many items as possible, given site conditions, economic considerations and the development program of the client, to make the facility unobtrusive and to blend in with surrounding landscape. There are numerous small residential and commercial development projects where the buildings and landscape are tastefully designed, only to have an unsightly vertical pipe in a conspicuously located detention pond.

Small R/D ponds are located on the site where they can collect the most runoff. Typically this is near the low area of the site. The ponds can be hidden using creative landscaping as long as maintenance access is maintained. In

residential areas, ponds can be an attractive nuisance, especially for children. Therefore, safety is a prime consideration in the design. Such features as gently sloping side slopes, benching the slopes, shallow depths (especially along edges) and fencing should be considered.

For larger retention ponds and lakes site location must consider factors such as:

- Is the drainage area sufficient to supply the requisite volume of water? The losses due to seepage and evaporation must be less than the runoff.
- Are the soils too porous? If the lake bed soils have high percolation rates, liners may be required.
- Are the soils suitable for a dam foundation?
- Can the excavated on-site soils be used for dam construction?

- Is there a natural constriction in the watershed that would serve as a good dam location?
- What must be the depth of facility given the sediment delivery ratio, recreational uses, losses from seepage and evaporation to sustain the prescribed water level and a natural balance of aquatic ecosystems.
- Wetlands impact.
- Surrounding land use.

The location should be integrated with the site design, either to minimize the impact on the development project or to enhance the development functionally and aesthetically (see Figure 22.13). If possible, a detention or retention facility should be located so that runoff will drain into it naturally, without requiring additional engineering measures such as storm sewers or channels to force drainage divides artificially. The most economical design for a wet or dry pond is one that requires the least amount of earthwork. If a natural valley is used for the impoundment, and

the dam is placed at a constriction in the valley, the earthwork will be minimized.

### Inflow Hydrographs

Development of inflow hydrographs consists of assembling hydrologic data relating to the design storms and the land use. A topographic map is necessary to determine the drainage divides of the watershed. It is recommended that hydrologic analysis be performed for pre-development (i.e., existing) conditions, post-development conditions and ultimate-development conditions. Hydrologic analysis for ultimate conditions is performed if a considerable amount of the watershed area is outside the development area and this offsite area is largely undeveloped. Zoning and comprehensive planning maps indicate the extent of development and projected (ultimate) land use within the watershed. If SCS methodology is used, obtain maps that show the soil series and its HSG as well as a delineation of the land cover type.

A composite map delineating the HSG and land cover types for the watershed is necessary to compile the runoff



**FIGURE 22.13** The use of a stormwater management facility as an amenity is exhibited in this permanent pond at Fair Lakes, a major mixed-use project in Fairfax Co. Extensive trees and area supplemented with additional landscape materials provide a varied edge environment and reinforce the water jet features with an attractive setting for the proximate office buildings. The facility likewise provides a rich visual experience as viewed from the pedestrian path (left) and adjacent roadway (upper right). The spillway riser structure behind the water jet is shown in Figure 22.16.

curve number. The area of each HSG and land cover type is determined to obtain the curve number. If the watershed is very large and contains a considerable amount of diversity in land cover types and HSG, it may be feasible to divide the watershed into homogeneous sub-basins.

Based on local design criteria, the designer determines the amount of offsite runoff that may need to be considered when determining the pond volume. Local design criteria may vary on the amount of offsite runoff that is to be controlled by a stormwater management facility.

The design storms for detention design and emergency spillway design, in many cases, are dictated by local reviewing agencies. Therefore, precipitation data and rainfall patterns or unit hydrographs may be readily available. If local agencies do not dictate the design storm, the designer must use judgment in selecting appropriate rainfall data. Factors included in selecting the design storm for detention include the downstream problems that need to be resolved (e.g., erosion, channel capacity or hydraulic structure capacity.) For emergency spillway design, the size of the facility, risk, potential damage to downstream property, and potential loss of life if the pond facility fails are considerations.

### Estimating the Volume of Storage Required

When initially sizing a SWM facility, the required storage volume to provide the required amount of detention is unknown. An initial estimate of the required storage volume may be made based on the inflow hydrograph and the required outflow rate. The amount of storage required for a given design storm is equal to the representative volume between the inflow and outflow hydrographs.

To obtain a first estimate of the storage required, the outflow hydrograph can be approximated by drawing a straight line from the beginning of substantial runoff on the inflow hydrograph to the point on the receding limb corresponding to the allowable peak outflow rate. Alternatively, TR-55 provides a dimensionless graph relating the ratio of storage volume to runoff volume to the ratio of peak outflow discharge to peak inflow discharge for the four types of synthetic storms (see Figure 22.14).

Once an initial estimate of the required volume has been made and the location has been determined, a preliminary grading design of the pond is done. In preparing the grading plan, the objective is to obtain the preliminary estimate of the storage while keeping in mind such things as minimizing earthwork, nominal height requirements of the embankment, depth limitations for safety, sediment storage, and aquatic vegetation, and aesthetic value of the project.

A graph of the stage-storage relationship is then developed. The stage-versus-storage relationship is derived by measuring the area within the pond at selected elevations and estimating the storage volume based on the area and depth. The areas can be measured with a planimeter on a contour map, or by other methods. (See Chapter 24, Grading and Earthwork, for more information.) The method for computing the storage volume is similar to the average end

area method used for measuring cut and fill quantities. After the areas are measured for each contour within the pond, the storage volume can be computed. Two different equations are commonly used to compute the volume: the average end area method and the conic approximation method. The conic approximation method is used in some computer methods such as HEC-1. The equations for each method are Eqs. (22.19) and (22.20).

Average End Area Method:

$$V_{\text{inc}} = \frac{A_n + A_{n+1}}{2} \times h \quad (22.19)$$

Conic Approximation Method:

$$V_{\text{inc}} = \frac{A_n + A_{n+1} + \sqrt{A_n \times A_{n+1}}}{3} \times h \quad (22.20)$$

where  $V_{\text{inc}}$  = incremental volume in pond between contour lines representing  $A_n$  and  $A_{n+1}$ ,  $A_n$  = measured area of  $n$ -th contour line,  $A_{n+1}$  = measured area of the successive contour line, and  $h$  = elevation difference between the two contour lines.

### Principal Spillway

A typical spillway consists of a drop inlet structure with one or more levels of weir at the top (see Figure 22.15), one or more orifice openings at various elevations, and an outlet pipe. The drop inlet may be a simple riser pipe with the top acting as a weir; or, it may be more complex, such as a structure with a low-flow orifice, two or more intermediate-level orifice openings, and a multi-level weir at the top. Often a modified concrete manhole can be used as a riser, or segments of reinforced concrete pipe may be used. For a dry pond, a special low level orifice opening may be incorporated to provide water quality enhancement, such as extended detention (see following section on Water Quality). For a wet pond, a valve or a sluice gate attached to an opening at the bottom of the pond should be included. This will allow the pond to be easily drained for maintenance tasks such as dredging accumulated sediments out of the pond.

The primary objective in designing the principal spillway is to size the weirs and orifices such that the peak discharge of the design storm is less than the allowable rate. The peak discharge through the principal spillway occurs at the maximum water surface elevation in the pond for a design storm, assuming the flow through the principal spillway is uncontrolled and depends only on available head. The maximum water surface elevation is obtained by routing the inflow hydrograph through the facility. The design of the principal spillway is an iterative process because of the number of controlling factors dictating the outflow hydrograph.

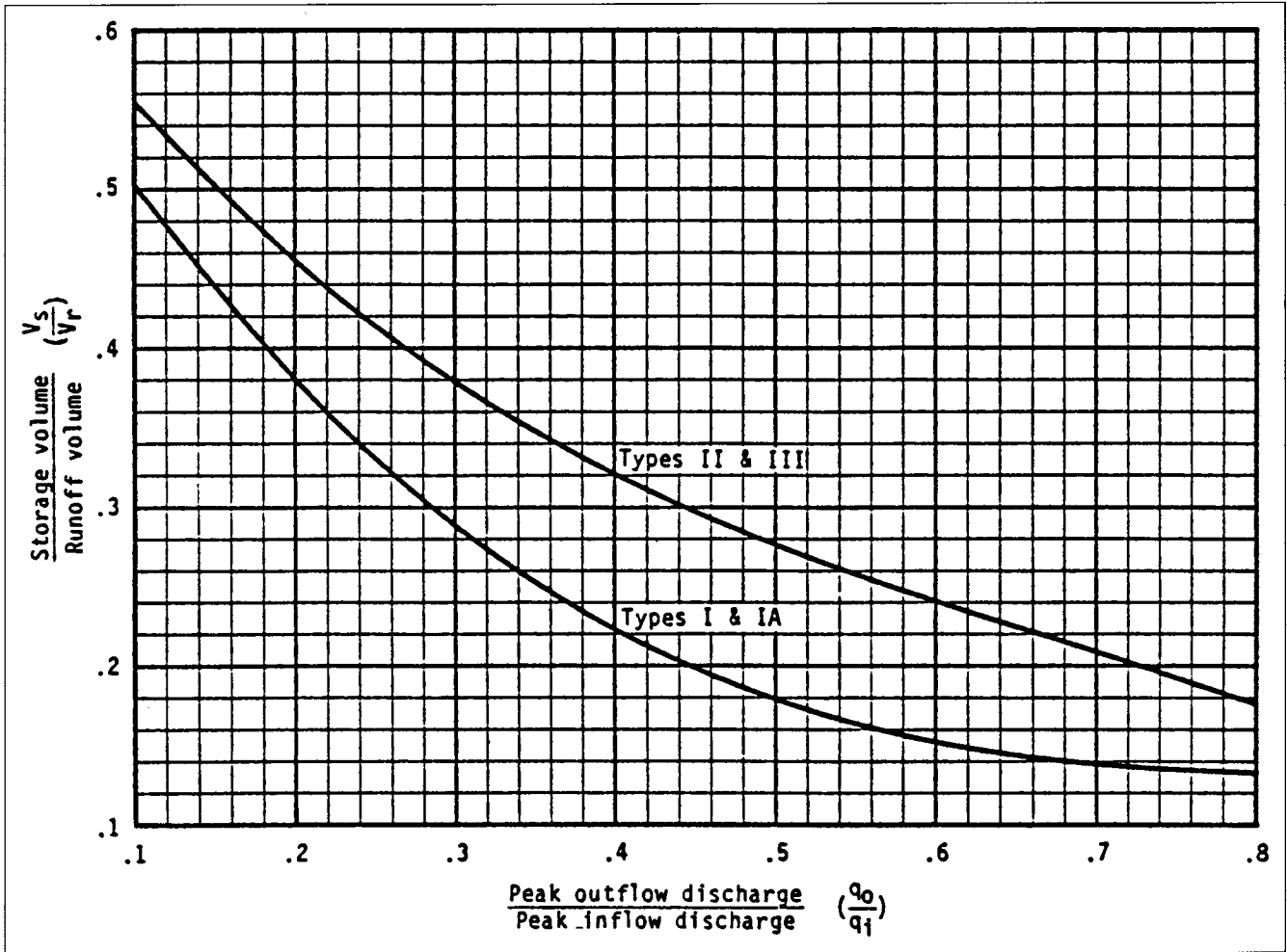


FIGURE 22.14 Approximate detention basin routing for synthetic rainfall distributions. (Source: USDA, TR-55.)

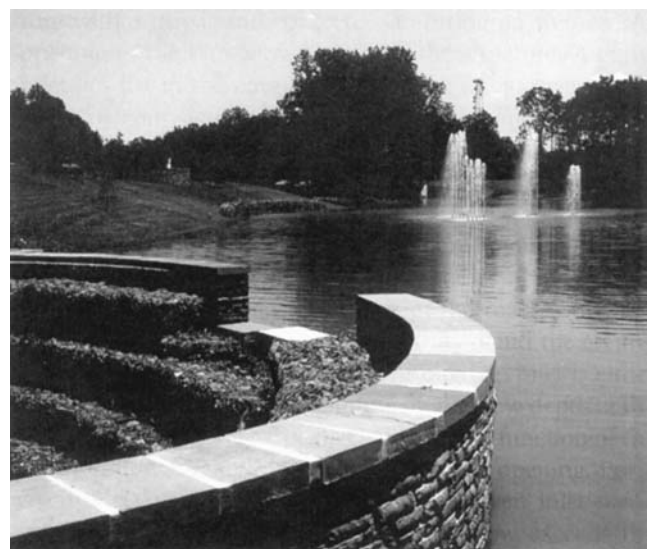


FIGURE 22.15 Typical riser structures; note the difference in aesthetic enhancements.

The following steps briefly outline the procedures for sizing the riser to meet a target discharge.

1. Compute an initial stage versus storage relationship for a given pond site.
2. Assume a set of dimensions for the outlet structure based on maximum allowable depth and/or allowable release rate.
3. Compute the stage-storage-discharge relationship for the pond site and assumed outlet structure.
4. Compute the storage indication curve (discussed below).
5. Route the inflow hydrograph through the storage facility to obtain the outflow hydrograph.
6. If the target discharge is not met (within a reasonable tolerance), modify the dimensions of the outlet structure or the pond and repeat.

**Single Stage Risers.** For the spillway configuration, an elevation-discharge relationship (i.e., rating curve), is developed. Discharge through an orifice is given by

$$Q = C \times A \times \sqrt{2 \times g \times H} \quad (22.22)$$

where  $C$  = a coefficient, typically = 0.6,  $A$  = area of orifice opening (sq ft),  $g$  = gravitational acceleration = 32.2 ft/s<sup>2</sup>,  $H$  = height of water (ft) in the pond above the centroid of the orifice or the difference in water surface elevations on the upstream and downstream sides of orifice.

Discharge over a weir is computed as

$$Q = C \times L \times H^{1.5} \quad (22.23)$$

where  $C$  = a weir coefficient usually  $\approx 2.5$  to 3.3,  $L$  = weir crest length (ft),  $H$  = height (ft) of water above weir crest.

If the weir is submerged (water elevation on downstream side of the weir is above weir crest), the effect of submergence on the computed discharges should be checked. Values of the orifice and weir coefficients and the effects of submergence on weirs can be obtained from hydraulics manuals such as Brater and King (1976).

For flow through a conduit, pipe, or culvert, the equations and nomograph from the publication *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (HDS-5) by the Federal Highway Administration (FHWA) can be used. HDS-5 provides methods for analyzing and designing pipes of all different sizes, shapes, and materials to determine the energy or head loss associated with a given discharge for both inlet and outlet control conditions. (See Chapter 21, Storm Drainage Design, for procedures on culvert design.)

A drop inlet spillway should be designed so that full flow is established through the culvert and riser at as low head over the riser crest as practical and before the head over the top of the riser becomes so large that the riser begins

to act as an orifice instead of a weir. If the top of the riser acts as an orifice, and the riser and outlet pipe are flowing partly full, surging and vibration can occur. This unstable situation can result in damage to the spillway, as well as excessive noise during operation. To prevent this, the riser should have a larger cross-sectional area than the outlet pipe. To determine if the top of the riser is flowing as a weir or an orifice, the discharge is computed for a given water surface elevation using both the orifice equation and the weir equation. The equation that gives the lesser amount of flow is the controlling equation.

Depending on the hydraulic characteristics of the culvert, there may be certain water surface elevations where the potential flow through the riser exceeds the capacity of the culvert. In such cases the actual flow through the riser-culvert system is dictated by the capacity of the culvert at those specific water surface elevations. This is referred to as culvert control, and the stage-discharge curve (i.e., the rating curve) *must* reflect this condition.

Another alternative is to design the outlet pipe such that it will never flow full for the anticipated range of flows that could occur, including ensuring that the outlet will not become submerged. For this situation, the possibility that the amount of flow passing through the pipe be controlled at the inlet of the pipe must be considered, in which case the entrance to the culvert may behave similar to an orifice. The inlet control nomographs or equations from HDS-5 may be used to check inlet control, but it should be noted that these nomographs were developed for highway culverts; attaching a riser to the end of the culvert will likely alter the flow conditions somewhat. The configuration and relative sizes of the riser and outlet pipe will determine how closely the riser and outlet pipe resemble a highway culvert in terms of hydraulic function.

**Multiple Stage Risers.** Depending on the design objectives for the pond, the outlet structure may consist of a simple culvert through a dam, or it may consist of a drop inlet spillway with a more complicated weir/orifice riser structure. Figure 22.16 shows a riser structure designed to meet several objectives. Table 22.6 shows a spreadsheet set up to calculate the discharge from a multi-stage riser. The values of Table 22.6 correspond to the riser of Figure 22.16.

**Orifices:** Columns Q1 and Q2 represent the discharge through the 6- and 12-in. diameter orifice, respectively. The discharge is computed using Eq. (22.22).

Column Q3 is the discharge through the top of the riser as computed using Eq. (22.22). This is used as a comparison with Q3 of the weir to determine whether the spillway is under orifice flow or weir flow control.

For example, the discharge through each orifice for water surface elevation 111 ft is

$$\begin{aligned} Q1 &= (0.6)(0.196) \sqrt{2 g (111 - 100.75)} = 3.0 \text{ cfs} \\ Q2 &= (0.6)(0.785) \sqrt{2 g (111 - 105.5)} = 8.9 \text{ cfs} \\ Q3 &= (0.6)(28.27) \sqrt{2 g (111 - 110)} = 136.1 \text{ cfs} \end{aligned} \quad (22.24)$$



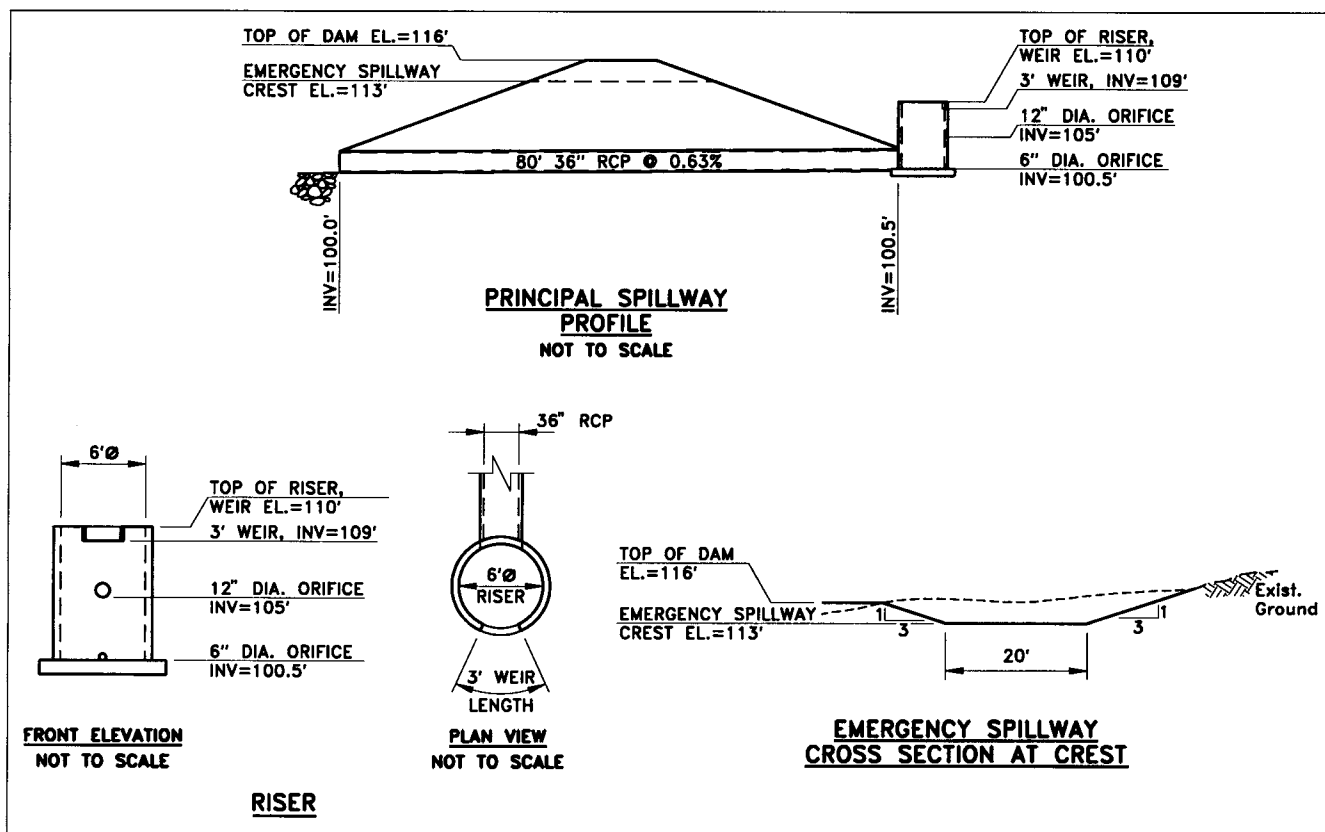


FIGURE 22.16 Riser structure schematic details.

**Weir:** Columns  $Q_1$  and  $Q_2$  are the discharge through weirs at elevations below the top of the riser (in this case there is no weir for  $Q_2$  column). The discharge is computed using Eq. (22.23).  $Q_3$  is the discharge over the riser crest as computed using Eq. (22.23). For example, the weir discharges for water surface elevation of 111 ft are

$$Q_1 = (3)(3)(111 - 109)^{3/2} = 25.5 \text{ cfs} \quad (22.25)$$

$$Q_3 = (3)(15.85)(111 - 110)^{3/2} = 47.5 \text{ cfs}$$

**Riser flow** is the total discharge through the riser without consideration of culvert capacity. The riser is assumed to act as either orifice control or weir control depending on the discharges computed for  $Q_3$  (orifice) and  $Q_3$  (weir). The lesser of these values dictates the type of control. If  $Q_3$  (orifice) is less than  $Q_3$  (weir) the total flow through the riser is  $Q_1$  (orifice) +  $Q_2$  (orifice) +  $Q_3$  (orifice). If the top of the riser is weir control, the total flow through the riser is  $Q_3$  (weir) +  $Q_2$  (weir) +  $Q_3$  (weir) +  $Q_2$  (orifice) +  $Q_3$  (orifice). The riser flow for water surface elevation 111 ft is

$$\text{Riser Flow} = 3.0 + 8.9 + 25.5 + 47.5 = 84.9 \text{ cfs} \quad (22.26)$$

**Culvert capacity** is the maximum discharge that the culvert can convey for the given water surface (see Chapter 14 on Storm Drain Design). The culvert discharge analysis is given in Table 22.7.

**Q P.S.W.** (discharge through the principal spillway) is the lesser value of either the riser flow column or the culvert capacity column.

**Q E.S.W.** (discharge through the emergency spillway) is computed assuming a control section exists and calculating critical depth. It is assumed that the reservoir water surface elevation equals the energy head associated with critical depth at the control section.

**Total flow** is the sum of columns Q P.S.W. and Q E.S.W.

To summarize the preceding discussion, the spillway system consists of the principal spillway (culvert and riser) and the emergency spillway. The total flow through the system is the sum of the principal and emergency spillway flows. The flow through the principal spillway depends on the controlling component—either riser or culvert. The greatest flow through the system can be no greater than the capacity of any single component for the given water surface elevation. For example, at elevation 110 the flow through the riser is 19.9 cfs while the capacity of the culvert is 88.3 cfs. The culvert would convey 88.3 cfs, if the riser structure

**TABLE 22.6 Spillway Rating Curve**

**PROJECT: EXAMPLE RATING CURVE**

**ENGINEER/DATE: GLC/11-19-93  
CULVERT RATING CURVE  
SIZE: 36" DIA. RCP**

**ORIFICE FLOW**

ORIFICE No.	CENTROID ELEV.	AREA	C	DESCRIPTION
1	100.75	0.196	0.6	6" dia. orifice
2	105.5	0.785	0.6	12" dia. orifice
3 (riser top)	110	28.274	0.6	72" dia. riser

**WEIR FLOW**

WEIR No.	CREST ELEV.	LENGTH	C
1	109	3	3
2			
3 (riser top)	110	15.850	3

**EMERGENCY SPILLWAY FLOW**

Crest Elev = 113  
Width = 20  
Z = 3  
C =

ELEVATION	DISCHARGE
100.5	0
102.6	20
103.8	40
105.55	60
108.53	80
112.07	100
114.06	110
116.18	120
118.08	130
120.11	140

ELEV.	ORIFICES			WEIRS			RISER FLOW	CULVERT CAPACITY	Q P.S.W. (CONTROL)	Q E.S.W.	TOTAL FLOW	COMMENTS
	Q1	Q2	Q3	Q1	Q2	Q3						
100.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
102	1.1	0.0	0.0	0.0	0.0	0.0	1.1	14.3	1.1	0.0	1.1	
103	1.4	0.0	0.0	0.0	0.0	0.0	1.4	26.7	1.4	0.0	1.4	
104	1.7	0.0	0.0	0.0	0.0	0.0	1.7	42.3	1.7	0.0	1.7	
105	1.9	0.0	0.0	0.0	0.0	0.0	1.9	53.7	1.9	0.0	1.9	
107	2.4	4.6	0.0	0.0	0.0	0.0	7.0	69.7	7.0	0.0	7.0	
108	2.5	6.0	0.0	0.0	0.0	0.0	8.5	76.4	8.5	0.0	8.5	
109	2.7	7.1	0.0	0.0	0.0	0.0	9.8	82.7	9.8	0.0	9.8	
109.5	2.8	7.6	0.0	3.2	0.0	0.0	13.5	85.5	13.5	0.0	13.5	
110	2.9	8.0	0.0	9.0	0.0	0.0	19.9	88.3	19.9	0.0	19.9	
110.5	3.0	8.5	96.3	16.5	0.0	16.8	44.8	91.1	44.8	0.0	44.8	
111	3.0	8.9	136.1	25.5	0.0	47.5	84.9	94.0	84.9	0.0	84.9	
111.5	3.1	9.3	166.7	35.6	0.0	87.4	135.3	96.8	96.8	0.0	96.8	culvert control
112	3.2	9.6	192.5	46.8	0.0	134.5	194.1	99.6	99.6	0.0	99.6	culvert control
113	3.3	10.4	235.8	72.0	0.0	247.1	249.5	104.7	104.7	0.0	104.7	culvert control
113.5	3.4	10.7	254.7	85.9	0.0	311.3	268.8	107.2	107.2	22.9	130.1	culvert control
114	3.4	11.0	272.3	100.6	0.0	380.4	286.7	109.7	109.7	68.0	177.7	culvert control
114.5	3.5	11.3	288.8	116.1	0.0	453.9	303.6	112.1	112.1	130.9	242.9	culvert control
115	3.6	11.7	304.4	132.3	0.0	531.6	319.6	114.4	114.4	210.6	325.0	culvert control
116	3.7	12.3	333.5	166.7	0.0	698.8	349.4	119.2	119.2	420.8	539.9	culvert control
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

**TABLE 2.2.7 Discharge through the Culvert**

STATION: SHEET: PROJECT: EXAMPLE RATING CURVE

CULVERT DESIGN FORM DESIGNER/DATE: GLC/11-19-93

HYDROLOGICAL DATA  
METHOD: RATIONAL  
DRAINAGE AREA: 40 ACRES

CULVERT TYPE: RCP  
ENT. TYPE: SQ. EDGE WITH HEADWALLS

DESIGN FLOWS / TAILWATER

R.I. (YEARS)	FLOW (CFS)	TW (FT)
0	0	100
20	20	101
40	40	102
80	80	104
120	120	106
160	160	106.5
200	200	107
250	250	107.2
300	300	107.3
400	400	107.4
600	600	107.5

NO. OF PIPES = 1  
DIAMETER = 3

MANNING'S  $n = 0.013$

$K_e = 0.7$

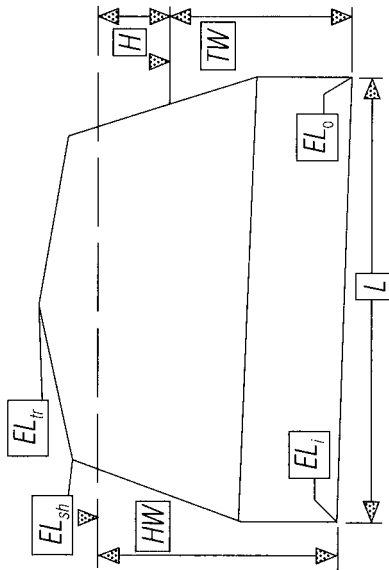
$EL_i = 100.5$

$EL_o = 100$

$EL_{tr} = 116$  top of dam elev.

$EL_{sp} = 113$  emer spillway crest

$L = 80$



HEADWATER CALCULATIONS

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE ENTRANCE	TOTAL FLOW $Q$ (CFS)	FLOW PER BARREL $Q/N$	INLET CONTROL			OUTLET CONTROL				CONTROL HW ELEVATION	APPROXIMATE OUTLET VELOCITY	COMMENTS	
			$HW/D$	$HW^2$	$EL(m)^3$	$TW^4$	$D_c$	$\frac{D_c + D}{2}$	$H_o^5$				$K_e$
1.3' dia. RCP, sq. edge with headwalls	20	20	0.70	2.10	102.60	1	1.45	2.23	0.28	102.51	102.60	5.90	I.C.
"	30	30	0.90	2.70	103.20	1.5	1.80	2.40	0.64	103.03	103.20	6.79	I.C.
"	40	40	1.10	3.30	103.80	2	2.08	2.54	1.13	103.67	103.80	7.65	I.C.

"	50	1.33	4.00	104.50	2.5	2.32	2.66	0.7	1.77	104.43	104.50	7.94	I.C.
"	60	1.62	4.85	105.35	3	2.53	2.76	0.7	2.55	105.55	105.55	8.49	O.C.
"	70	1.96	5.88	106.38	3.5	2.68	2.84	0.7	3.46	106.96	106.96	9.90	O.C.
"	80	2.36	7.08	107.58	4	2.78	2.89	0.7	4.53	108.53	108.53	11.32	O.C.
"	90	2.82	8.45	108.95	4.5	2.86	2.93	0.7	5.73	110.23	110.23	12.73	O.C.
"	100	3.32	9.97	110.47	5	2.90	2.95	0.7	7.07	112.07	112.07	14.15	O.C.
"	110	3.88	11.63	112.13	5.5	2.93	2.97	0.7	8.56	114.06	114.06	15.56	O.C.
"	120	4.48	13.45	113.95	6	2.95	2.98	0.7	10.18	116.18	116.18	16.98	O.C. RD.O.T.
"	130	5.16	15.47	115.97	6.125	2.97	2.98	0.7	11.95	118.08	118.08	18.39	O.C. RD.O.T.
"	140	5.92	17.75	118.25	6.25	2.97	2.99	0.7	13.86	120.11	120.11	19.81	O.C. RD.O.T.
"	150	6.81	20.43	120.93	6.375	2.98	2.99	0.7	15.91	122.29	122.29	21.22	O.C. RD.O.T.
"	160	7.89	23.68	124.18	6.5	2.98	2.99	0.7	18.10	124.60	124.60	22.64	O.C. RD.O.T.

TECHNICAL FOOTNOTES:

<sup>1</sup>HW<sub>i</sub> based on polynomial best-fit equations from the FHA publication entitled *Calculator Design Series #3*.

<sup>2</sup>HW<sub>i</sub> may not be accurate for values <0.5D and >4.5D.

<sup>3</sup>EL(h<sub>i</sub>) = HW<sub>i</sub> + EL<sub>i</sub> (invert of inlet control section).

<sup>4</sup>TW based on downstream control or flow depth in channel.

<sup>5</sup>h<sub>0</sub> = TW or (d<sub>c</sub> + D)/2, whichever is greater.

<sup>6</sup>H = (1 + K<sub>e</sub> + (n̂<sup>2</sup>L)/R̂<sup>4.75</sup>)<sup>0.5</sup> V<sup>2</sup>/2g.

<sup>7</sup>EL(h<sub>0</sub>) = EL<sub>0</sub> + H + h<sub>0</sub>.

were not there to limit the flow to the culvert. At elevation 111.5 the reverse is true. The potential flow through the riser is 135.3 cfs while the capacity of the culvert is 96.8 cfs. Since the culvert is the limiting component, the culvert controls.

Note that the flow through the emergency spillway begins at elevation 113. The total flow through the system is then the sum of the flow of the controlling component of the principal spillway (riser or culvert) plus the flow through the emergency spillway. The composite rating curve of Figure 22.17 is obtained from the last column of Table 22.6. This curve represents the combined hydraulic effects of the individual spillway components.

### Emergency Spillways

The typical function of SWM facilities is to detain runoff for selected design storms and to allow the runoff from larger storms to pass through an emergency spillway. The emergency spillway safely conveys the excess water through, over or around the dam to prevent uncontrolled overtopping—a major cause of dam failure. For large SWM facilities, an emergency spillway typically consists of a wide channel cut along the side of the dam and below the top of dam elevation. For small SWM facilities, the emergency spillway may be overland relief. That is, it may be a quasi-natural flowpath (e.g., over roads, parking areas, lawns, etc.) for the major storm event such that buildings are not inundated.

For major SWM facilities the determination of the appropriate SDF and design of the emergency spillway may consist of a detailed study of potential damage to downstream property in the event of failure. Many small on-site facilities pose minimal risk in the event of failure since their capacity is too small to create a high-risk flood wave. The storm event used for designing the emergency spillway may vary from the 100-yr flood to the probable-maximum flood (PMF) depending on the risk associated with a failure due to overtopping. The PMF is the flood discharge that may

be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMF is often estimated using hydrologic modeling techniques to determine the runoff hydrograph associated with the probable maximum precipitation. The PMP is the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographic location at a certain time of the year.

### Maintenance Access

Urban stormwater management facilities must be designed such that access is provided to allow repairs, cleaning and inspections. Areas of the dam frequented by maintenance personnel are the top of the dam, spillway inlet/outlet structures and channels, shoreline areas, downstream face of dam, mowing areas, and other mechanical areas such as pump and valve houses. Preferably an all-weather surface leading to these points of interest and wide enough for utility vehicle travel should be part of the design. A vehicle turn-around area might also be a consideration.

### Hydrologic (Reservoir) Routing

After a preliminary riser design has been completed, its effectiveness must be evaluated by routing the design storm hydrograph through the pond to obtain the outflow hydrograph. As a flood wave passes through a storage facility, it is both delayed and attenuated as it spreads over the pool surface. The amount of attenuation depends on the stage-storage characteristics of the pond *and* the stage-discharge characteristics of the outflow structure. The amount of attenuation and delay is reflected in the shape of the outflow hydrograph and the position of the peak outflow.

Reservoir routing or hydrologic routing uses mathematical relationships to calculate the outflow hydrograph from the following known information: inflow hydrograph, initial reservoir conditions, reservoir characteristics and principal spillway characteristics. Perhaps the most widely used method of reservoir routing, where the relationship between outflow and storage is non-linear, is the storage-indication method, also known as the modified Puls method. To employ this method the known parameters must include: (1) inflow hydrograph, (2) elevation-storage relationship, (3) elevation-outflow relationship, (4) storage-outflow relationship, and (5) storage indication relationship. Items 4 and 5 are derived from items 2 and 3.

Underlying assumptions behind the storage routing method are that the reservoir water surface is always level and outflow is a unique function of storage. The procedure is based on the repetitive application of the conservation-of-mass equation. The difference between the inflow,  $I$ , and the outflow,  $O$ , during an incremental element of time,  $\Delta t$ , is the change in storage  $\Delta S$ . This continuity principal is mathematically expressed as

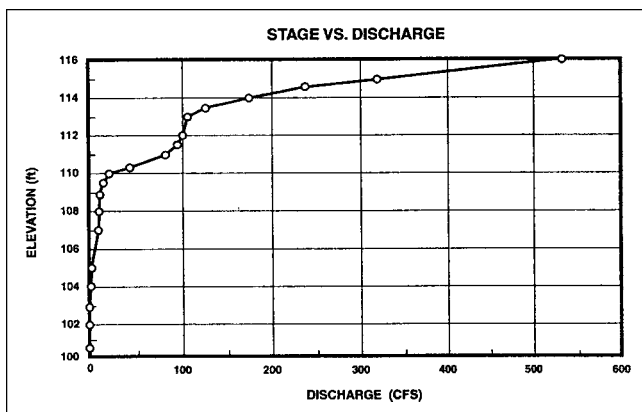


FIGURE 22.17 Typical spillway rating curve.

$$I - O = \frac{\Delta S}{\Delta t} \tag{22.27}$$

With reference to Figure 22.18, the equation in discretized format, where the subscripts  $n$  and  $(n + 1)$  represent the values at the beginning and ending points of the  $\Delta t$  increment, is

$$\left( \frac{I_n + I_{n+1}}{2} - \frac{O_n + O_{n+1}}{2} \right) \Delta t = S_{n+1} - S_n \tag{22.28}$$

Rearranging terms to get the known quantities on the left side of the equality gives

$$(I_n + I_{n+1}) + \left( \frac{2S_n}{\Delta t} - O_n \right) = \frac{2S_{n+1}}{\Delta t} + O_{n+1} \tag{22.29}$$

where  $I$  = inflow,  $O$  = outflow,  $S$  = storage and  $t$  = time and the right side of the equation is the storage indication variable. The increment of  $\Delta t$  is selected such that the hydrograph shape between the values of  $I_n$  and  $I_{n+1}$  and  $O_n$  and  $O_{n+1}$  is approximately linear.

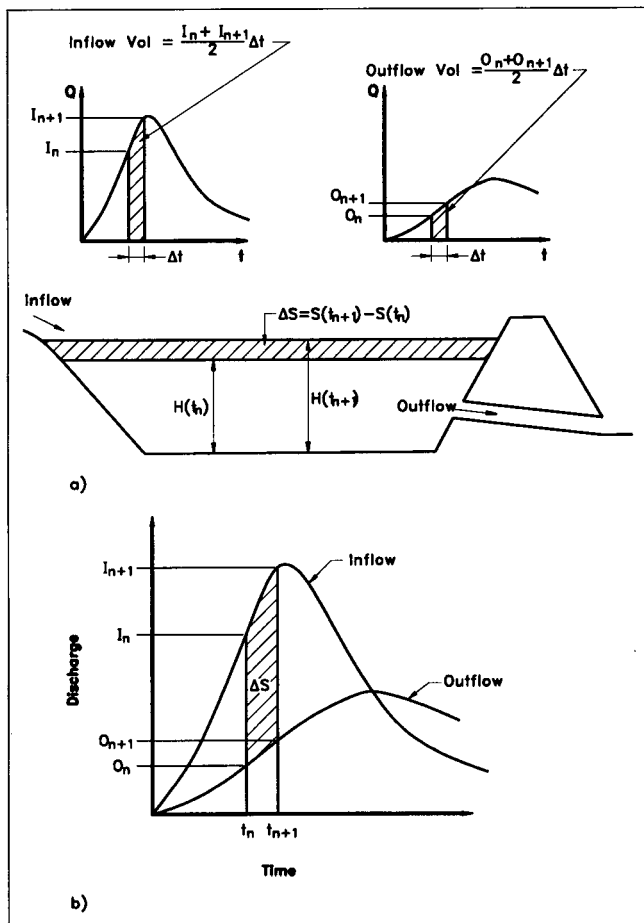
The graphical representation of this equation is shown in Figure 22.18. In the calculation process, the elevation-storage and elevation-discharge relationships are used to develop a storage-indication discharge relationship; this curve is then used to find the value of  $O_{n+1}$ .

**Detention Pond Design Example**

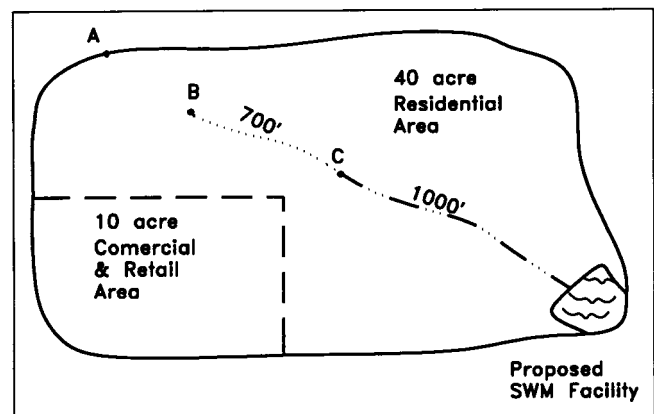
A 50-acre parcel covered with brush and woods is projected for development into 40 acres of 1-acre residential lots and 10 acres of commercial/retail development. The site is located in southeastern Virginia. The HSG is type B (fair condition). Using SCS hydrologic methods, determine a preliminary size of the detention pond and the spillway to detain the 10-yr 24-hr design storm and to pass the 100-yr 24-hr storm without overtopping the embankment. Allowable discharge rate from the spillway for the 10-yr storm cannot exceed the pre-development peak runoff. Site constraints dictate that the maximum allowable height of the embankment is 10 ft. A schematic diagram of the project site is shown in Figure 22.19.

1. The time of concentration and CN is needed for pre-development conditions to compute the pre-development peak runoff rate.

- (a) From Figure 22.3, the 2-yr 24-hr rainfall depth is 4 in., and from Table 22.2,  $n = 0.4$ . From topographic maps of the site, the average ground slope for the first 300-ft flow path of sheet flow runoff (shown A to B in Figure 22.19) is 0.035 ft/ft. From Eq. (22.4) the travel time is



**FIGURE 22.18** Storage change (a) in a reservoir (b) from inflow-outflow hydrographs. (Courtesy of Haan, C.T., and B.J. Barfield, Hydrology and Sedimentology of Surface Mixed Lands. Copyright © 1978 by the Office of Continuing Education and Extension, College of Engineering, University of Kentucky, Lexington, Kentucky, 40506.)



**FIGURE 22.19** 50-acre parcel for detention pond design.

$$T_t = \frac{(0.007)\{(0.04)(300)\}^{0.8}}{(4)^{1/2}(0.035)^{0.4}} = 0.51 \text{ hr} \quad (22.30)$$

(b) The travel time for 700 ft of shallow concentrated flow (B to C on Figure 22.19) over an average ground slope of 0.02 ft/ft (from a topographic map of the site) from Eq. (22.5) is

$$V = (16.1345)(0.02)^{1/2} = 2.3 \text{ fps} \quad (22.31)$$

$$T_t = \frac{700}{(3600)(2.3)} = 0.09 \text{ hr}$$

(c) From field reconnaissance the 1000-ft length of stream is rectangular in shape with average dimensions; bottom width = 4 ft and depth = 2 ft. Manning's  $n \approx 0.03$  and  $S = 0.004$  ft/ft. Using Eq. (22.6).

$$V = \frac{1.49}{0.03} \left( \frac{(2)(4)}{(2)(2) + 4} \right)^{0.67} (0.004)^{1/2} = 3.1 \text{ fps}$$

$$T_t = \frac{1000}{(3600)(3.1)} = 0.09 \text{ hr} \quad (22.32)$$

(d) The pre-development  $t_c = (0.51) + (0.09) + (0.09) = 0.69$  hr.

(e) The  $CN = 65$  is obtained from Table 22.1 for woods-grass category under HSG B (fair condition). Using Eq. (22.2),

$$S = \frac{1000}{65} - 10 = 5.4 \quad (22.33)$$

(f) From Figure 22.4, the 10-yr 24-hr rainfall is 6 in. Substituting for  $P$  and  $S$  in Eq. (22.3), the depth of runoff is

$$Q = \frac{\{6 - (0.2)(5.4)\}^2}{6 + (0.89)(5.4)} = 2.3 \text{ in.} \quad (22.34)$$

(g) Since  $I_a = 0.2 S$ ,  $I_a = 1.08$  and  $I_a/P = 1.08/6 = 0.18$ . Use Figure 22.12 (or Eq. (22.18) with appropriate coefficients) to obtain  $q_u = 305$  cfs. Using Eq. (22.16), the peak discharge for pre-development conditions is

$$q_p = (305) \left( \frac{50}{640} \right) (2.3) = 55 \text{ cfs} \quad (22.35)$$

2. An inflow hydrograph (i.e., direct runoff hydrograph) is needed to design the principal spillway. To determine the inflow hydrograph, the  $CN$  and  $t_c$  for post-development conditions is needed. The weighted  $CN$  value is

$$CN = \frac{92 (10 \text{ acre}) + 68 (40 \text{ acre})}{50} = 73 \quad (22.36)$$

The decrease in impervious area for the post-development condition results in a decrease in  $t_c$  and consequently an increase in peak runoff. It is reasonable to assume a 50% (or more) decrease in the length of flow path for both overland flow and shallow concentrated flow due to the increase in imperviousness and addition of pipes and inlets. For overland flow computation estimating  $L = 100$  ft and assuming there is nominal change in ground slope, the overland flow time for post-development conditions (assuming  $n = 0.24$ —e.g. dense grasses listed in Table 22.2) is

$$T_t = \frac{(0.007)\{(0.24)(100)\}^{0.8}}{(4)^{1/2}(0.035)^{0.4}} = 0.13 \text{ hr} \quad (22.37)$$

The initial 700-ft length used for shallow concentrated flow travel time will undoubtedly be changed. Assume the preliminary plans show that the length for post-development conditions is 100 ft and also that there is little change in ground slope. The post-development travel time for shallow concentrated flow is

$$V = 20.3282 (0.02)^{1/2} = 2.9 \text{ fps} \quad (22.38)$$

$$T_t = \frac{100}{(3600)(2.9)} = 0.01 \text{ hr}$$

In computing the travel time for post-development open channel flow, based on a preliminary layout, part of the flow path will be in pipes and, also, part of the existing stream will be improved to increase capacity. If the length of flow path in pipes is estimated to be 450 ft with an average diameter of 2 ft, the travel time in the pipes is

$$V_p = \frac{1.49}{0.013} \left( \frac{2}{4} \right)^{0.67} (0.02)^{1/2} = 10.2 \text{ fps} \quad (22.39)$$

$$T_t = \frac{450}{(3600)(10.2)} = 0.01 \text{ hr}$$

The preliminary design projects the improved stream to be increased in size to 5 ft wide and 3 ft deep ( $n \approx$

0.02) with a new channel length of 700 ft. The corresponding travel time is

$$V_{st} = \frac{1.49}{0.02} \left( \frac{(5)(3)}{2(3) + (5)} \right) (0.008)^{1/2} = 8.2 \text{ fps}$$

$$T_t = \frac{(700)}{(3600)(8.2)} = 0.02 \text{ hr} \quad (22.40)$$

The  $t_c$  for post development conditions:  $t_c = (0.13) + (0.01) + (0.01) + (0.02) = 0.17$  hrs.

3. The inflow hydrograph is computed using TR-55 tabular method. As with the pre-development peak discharge computations, values of  $S$ ,  $Q$ ,  $I_a$ , and  $I_a/P$  are required. The results are shown in Figure 22.20. (Note that the tabular method only applies to 24-hr storm durations). From the inflow hydrograph, the runoff peak discharge is 190 cfs.

To get a first estimate of the storage volume, draw a line from the point where significant runoff begins to the target discharge release rate on the falling limb of the inflow hydrograph. This line represents the rising limb of the outflow hydrograph. The area between this line and the inflow hy-

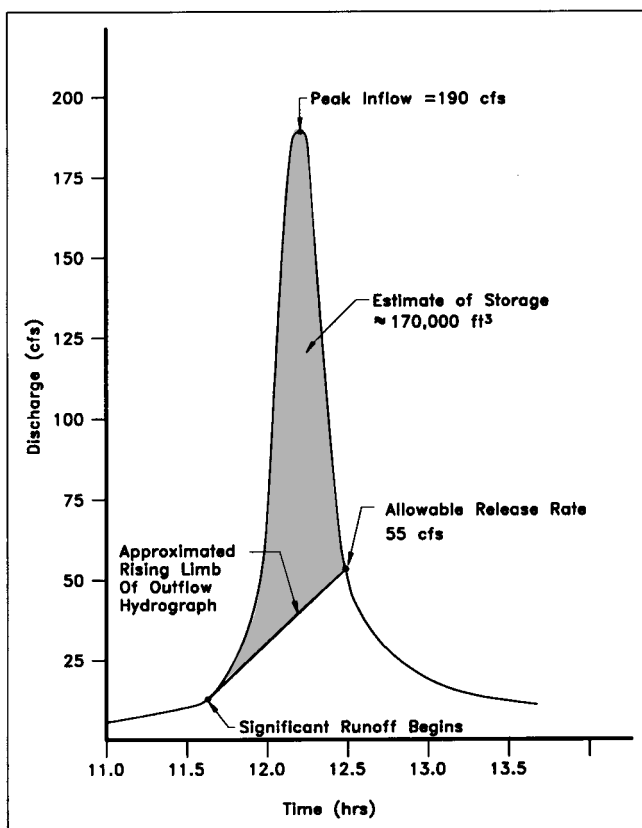


FIGURE 22.20 Inflow hydrograph.

drograph represents the storage of the pond. For this example the initial estimate for storage is about 4 acre-ft ( $\approx 170,000 \text{ ft}^3$ ).

Before the pond is graded using this as the target volume, the site is assessed as to the controlling constraint, area or depth or perhaps other limitations. For example, since the embankment height is limited to 10 ft, the maximum depth of the water is 9 ft (if 1 ft is allowed for free-board). Allowing for 2 ft of depth (as a first guess) over the crest of the weir for the 100-yr storm sets the weir elevation at 147. This limits the depth of water over the low-flow orifice at seven ft. Based on these assumptions, the diameter of the orifice is

$$55 \text{ cfs} = (0.6)(A) \sqrt{2gH} \text{ or} \quad (22.41)$$

$$A = 4.3 \text{ ft}^2$$

which corresponds to a diameter of 2.3 ft. A 27-in. diameter hole is selected.

By the same procedure used to determine the pre-development peak discharge for the 10-yr storm, the 100-yr 24-hr post-development peak discharge is found to be 362 cfs. (Use  $P_{100} = 9$  inches, weighted  $CN = 73$ , and  $t_c = 0.17$  hr.) Using a depth of 2 ft over the crest of the weir determines the length of weir needed to pass 362 cfs.

$$362 \text{ cfs} = 3.0(L)(2^{3/2}) \quad (22.42)$$

$$\text{or } L \approx 43 \text{ ft}$$

To verify the orifice diameter, weir elevation, and length, the inflow hydrograph is routed through the pond. Hydrologic routing using Eq. (22.29) is found in most elementary hydrology textbooks. Long-hand calculations are tedious, and although it is good practice for an engineer to have performed such calculations at least once, routing calculations are typically done using microcomputers and appropriate software.

Figure 22.21 shows the outflow hydrograph for the routed 10-yr inflow hydrograph. Note that the peak discharge of 51 cfs is less than the maximum allowable peak discharge of 55 cfs. The orifice size and/or weir length would have been modified if the outflow peak discharge had been substantially higher or lower than the maximum allowable 55 cfs. Similar procedures are followed if the detention facility is required to attenuate peak discharges for other storm events. Multi-stage riser structures (those with multiple weirs and orifices) can be designed for this purpose.

Note that this example did not consider any factors that might reduce the flow through the spillway. In the analysis, consideration must be given to downstream tailwater effects that may cause a culvert to function as outlet control rather



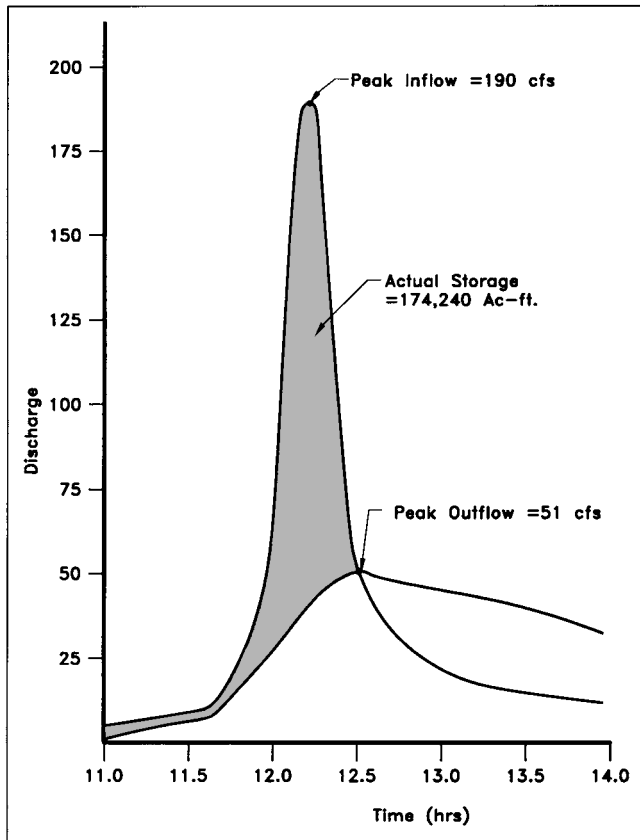


FIGURE 22.21 Inflow-outflow hydrograph.

than inlet control, or may submerge a weir structure, thereby reducing the effective discharge coefficient.

## DAM DESIGN

There are many different types of dams, including concrete, rockfill, and earthfill. Earthfill dams are by far the most common for SWM ponds in urban development projects. Regardless of the size or location of a proposed SWM facility, or the similarity to other projects nearby, the design of the pond, and especially the embankment, should always be based on site-specific information. The actual subsurface conditions for the particular site and the fill materials intended for use as the embankment must be tested for bearing capacity, seepage and stability. The structural integrity should not arbitrarily be assumed just because of the proximity or superficial similarities to another pond. In most cases a geotechnical consultant provides the geotechnical advice.

### Geotechnical Design Considerations

**Embankments.** The embankment should be constructed to control the amount of seepage through the dam in order to prevent piping, as this could lead to dam failure. Depending on the availability of suitable material, the dam may be designed as either a homogeneous or zoned em-

bankment. As the name implies, a homogenous dam embankment is constructed entirely of one type of fill material. Preferably, the material will be sufficiently impervious to inhibit seepage. Some seepage can be expected for earthfill dams of wet ponds. For this reason, homogeneous dams should not be constructed of highly erodible materials such as silts or fine sands, which could be carried away by water seeping through the embankment (piping).

A zoned embankment may be preferred if sufficient impervious material is not available to construct the entire embankment or if the material requires additional measures to prevent slope failures or erosion. For this alternative, two or more zones of different fill material are used to construct the embankment, with one zone, called the dam core, constructed of impervious material. The other zone(s) may be constructed of more pervious materials, such as sand, gravel, and cobbles, and may provide structural stability to the embankment.

If impervious material is not available, an impervious liner may be placed over the embankment to prevent seepage. The liner may consist of clay, possibly mixed with bentonite, or it may be a manufactured synthetic liner. Special care must be taken to ensure that the liner does not crack or puncture. The liner should extend from above the normal pool of the pond to the pond bottom and some additional distance upstream. The distance upstream depends on the imperviousness of the foundation. For very pervious foundation materials, the entire pond may require a liner.

**Drainage Blankets.** The purpose of a drainage blanket is to allow any seepage through the embankment to drain in a controlled way and thereby minimize erosion of the embankment. A drainage blanket consists of a layer of sand, gravel or other pervious material surrounded by appropriate filter material to prevent particles, or fines, from passing from the embankment into the blanket. The filter material may be either a fine-grained pervious material or a synthetic filter fabric. The filter material must be able to filter out particles based on the grain size of the embankment material. The blanket is placed under the downstream side of the embankment, downstream of the dam core. If the drainage blanket is at the toe of the embankment, water may drain from the blanket into a trench drain along the toe of the dam. If a trench drain is not used, a toe drain consisting of a perforated pipe embedded in the toe of the drainage blanket can be used to carry water into the outfall channel.

**Cutoff Trench.** A cutoff trench is usually recommended under the embankment to prevent seepage from passing through the foundation and exiting on the downstream side of the dam when the foundation material immediately below the dam is more pervious than the dam core material. A cutoff trench is excavated into the foundation soil and backfilled with the same impervious material used for the dam core. The cutoff trench should extend through the pervious stratum to rock or some less pervious stratum.

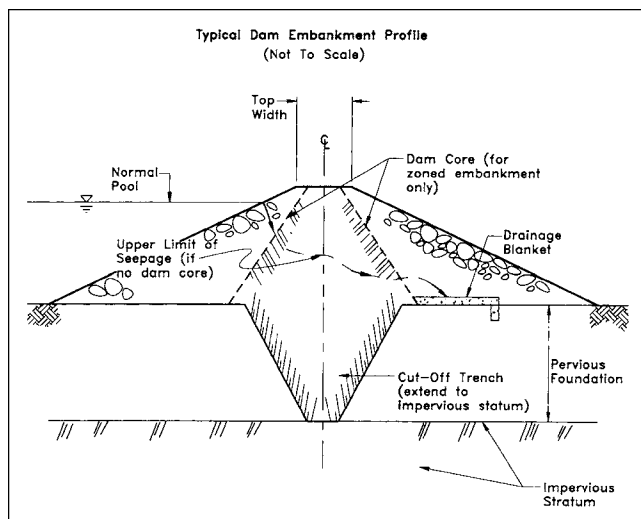
The cutoff trench should be located at or upstream of the centerline of the crest of the dam, but not so far up-

stream that the cover over the trench provides less resistance to seepage than the trench itself. It should extend the full length of the dam. From the U.S. Bureau of Reclamation publication *Design of Small Dams*, the width of the cutoff trench may be determined by the equation

$$w = h - d \quad (22.43)$$

where  $w$  = the bottom width of the cutoff trench,  $h$  = the reservoir head above ground surface and  $d$  = depth of cutoff trench below ground surface.

**Outlet Pipe.** Another potential path for seepage to follow is along the contact between the outside surface of the outlet pipe and the embankment. The compaction of the embankment fill around the outlet pipe must be equivalent to that elsewhere in the embankment. Anti-seepage collars have been used extensively in the past to lengthen the seepage path along the pipe. If used, it is essential that proper compaction around the collars be achieved. At least three federal agencies—the Corps of Engineers (COE), the SCS and the Bureau of Reclamation—recommend other measures in lieu of anti-seepage collars in recent publications (SCS TR-60, *Earth Dams and Reservoirs*, and Bureau of Reclamation, ACER Technical Memo No. 9, *Guidelines For Controlling Seepage Along Conduits Through Embankments*). The SCS recommends use of a “drainage diaphragm,” which is a layer of drainage material, such as sand or gravel, surrounded by a filter layer, designed to allow water to drain out of the embankment but to trap any soil particles and prevent them from being washed away. If a drainage blanket is also used, then the drainage diaphragm should be designed in conjunction with the blanket. (See Figure 22.22 shows a typical dam embankment.)



**FIGURE 22.22** Typical dam embankment profile.

## Spillway Design Considerations

Some items to be considered in designing a spillway structure include the following:

- Reinforced concrete outlet pipes are recommended and must be of sufficient strength to support the embankment fill.
- All joints in the riser and outlet pipe should be made watertight. This is especially important for a wet pond but also good practice for a dry pond.
- The riser should be designed to withstand buoyant forces, with a minimum factor of safety of 1.3.
- Vortex action at the top of the riser should be controlled. An anti-vortex plate or a headwall may be used to prevent a vortex from forming as the water enters the riser. Anti-vortex devices can often be incorporated with a trash rack.
- Trash racks or other debris control devices should be used to prevent the spillway from becoming clogged. Trash racks can also be used to discourage children from playing inside risers and pipes. The trash rack should be designed such that for the range of design flows, the average velocity of flow through the trash rack is 2- to 3-fps maximum. An alternative criteria is to design the trash rack with a surface area of 10 times the cross-sectional area of the riser or outlet pipe opening. The Federal Highway Administration's (FHWA) Hydraulic Engineering Circular No. 9, *Debris Control Structures* gives examples and guidelines for design.
- The outfall channel downstream of the spillway should be protected from erosive velocities of flow exiting the spillway. Riprap lining or energy dissipators may be necessary. The FHWA's Hydraulic Engineering Circular No. 14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* gives examples and procedures for design.

The slopes of the embankment must be such that the allowable foundation pressure is not exceeded, but flat enough to prevent sloughing. Generally, 3H:1V or 4H:1V slopes are adequate and are flat enough that maintenance (mowing, etc.) can be accomplished.

The minimum recommended dam top-width is 8 ft. Table 22.8, excerpted from TR-60, gives a list of minimum dam topwidths for varying dam heights as recommended by the SCS. However, the dam top-width may need to be increased to meet local standards, to allow for different zones of material within the embankment, or to allow traffic to cross the dam.

## Dam Safety

Regardless of how careful a dam is designed and how well it is built, there is always the potential for a dam to be

**TABLE 22.8 Top Width of Dam versus Embankment Height**

TOTAL HEIGHT OF EMBANKMENT (FT)	MINIMUM TOP-WIDTH (FT)
<14	8
14–19	10
20–24	12
25–34	14

breached, either due to overtopping or to erosion from seepage or piping through or around the embankment. One of the important considerations in pond design is the potential for a failure of the dam embankment and the potential hazard to downstream features if such an event occurs. If a dam fails, the volume of water stored in the reservoir can be released downstream, posing a risk to roads, buildings, homes, and other property, as well as to human lives. Many state and federal regulatory agencies have developed classification systems regarding the hazard potential for damages due to a dam failure. The Soil Conservation Service's classification system is presented below (excerpted from TR-60, *Earth Dams and Reservoirs*, revised October 1985):

Class (a)—Dams located in rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and country roads.

Class (b)—Dams located in predominantly rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads or cause interruption of use or service of relatively important public utilities.

Class (c)—Dams located where failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

To assist in determining the dam classification, a dam breach analysis may be necessary. This is a procedure where the outflow hydrograph resulting from a dam breach is routed down the stream valley below the dam to determine the area that may be inundated. Two types of analysis may be performed. The first is sometimes called a "sunny day" dam failure, where the dam is assumed to fail because of water seeping or piping through the embankment, with no additional precipitation. The normal pool volume of water is then routed downstream to determine the breach floodplain. The second type of analysis assumes the dam failure occurs as a result of being overtopped by a large storm (a storm that exceeds the spillway capacity). Once the reser-

voir water level reaches a specified depth above the top of the dam, the dam is assumed to be breached, and the total volume of water stored in the reservoir is routed downstream.

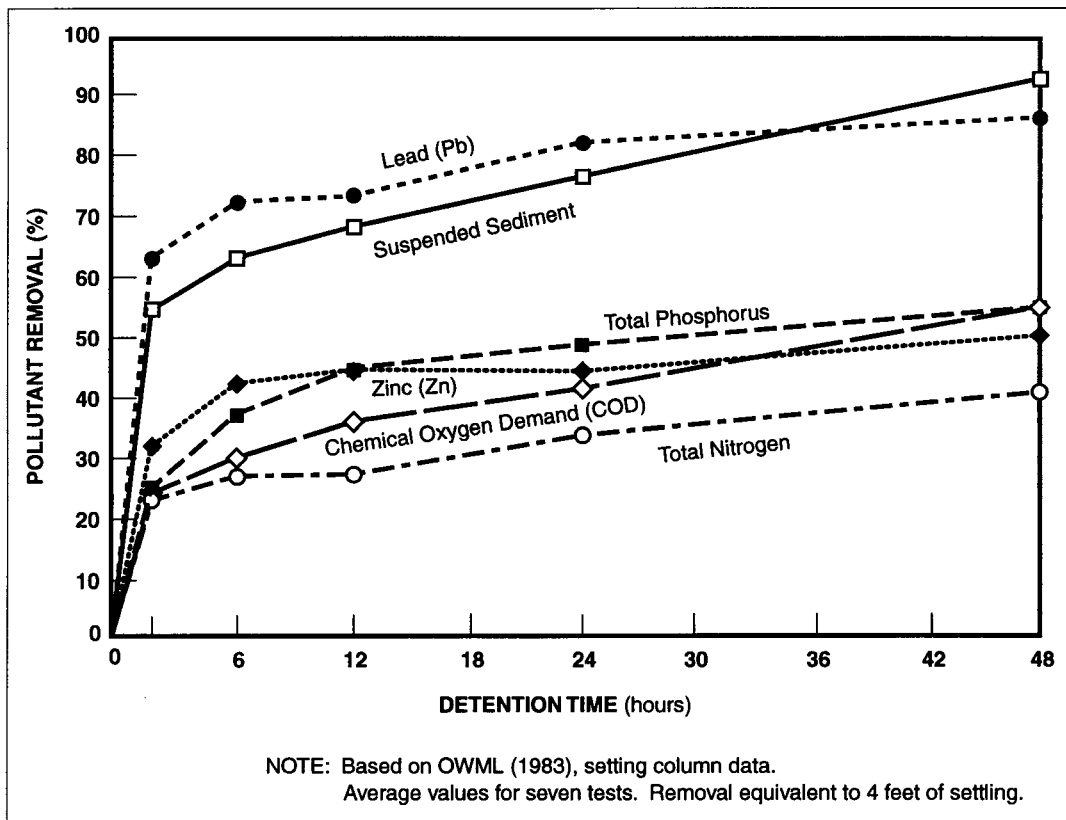
### BEST MANAGEMENT PRACTICES

Best Management Practices (BMPs) are policies, practices, procedures or structures implemented to mitigate the adverse impacts to surface water quality resulting from development. BMPs are categorized as structural or non-structural. A BMP policy may affect the limits on a development's maximum density and open space requirements. A public education program may be considered as a non-structural BMP. Educating the public about activities that contribute to pollution and about practices that may help to reduce pollution can provide the incentive and ability for homeowners to be directly involved in reducing non-point-source pollution and improving water quality. Such practices include the proper use of fertilizers, disposal of used petroleum products, water conservation, litter control, and controls for minimizing erosion from disturbed ground. These most fundamental types of BMPs may only be of limited use in land development engineering but should be recognized as alternatives to be considered for a development or community.

In terms of land development engineering, either structural or non-structural BMPs may be utilized. Structural BMPs include such controls as extended detention ponds, dry ponds, infiltration trenches, porous pavements, and water quality inlets. Non-structural BMPs include street cleaning, fertilizer application control, and certain vegetative practices such as grass swales, filter strips, and shallow marshes/wetlands.

The primary mechanisms for pollutant removal in BMP facilities are settling of particulate pollutants and sediments, and to a lesser degree infiltration of soluble nutrients through the soil profile, and biological and chemical stabilization of nutrients. Settling column experiments have been performed to estimate the removal of pollutants from urban runoff. Figure 22.23 (Scheuler, 1987), illustrating the percentage of pollutant removal with time, shows that each curve roughly parallels that of suspended sediments. Thus, suspended sediment removal frequently forms the basis of BMP designs. It should be noted that the efficiency of removal of pollutants is related to the particle size of the pollutants (which affects the settling velocity), the velocity of flow through the storage area, and the depth and total storage volume available (related to the hydraulic residence time, or the length of time during which settling may occur). Since the majority of pollutants is normally washed off the land surface in the first flush, or the initial portion of runoff from a storm event, most BMPs are designed to capture or treat the "first flush" of runoff volume.

The selection for and effectiveness of a BMP facility depends on numerous site conditions such as climate, watershed size, soil permeability, ground slope, subsurface conditions (e.g., bedrock and groundwater), and land use



**FIGURE 22.23** Removal rate versus detention time for selected pollutants. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

to name a few. Figure 22.24 through Figure 22.26 recommends various restrictions to specific BMP facilities.

Another factor influencing the design and selection of a BMP facility is what the governing criteria are for efficiency. Local criteria prescribe the allowable post-development pollutant loading. Typical policies range from “no net gains,” where post-development pollutant loading must be less than or equal to pre-development pollutant loads, to reducing the post-development pollutant loads to a specific percentage of the pre-developed conditions. Other criteria may set the allowable loadings on a site-by-site basis, perhaps based on location within a water or sewer district. The pollutant removal efficiencies shown in Figure 22.27 might be considered for ideal conditions. These values are to be used as a guide since some studies have shown substantial differences between these efficiencies. Of the types of BMP measures listed in Figure 22.29, the most effective types of facilities, according to a study done in Maryland (Galli, 1982), are the wet ponds, extended detention ponds, artificial marshes and infiltration trenches (see Table 22.9).

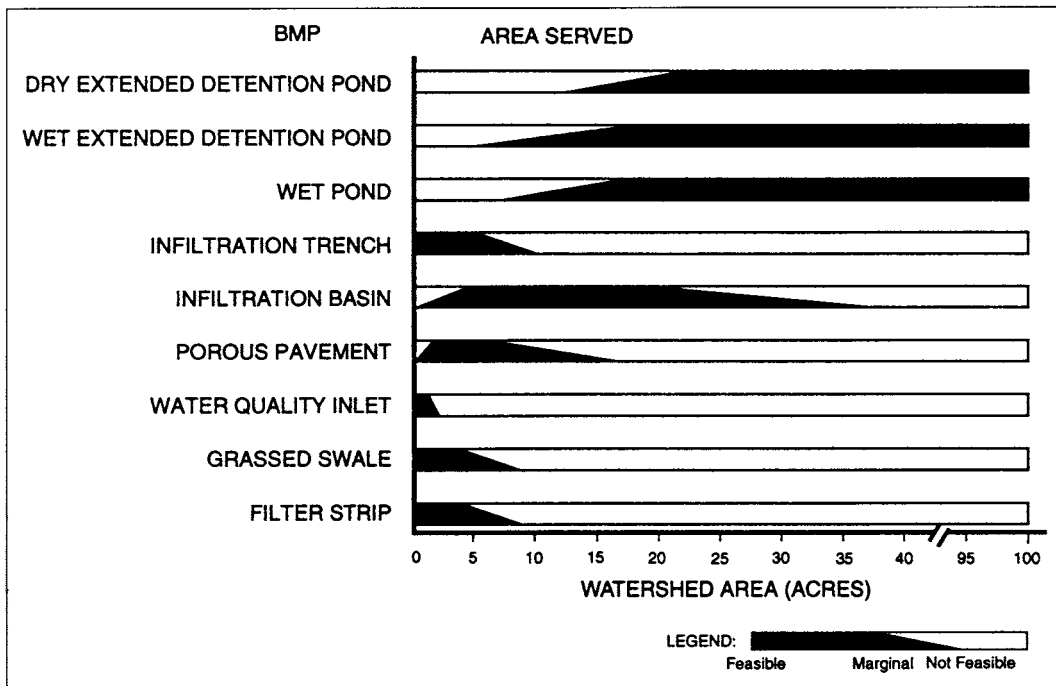
### Extended Detention (ED) Ponds

Extended detention ponds operate in a manner similar to the conventional detention ponds used for controlling the quantity of stormwater runoff. For ED ponds, storage times

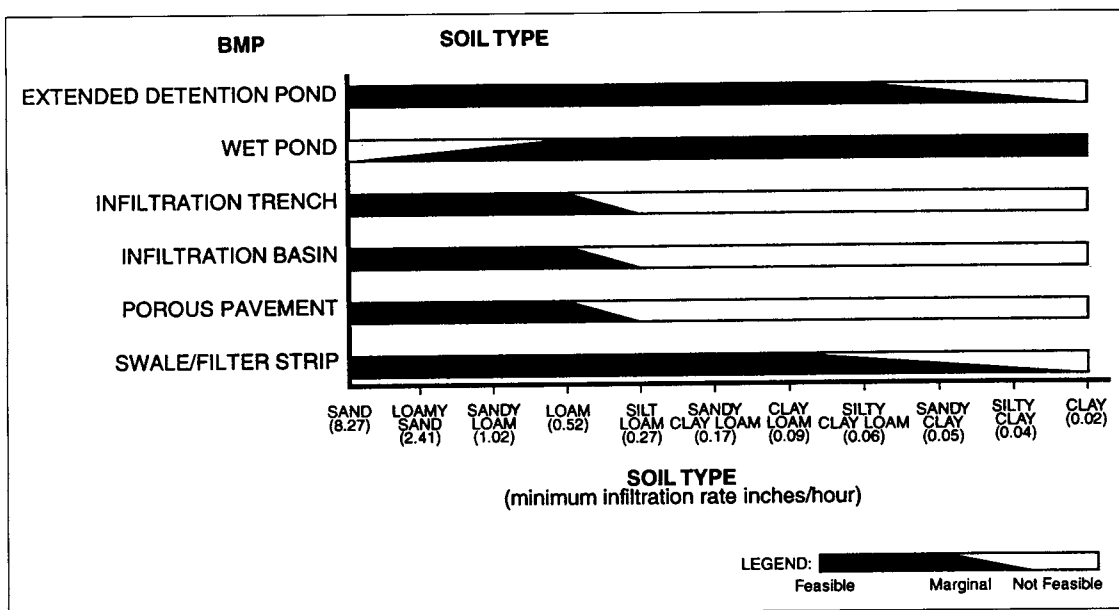
of up to 48 hr may be required to allow sediments to settle out. The storage time is a function of the volume and outlet device used to control the release rate. Figure 22.28 shows a typical facility. A modification to this ED pond is the two-stage ED pond of Figure 22.29. This type of BMP facility is designed such that the volume of the lower part is the water quality control area, and the upper area is the water quantity control area. The lower part has sufficient BMP volume for the first flush of stormwater runoff. The upper area is sized to control the runoff larger than the first flush volume.

Extended detention ponds rely primarily on settling to remove pollutants; thus, they are relatively effective in removing particulate pollutants and pollutants that are adsorbed onto particulates, but they are relatively ineffective in removing soluble pollutants.

**Storage Volume/Detention Period.** There are various methods in use for estimating the appropriate BMP storage volume and detention time. Frequently the state or local government will mandate the volume to be detained and the detention period. A typical BMP design may involve detaining the runoff from a one-in rainfall over a minimum 24-hr drawdown period. If design methods are not specified, one may design a facility to capture the runoff from 90% of the rainfall events and to store the runoff for a period between 6 and 48 hr. Table 22.10 presents rainfall



**FIGURE 22.24** Watershed area restrictions for BMPs. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)



**FIGURE 22.25** Soil permeability restrictions for BMPs. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

BMP	Slope	High Water Table	Close to Bedrock	Proximity to Foundations	Space Consumption	Maximum Depth	Restricted Land Uses	High Sediment Input	Thermal Impacts
Extended Detention Pond	●	●	●	●	○	●	●	●	●
Wet Pond	●	●	●	●	○	○	●	●	○
Infiltration Trench	○	○	○	○	●	○	●	○	●
Infiltration Basin	●	○	○	●	●	○	●	○	●
Porous Pavement	○	○	○	○	○	○	○	○	●
Water Quality Inlet	●	●	○	○	●	○	○	○	●
Grassed Swale	○	○	●	●	●	●	○	○	●
Filter Strip	●	●	●	●	●	●	●	○	●

LEGEND ● May preclude the use of a BMP  
 ● Can be overcome with careful site design  
 ○ Generally not a restriction

**FIGURE 22.26** Other common restrictions on BMPs. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

amounts for several cities. In the absence of explicitly stated volume and detention time requirements, judgment must be exercised to determine the appropriate volume for a given location and conditions.

Some alternative criteria dictating requisite volumes and detention times are presented in Table 22.11.

**Outlet Configuration.** The duration of the BMP storage is controlled by the outlet device. The basic outlet device consists of a small orifice opening or a series of openings sized to provide the desired drawdown time. Sizing the orifice openings such that the pond provides adequate detention time for the entire spectrum of storms requires careful analysis. Generally, the pond designer should perform several storage routing calculations to determine the approximate detention time for smaller, more frequent runoff events. Grizzard et. al. (1986) suggest that, as a target, the average detention time in the pond should be 24 hr for the entire spectrum of storms each year.

Figure 22.30 shows two examples of devices that have been used as BMP extended detention outlet systems. The first device shown is a perforated riser with a gravel jacket. The holes in the riser allow the detained runoff to drain from the pond, and the gravel jacket wire mesh screen prevents clogging of the holes. Disadvantages of this design are that the outer layers of gravel, especially the lower layers, may become clogged with sediments.

The second device shows a short section of pipe connected to the riser at one end and covered with a steel plate at the other. A small diameter orifice in the plate controls the outflow. The steel plate is attached after the pond is

fully constructed and stabilized. This method allows construction to proceed after rainstorms since the short section of pipe without the plate attached allows complete dewatering of the pond.

Key points used in designing ED ponds are outlined in Table 22.12 and schematically diagrammed in Figure 22.31.

### Wet Ponds

Wet ponds, or retention ponds, can function as multi-purpose facilities. As a BMP facility, a wet point is one of the most effective means for improving water quality. A wet pond, when properly designed, uses sedimentation, biological and chemical mechanisms to remove soluble and insoluble pollutants. Additionally, a wet pond can be used for controlling stormwater runoff, ecosystem habitat, source for groundwater recharge, and as an amenity to the project.

**Permanent Pool Volume.** The volume of the permanent pool of a BMP wet pond significantly affects the efficiency of pollutant removal. The efficiency of pollutant removal is related to the ratio of the volume of the pond basin to the volume of runoff generated from the “mean” storm, referred to as the VB/VR ratio. The mean storm must be determined from rainfall records on a site-specific basis. As a figure for comparison purposes, the mean storm for the Northern Virginia area is estimated at approximately 0.4 in. of rain. This is based on an average annual rainfall of 40 in. and an average of 100 storm events per year.

Figure 22.32 relates the expected long-term efficiency of pollutant removal to the VB/VR ratio. As can be seen in the figure, the larger the VB/VR ratio (i.e., the larger the basin

BMP/Design	SUSPENDED SEDIMENT	TOTAL PHOSPHORUS	TOTAL NITROGEN	OXYGEN DEMAND	TRACE METALS	BACTERIA	OVERALL REMOVAL CAPABILITY
<b>EXTENDED DETENTION POND</b>							
DESIGN 1	◐	◐	◐	◐	◐	⊗	MODERATE
DESIGN 2	●	◐	◐	◐	◐	⊗	MODERATE
DESIGN 3	●	◐	◐	◐	◐	⊗	HIGH
<b>WET POND</b>							
DESIGN 4	◐	◐	◐	◐	◐	⊗	MODERATE
DESIGN 5	◐	◐	◐	◐	◐	⊗	MODERATE
DESIGN 6	●	◐	◐	◐	◐	⊗	HIGH
<b>INFILTRATION TRENCH</b>							
DESIGN 7	◐	◐	◐	◐	◐	◐	MODERATE
DESIGN 8	●	◐	◐	◐	●	◐	HIGH
DESIGN 9	●	◐	◐	●	●	●	HIGH
<b>INFILTRATION BASIN</b>							
DESIGN 7	◐	◐	◐	◐	◐	◐	MODERATE
DESIGN 8	●	◐	◐	◐	●	◐	HIGH
DESIGN 9	●	◐	◐	●	●	●	HIGH
<b>POROUS PAVEMENT</b>							
DESIGN 7	◐	◐	◐	◐	◐	◐	MODERATE
DESIGN 8	●	◐	◐	◐	●	●	HIGH
DESIGN 9	●	◐	◐	●	●	●	HIGH
<b>WATER QUALITY INLET</b>							
DESIGN 10	○	⊗	⊗	⊗	⊗	⊗	LOW
<b>FILTER STRIP</b>							
DESIGN 11	◐	○	○	○	◐	⊗	LOW
DESIGN 12	●	◐	◐	◐	●	⊗	MODERATE
<b>GRASSED SWALE</b>							
DESIGN 13	○	○	○	○	○	⊗	LOW
DESIGN 14	◐	◐	◐	◐	○	⊗	LOW

- Design 1: First-flush runoff volume detained for 6-12 hours.
- Design 2: Runoff volume produced by 1.0 inch, detained 24 hours.
- Design 3: As in Design 2, but with shallow marsh in bottom stage.
- Design 4: Permanent pool equal to 0.5 inch storage per impervious acre.
- Design 5: Permanent pool equal to 2.5 (Vr); where Vr mean storm runoff.
- Design 6: Permanent pool equal to 4.0 (Vr); approx. 2 weeks retention.
- Design 7: Facility exfiltrates first-flush; 0.5 inch runoff/impervious acre.
- Design 8: Facility exfiltrates one inch runoff volume per impervious acre.
- Design 9: Facility exfiltrates all runoff up to the 2 year design storm.
- Design 10: 400 cubic feet wet storage per impervious acre.
- Design 11: 20 foot wide turf strip.
- Design 12: 100 foot wide forested strip, with level spreader.
- Design 13: High slope swales, with no check dams.
- Design 14: Low gradient swales with check dams.

- KEY:
- 0 TO 20% REMOVAL
  - ◐ 20 TO 40% REMOVAL
  - ◑ 40 TO 60% REMOVAL
  - ◒ 60 TO 80% REMOVAL
  - 80 TO 100% REMOVAL
  - ⊗ INSUFFICIENT KNOWLEDGE

**FIGURE 22.27** Comparative pollutant removal of urban BMP designs. (Courtesy of Galli, John, 1982, *Analysis of Urban BMP Performance and Longevity in Prince George's County, Maryland*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

**TABLE 22.9 Summary: General Attributes of BMP Systems Field Surveyed**

FACTOR	I. BENEFITS									
	INFILTRATION TRENCHES	POROUS PAVEMENT & DRY WELLS	INFILTRATION BASINS	GRASS FILTERS/SWALES	ED DRY PONDS	WET PONDS	ARTIFICIAL MARSHES	POCKET WETLANDS	OIL / GRIT SEPARATORS	DRY PONDS
Downstream Channel Protection	●	○	○	○	●	●	●	●	○	○
Removal of Particulate Pollutants	●	○	●	○	●	●	○	●	○	○
Removal of Soluble Pollutants	●	○	○	○	○	●	●	○	○	○
Aquatic/Wildlife Habitat Creation	○	○	○	○	○	●	●	○	○	○
Wetland Creation	○	○	●	○	○	●	●	●	○	○
Thermal Impact Protection	●	●	●	○	○	●	○	○	○	○
II. DISADVANTAGES										
Soils Limitations	●	●	●	●	●	●	●	●	○	●
Maintenance Requirements	●	●	●	●	●	●	●	●	●	●
Space Consumption	●	○	●	●	●	●	●	●	○	●
Public Safety Hazards	○	○	●	○	●	●	●	●	○	●
Functional Life/Reliability*	○	○	○	●	○	●	●	○	○	○

Source: Courtesy of Galli, John, 1982, *Analysis of Urban BMP Performance and Longevity in Prince George's County, Maryland*. Washington, D.C.: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

- Low
- Moderate
- High

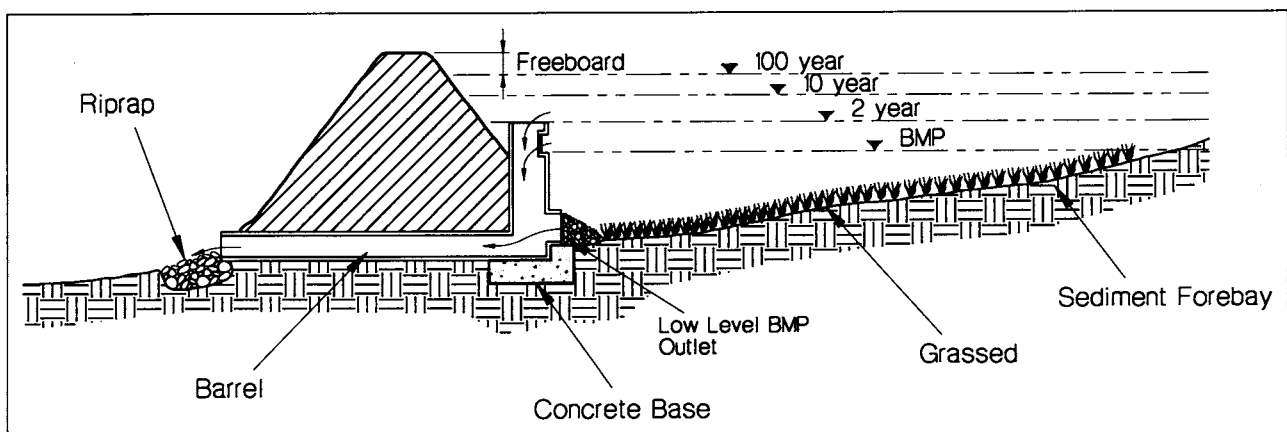
\*General ability to provide water quality/quantity control benefits for ≥5 years without regular sediment/trash removal.

size), the greater the efficiency that can be expected. However, the figure also illustrates that after a certain point, only a small increase in efficiency can be expected for a large increase in basin volume.

The size of pond basin volume to be used for a particular application should be determined to meet the desired pollutant levels. However, other factors must also be accounted for, such as land requirements and construction costs. In

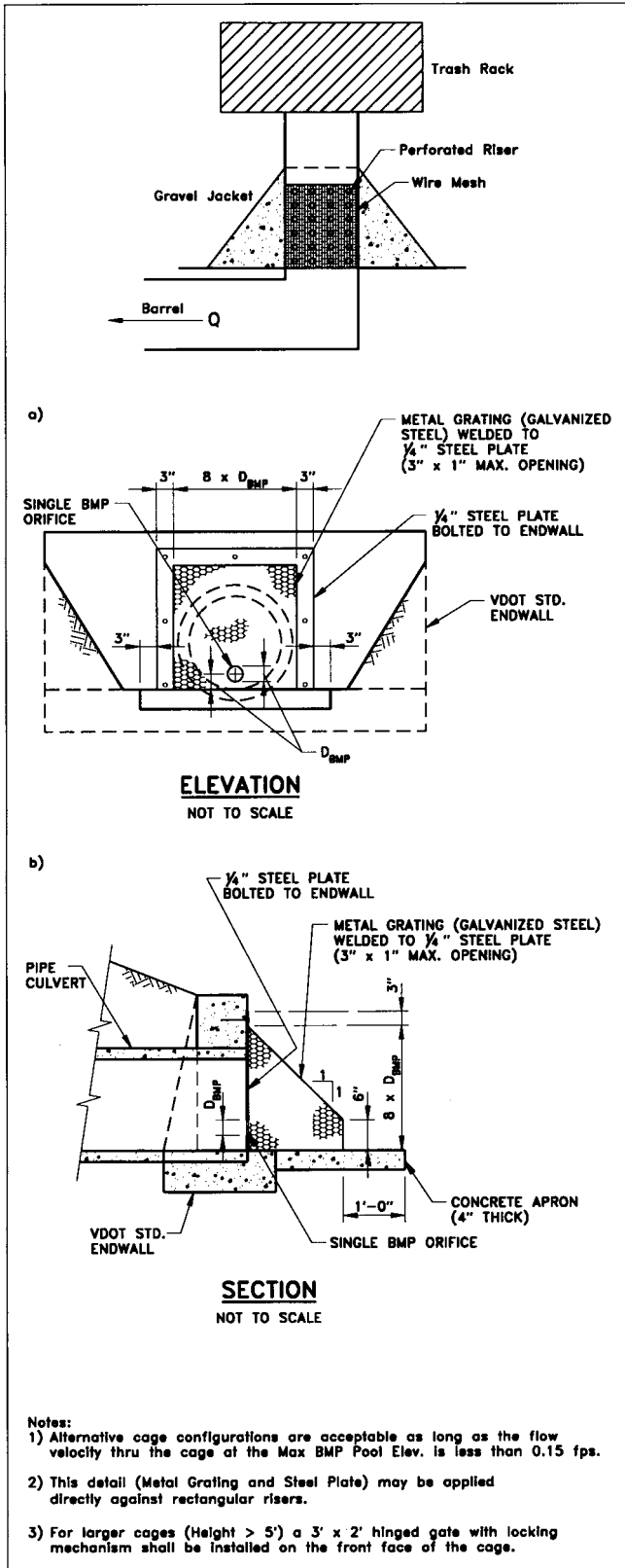
addition, it is recommended that the expected sediment load to the pond be determined. Based on this sediment load and the expected trap efficiency for the facility, the rate of sediment accumulation may be estimated. This sediment will likely require periodic dredging from the pond, so additional costs for maintenance will be incurred.

The diagram of Figure 22.33, a BMP detention outlet system, is used for ponds with a permanent pool or shallow



**FIGURE 22.28** Typical ED pond. (Courtesy of Engineers & Surveyors Institute and Northern Virginia Planning District Commission, 8401 Arlington Blvd. Fairfax, Va. 22031.)





**FIGURE 22.29** Methods for extending detention time in dry ponds. (Figure 22.33a courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

**TABLE 22.10** Approximate Storm Event Depth (in.) for the 90% Event Frequency of Storms and Yearly Volumes (with Minimum Inter-Event Dry Periods of Four Hours)

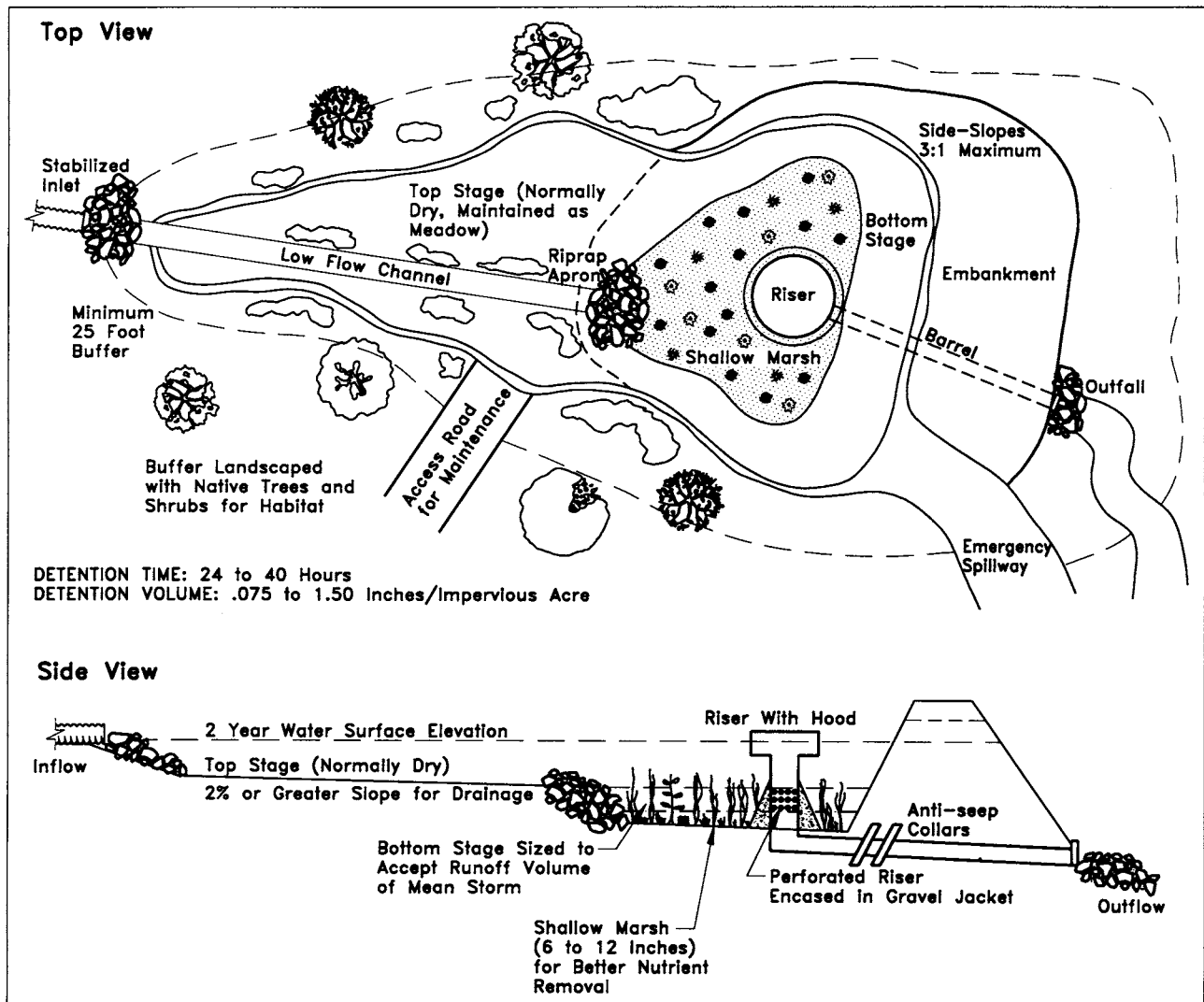
LOCATION	APPROXIMATE VOLUMES (IN)	
	≥90% FREQUENCY	ANNUAL VOLUME
Florida	1.0	52
Boston, Massachusetts	0.9	45
Baltimore, Maryland	1.0	42
Denver, Colorado	0.5	15
Washington, D.C.	1.0	44
Austin, Texas	1.0	32

Source: Warielista, Martin P. *Stormwater Management*, New York: John Wiley & Sons. Reprinted by permission of John Wiley & Sons.

**TABLE 22.11** Selected Alternative Criteria Used for Determining ED Volumes and Detention Times

- A volume equivalent to 1/2 in. of runoff distributed over the contributing watershed released over a 50-hr period (Montgomery County, MD).
- The runoff volume generated from the 1-yr 24-hr storm be released over a minimum of 24 hr (Prince Georges County, MD).
- The runoff volume generated from the 2-yr 24-hr design storm released over 24 hr.
- The runoff volume generated from a one inch storm released over 24 hr.
- A volume equivalent to a land use dependent runoff depth distributed over the contributing watershed, released over a 40-hr period (Fairfax County, VA).
- The "first flush" runoff volume (i.e., one-half inch per impervious acre) released over 24 hr (MD WRA).

Source: Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)



**FIGURE 22.30** Schematic of ED pond design features. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

marsh at the bottom. It consists of a pipe extending from the riser at a negative slope into the permanent pool or marsh. The pipe may be sized to provide the flow rate required, or a cover plate may be bolted to the headwall, with the appropriate sized hole or holes drilled in the plate.

Table 22.13 lists various rules that determine the minimum volume of the permanent pool for areas in the metropolitan Washington, D.C. area.

**Pond Shape and Condition.** It is important to provide the maximum possible distance between the inlet(s) and the outlet provided that the incoming water displaces water already in the pond. If a pond is short-circuited—that is, if the incoming water passes directly to the outlet without causing displacement—the pollutant removal will be reduced. It is sometimes recommended to design the pond

to have a length-to-width ratio of 3:1 to prevent short-circuiting.

The average depth of the pond is recommended to be between 3 and 6 ft. Shallower ponds are more efficient, since settling is the primary removal mechanism, and removal is considered accomplished when the particles reach the pond bottom. However, a very shallow pond causes resuspension of settled particles by wave action and promotes vegetation throughout the entire pond. A very deep pond could become thermally stratified, which can result in anoxic conditions and possible subsequent resuspension of pollutants in the lower layers.

Vegetation within the pond also enhances pollutant removal. It may, therefore, be advantageous in grading the pond to provide a shallow area around the perimeter for

**TABLE 22.12 Design Summary: Extended Detention Ponds**

- **QUANTITY DETAINED:**  
 At a minimum, the volume of runoff detained should be equivalent to the runoff volume produced by a 1-in. storm. This volume is sufficient to achieve both high levels of particulate removal and downstream channel erosion protection for most of the storms that occur during a year. Higher levels of control can be achieved when the runoff volume from the one or 2-yr storm is detained.
- **DURATION:**  
 24 hr of extra detention are needed for optimal pollutant removal for the design detention volume. The control device should be adjusted so that smaller runoff events (0.1 to 0.2 in.), which normally pass through the pond quickly, are detained for at least a minimum of 6 hr. In larger watersheds, up to 40 hr of extended detention may be needed for streambank erosion control. As a final check, the runoff velocity of the downstream channel at the extended detention release rate should be computed to make sure that it is not erosive (Example 23.1).
- **TWO-STAGE DESIGN:**  
 A two-stage pond design is recommended when extended detention is applied to dry ponds. The upper stage of the pond is sized and graded (2% minimum) to remain dry except during infrequent large storms, while the bottom stage is expected to be regularly inundated. The volume of the bottom stage should be set to store the runoff produced by the mean storm (approximately 0.45 in.; see Eq. (3.3)). The bottom stage will frequently be too wet to mow and is best managed as a wetland or as a shallow pool. Both techniques act to prevent resuspension of previously deposited materials. Extra storage, over and above stormwater and extended detention requirements, should be provided within the bottom stage, or at the inlet to account for 20 yr of sediment deposition.
- **WETLAND CREATION:**  
 Wherever possible, a wetland marsh should be created in the bottom stage of an extended detention pond to help remove soluble pollutants that cannot be removed by conventional settling. Wetlands also provide wildlife habitat and hide unsightly debris and sediment deposits that frequently accumulate near the riser. The area of the wetland should be adjusted so that the average annual watershed loading (as computed by the Simple Method) does not exceed 45 lb of phosphorus or 225 lb of nitrogen per surface acre of wetland. Water depths of 6 to 12 in. are needed for optimal wetland growth. The wetland should be planted with native species that are suited to that environment (see Chapter 9 for further guidance on wetland plantings).
- **EXTENDED DETENTION CONTROL DEVICE:**  
 In dry ponds, a vertical, internally controlled extension of the low flow orifice is the most trouble-free design, since it can withstand partial clogging and gradual sediment accumulation, and also can be used to set water levels. If the control device is below the ground surface, it should be protected with filter cloth and/or wire mesh, and encased in a trench of stone or gravel with a diameter greater than the orifice. The device should also have an above-ground extension, with a tight-fitting replaceable cap to facilitate clean-out. In wet ponds, a negatively-sloped pipe protected by a wire mesh that extends from riser and withdraws water from at least a foot below the surface should be used.
- **PILOT CHANNELS:**  
 A riprap, concrete or paved low flow channel is required to route water through the upper stage of the extended detention pond. The pilot channel should end at the lip of the lower stage, where riprap or gabion baffles are placed to reduce velocities and spread out the flow path of the runoff reaching the lower stage, thus preventing scour and resuspension.
- **SIDE SLOPES:**  
 Side slopes should be no steeper than 3:1 (h:v), and no flatter than 20:1 (h:v).

**TABLE 22.12 Design Summary: Extended Detention Ponds (Continued)****■ POND BUFFER:**

A minimum 25-foot-wide buffer strip away from pond to the nearest lot should be reserved and landscaped using low-maintenance grasses, shrubs and trees. A landscaping plan should be prepared for the pond and buffer that improves the appearance for adjacent residents, meets specific design functions, and provides local wildlife habitat (see Chapter 9 for further details).

**■ EMBANKMENT:**

At least 10–15% extra fill should be allowed on the embankment to account for possible subsidence. The embankment should have at least one ft of freeboard above the emergency spillway. Anti-seep collars should be used to prevent seepage around the barrel. The embankment should be graded to allow access for heavy equipment and should be mowed twice a year to prevent woody growth.

**■ SITE ACCESS:**

Adequate access from public or private right-of-ways to the pond should be reserved. The access should be at least ten ft wide, on a slope of 5:1 (h:v) or less, and stabilized to withstand the passage of heavy equipment.

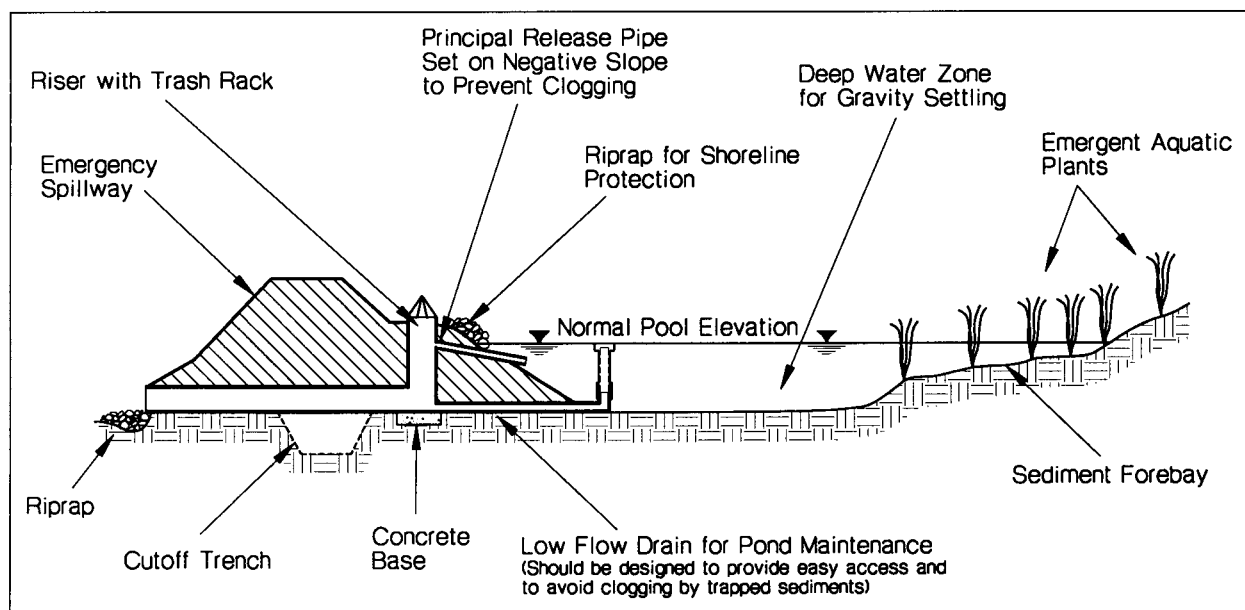
**■ MAINTENANCE:**

Wet-weather inspections should be conducted annually, with as-built plans in hand. Inspections should emphasize the condition of the extended detention control device and low flow pilot channel. Extended detention facilities should be maintained as a meadow to reduce mowing frequency (two times per year) and maintenance costs. Maintenance responsibilities should be clearly vested with funds reserved for both routine and non-routine activities.

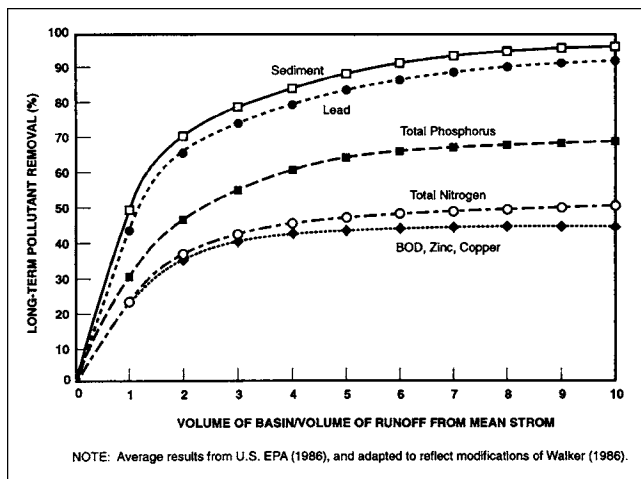
**■ SEDIMENT REMOVAL:**

A five to ten-year sediment clean-out cycle is recommended. Extra storage in the lower stage of the pond can be provided to accommodate sediment deposition. Also, on-site sediment disposal areas should be reserved to reduce removal costs. Do not begin final pond construction until upland area is stabilized.

Source: Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.



**FIGURE 22.31** Typical wet pond profile. (Courtesy of Engineers & Surveyors Institute and Northern Virginia Planning District Commission, 8401 Arlington Blvd. Fairfax, Va. 22031.)

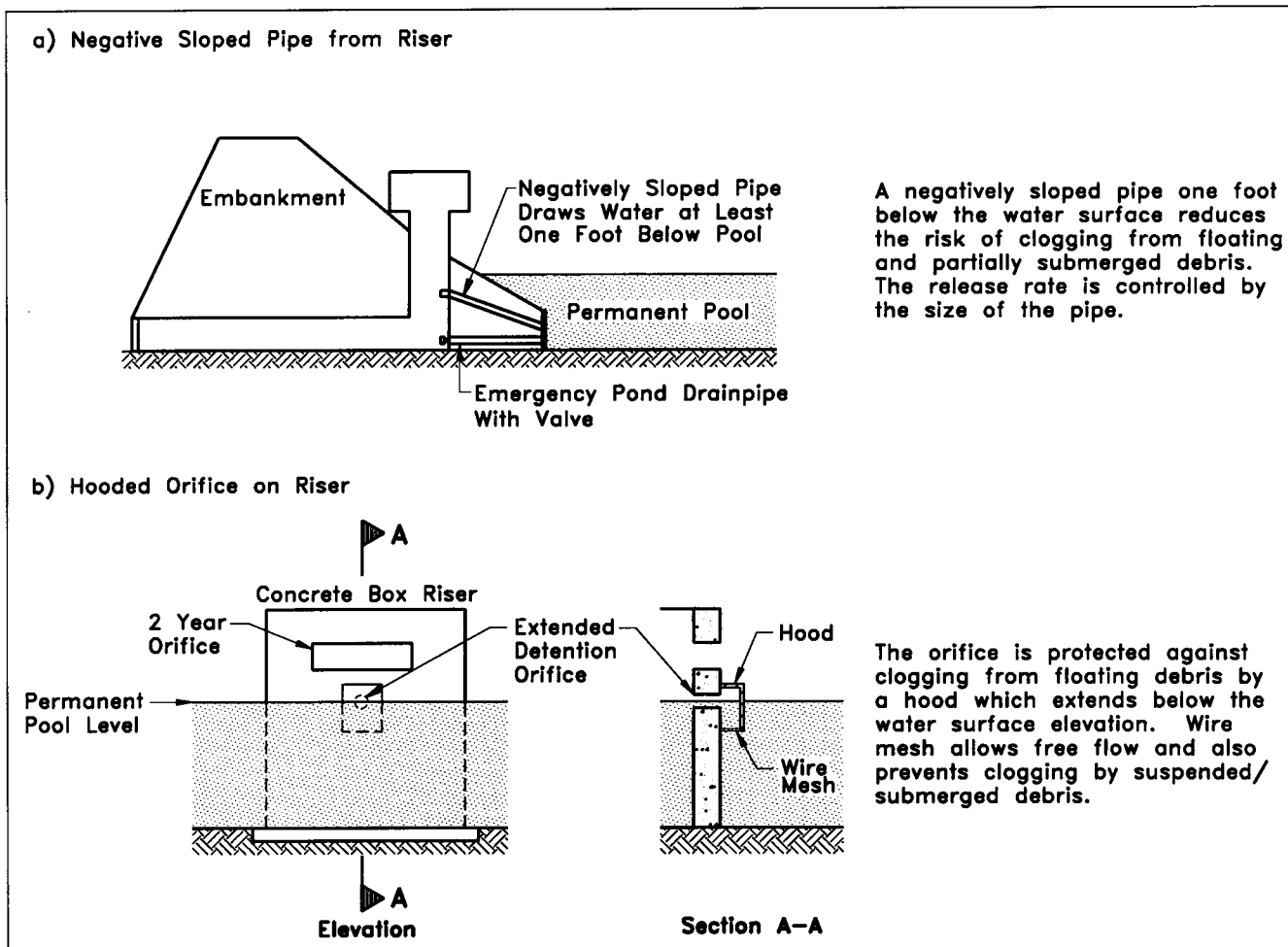


**FIGURE 22.32** Estimated removal of selected urban pollutants as a function of permanent pool size. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

aquatic plants to grow. Vegetation around the edges of the pond also provide the benefit of protecting against erosion. The shallow area may also serve as a safety feature. See Table 22.14 for summary of design features for wet ponds and Figure 22.34 for the schematic diagram.

**Infiltration Facilities**

Infiltration facilities include trenches, basins, and porous pavement, all of which are designed to allow a specified volume of runoff to be stored in the facility until it infiltrates into the ground. As such, these types of facilities are only feasible if the soils are permeable and if the water table is situated well below the ground surface. Because the volume of water does not drain directly to a stream or other surface water, both particulate and soluble forms of pollutants may be controlled. However, coarse sediments should be trapped before entering these types of facilities, as they may easily become clogged. In addition to providing BMP controls, infiltration trenches and basins can contribute to recharging groundwater, as well as augmenting low flow and helping to control bank erosion in streams. There is a pos-



**FIGURE 22.33** Method for extending detention time in wet ponds. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

**TABLE 22.13 Selected Criteria for Sizing the Permanent Pool Volume of a Wet Pond**

- a) One-half in of runoff distributed over the contributing watershed area (Montgomery County, MD).
- b) One-half in of runoff distributed over the impervious portion of the contributing watershed (MD WRA).
- c) Volume of permanent pool equivalent to a variable depth of runoff distributed over the contributing watershed, depending on land use (Fairfax County, VA).
- d) Two-and-a-half times the volume of runoff generated from the mean storm over the watershed area (MD WRA).
- e) Adjusted to achieve an average of two weeks of retention within the pond, or about four times the volume of runoff generated by the mean storm over the watershed area.

Source: Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

sible risk of groundwater contamination with these facilities, however, and they have been known to fail due to unsuitable soils or clogging from sediments.

**Infiltration Trenches.** These are constructed by excavating a trench, generally about 3 to 8 ft deep, and filling the excavation with stone. The resulting trench volume is designed with a storage capacity equivalent to a prescribed design storm runoff volume. Trenches can be designed to accept runoff entering as sheet flow along the length of the trench, or as inflow from a pipe or conduit. Infiltration trenches have a more limited capacity than infiltration basins and are generally used as an on-site control. They often may be suitable for placement along the perimeter or within the available open spaces of a development site, and in fact are more typically employed at small sites. Stormwater volume and peak runoff rates may be controlled by these facilities, but the large volume required may make other controls more acceptable.

A variation of the standard infiltration trench is a system with an underdrain of perforated pipe to assist in draining the runoff out of the trench. The underdrain augments the infiltration of stormwater into the subsoil. At one end, the underdrain is connected to a central outlet point.

This type of design will likely result in poor pollutant removal, because the residence time in the trench will be short, and the amount of stormwater infiltrating into the soil will be small compared to the amount draining through

the perforated pipe. However, a valve can be placed on such an underdrain outlet, and the underdrain system can then serve as a backup in case of clogging of the pores in the interface between the trench and the subsoils, or if there are concerns about the permeability of the soil.

Another design technique to incorporate a perforated drain pipe is to place the pipe near the top of the trench. This allows the volume of stormwater in the trench up to the level of the drain pipe to infiltrate into the subsoil. Any stormwater in excess of this volume will then be carried in the drain pipe to the outlet.

A provision in the design must be made to account for and adequately handle storm flows in excess of the capacity of the trench drain. If the infiltration trench is situated at an intermediate point along a slope, this may be accomplished by placement of an overflow berm on the downstream side of the trench. When a large runoff volume exceeds the capacity of the trench, it will flow over the berm and should then be allowed to flow to a drainage conveyance system (ditch, pipe inlet, stream, SWM pond, etc.). If the trench is placed along the invert of a flat swale, such as in the median of a road or parking lot, then a perforated pipe may be used to handle the excess flow. Overland relief must be provided so that runoff from very large storms does not cause unnecessary flooding or damage.

Figure 22.35 shows a typical schematic layout for an infiltration trench. As previously stated, the overall trench dimensions should be determined to provide the desired detention volume. Sizing guidelines for determining the appropriate volume for non-point-source pollution control are the same as for extended detention dry ponds, in which the goal is to capture the first flush of runoff and pollutants.

A grass filter strip is recommended along the inlet side(s) of an infiltration trench to filter out grit, oil, etc. It is recommended that the slope of the filter strip be no greater than about 5%. (Further information on grass filter strips is provided in the section on Vegetative Practices.) The sides, top, and bottom of the trench are lined with a permeable filter material, and the trench is filled with one to three-in washed stone aggregate (see Figure 22.36).

The rate of infiltration is probably the most significant factor in determining the applicability of using infiltration trenches for a site. In general, a trench should be designed to completely drain in about three days or less. This will help prevent anoxic conditions from occurring in the trench bottom and, therefore, help to maintain a bacterial population in the subsoil that will aid in pollutant removal. Also, as the trench is drained more quickly, it is sooner available to accept runoff from a subsequent storm. The soil types applicable for use with an infiltration trench may be checked by the SCS soil classification system. SCS HSG type A or B are most suitable as subsoils for this type of application. Soil type C is marginal, and soil type D is likely to be too impermeable for application in an infiltration trench. To enhance the performance of an infiltration trench, the surface area available for infiltration into the subsoils should be maximized. Therefore, to the extent possible with re-

**TABLE 22.14 Design Summary: Wet Ponds**

- **SIZE:**  
At a minimum, the volume of the permanent pool should be at least 2.5 times greater than the runoff volume generated by the mean storm ( $0.45 \text{ in}^* \text{ runoff coefficient}^* \text{ watershed acres}$ ). The contributing watershed area should be at least ten acres in size, the pond surface area at least  $\frac{1}{4}$  acre.
- **SHAPE:**  
The pond should be wedge-shaped, narrowest at the inlet and widest at the embankment. A minimum length-to-width ratio of 3:1 should be used unless gabion baffles are used to extend the flow path. Irregular shorelines are preferred.
- **DEPTH:**  
Pond depth should average three to six ft, with a shallow underwater bench (minimum ten ft wide) around the pond's perimeter. Water depths should be shallow near the inlet and deeper at the riser. Extremely deep ponds (average depth eight ft or more) should normally be avoided to prevent stratification and to minimize excavation costs.
- **SIDE SLOPES:**  
Side slopes should be no steeper than 3:1 (h:v) and not flatter than 20 to 1. A flat safety bench, at least ten ft wide, should be located near the toe of the slope.
- **SOILS:**  
If soils at the pond site are highly permeable (in the "A" or "B" hydrologic groupings), it may be necessary to line the bottom of the pond with an impermeable geotextile or a six-in clay liner. Often, impermeable fill soils can be found elsewhere in the development for this purpose.
- **RISER DESIGN:**  
Where feasible, wet ponds should be designed to accommodate 24 to 40 hours of extended detention for a one-in storm. Orifices used to maintain the permanent pool level should withdraw water at least one ft below the surface, and where environmental conditions warrant, close to the pond bottom. Hoods or trash racks should be installed on the riser to prevent clogging. For access and aesthetic reasons, the riser should be placed near or within the embankment. However, if the riser is located on the face of the embankment, fencing may be required. To reduce future maintenance requirements, concrete, rather than corrugated metal pipe, should be used for barrels and risers.
- **OUTFALL PROTECTION:**  
The channel immediately below the pond should be modified to conform to natural dimensions and lined with large rip-rap placed over filter cloth. Stilling basins, rock deflectors, check dams and other devices should be used to reduce flow velocities to non-erosive levels.
- **POND BUFFER:**  
Wet ponds should be surrounded by a buffer strip at least 25 ft wide. The buffer strip should be planted with water-tolerant, low-maintenance grasses, shrubs and trees.
- **VEGETATION:**  
When feasible, it is suggested that artificial marsh fringe be established near the inlet or forebay, and around at least 50% of the pond perimeter.
- **EMBANKMENT:**  
At least 10–15% extra fill should be allowed on the embankment to account for possible subsidence. The embankment should have at least one ft of freeboard above emergency spillway. Anti-seep collars should be used to prevent seepage around the barrel, and a core trench should be installed under the embankment to key it to the substrate. The embankment should be graded to allow access and mowed twice annually to prevent woody growth.
- **SITE ACCESS:**  
Adequate access from public or private right of way to the pond should be reserved. The access should be at least ten ft wide, on a slope of 5:1 or less, and stabilized to withstand the passage of heavy equipment. The access road should not cross the emergency spillway, unless it is properly stabilized.

**TABLE 22.14 Design Summary: Wet Ponds (Continued)**

■ ENVIRONMENTAL REVIEW:

Consult with planners and biologists to develop a design that mitigates impacts to downstream aquatic life. Prepare a landscaping plan that provides habitat requirements for wildlife and waterfowl and is attractive to local residents.

■ MAINTENANCE:

Wet weather inspections should be conducted annually, with as-built plans in hand. Maintenance responsibilities should be clearly vested, with funds reserved for both routine and non-routine tasks.

■ SEDIMENT REMOVAL:

Begin final construction after upland area has been stabilized. Construct a sediment forebay near the inlet of the pond with extra storage equal to the volume of projected sediment trapping over a 20 to 40-year period. Reserve on-site sediment disposal areas near the pond in the form of a site easement.

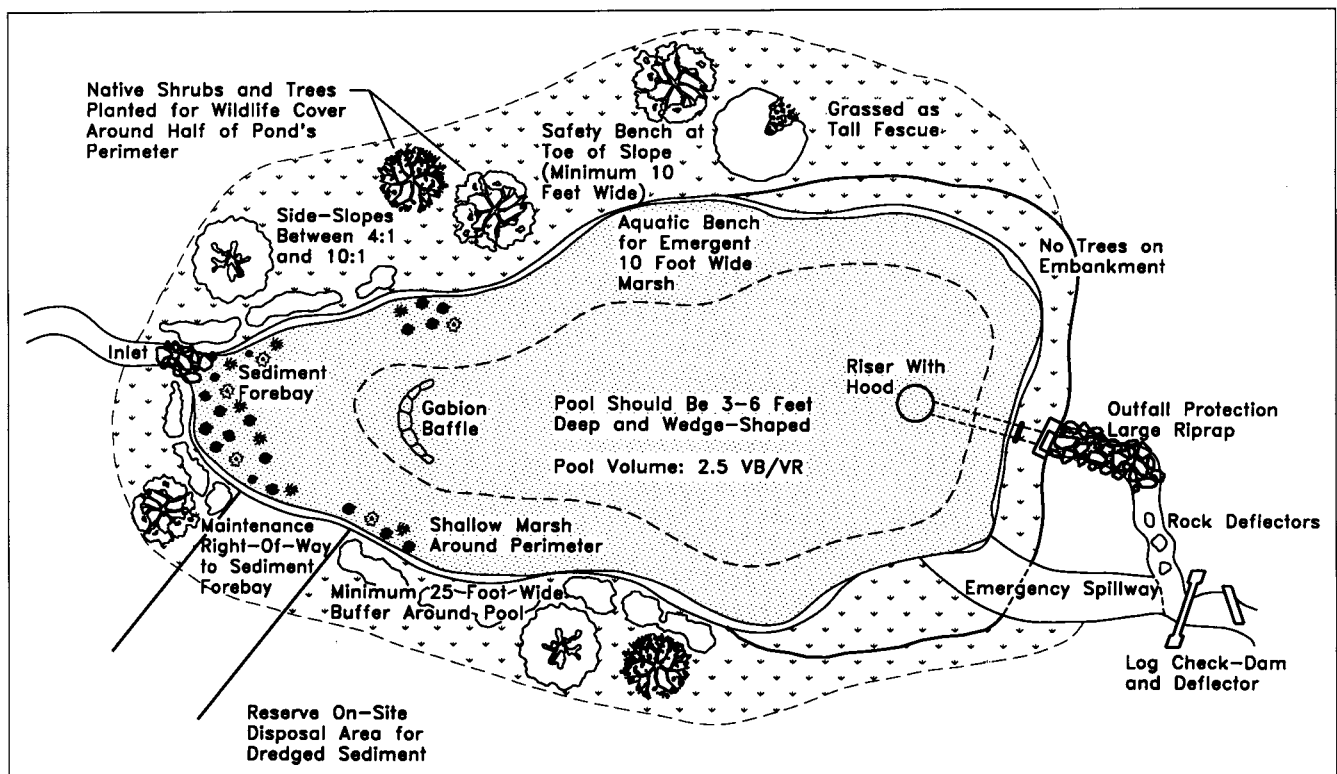
Source: Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

spect to the available land area, the trench bottom should be made as wide as possible. This also will increase the rate of infiltration into the subsoil and will reduce the risk of clogging of the interface between the trench and the subsoil. In addition, the bottom of the trench should have a flat slope and be located 2 to 4 ft above the seasonal high water table and/or bedrock.

Infiltration trenches should be placed such that no adverse impacts will be inflicted on drinking water wells or

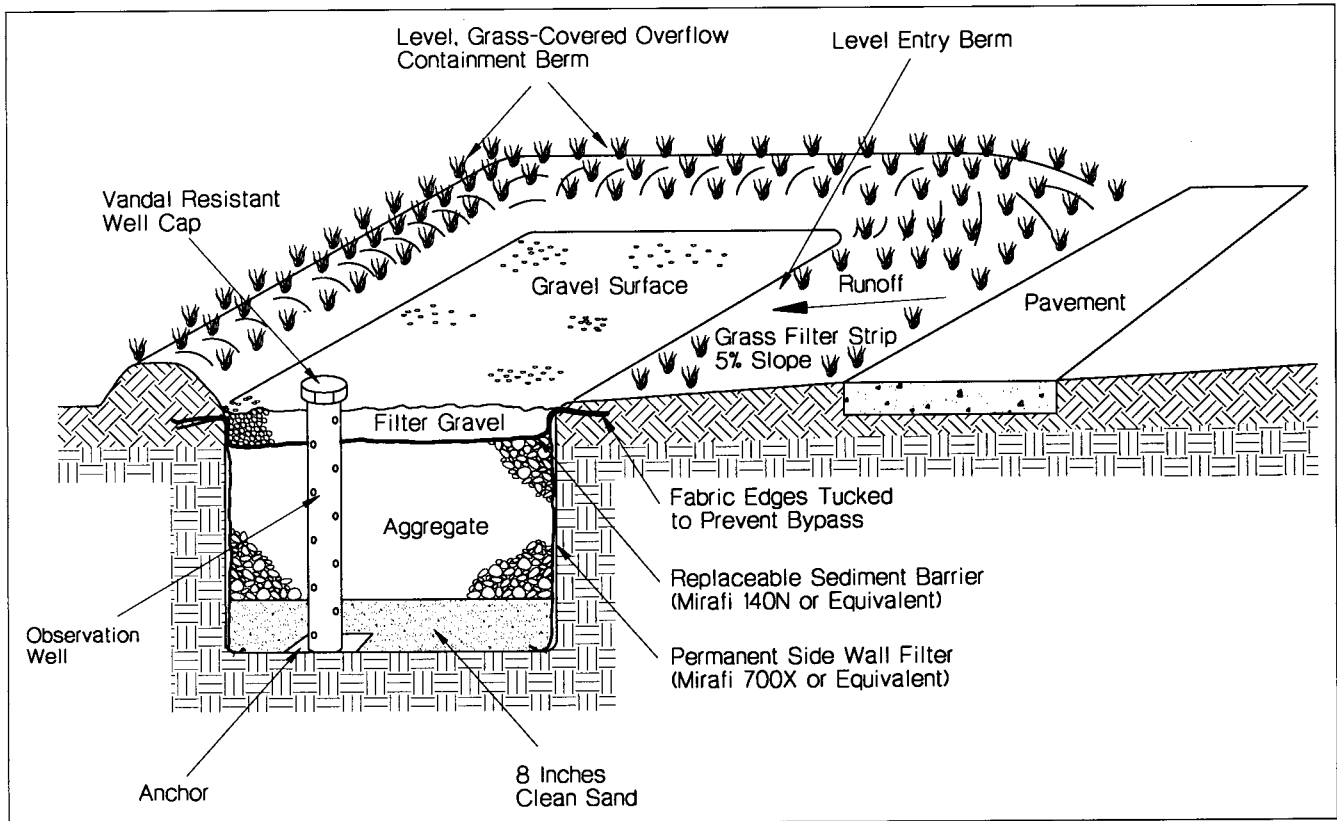
building foundations. Also, trees should be cut back away from infiltration trenches so that leaves do not fall on the surface and cause clogging, and so that roots do not puncture the filter fabric surrounding the trench.

**Infiltration Basins.** These are capable of treating stormwater runoff from a much larger contributing area than infiltration trenches. A basin is constructed by either excavating a storage area or creating one by placement of an embankment. These basins are designed with a storage

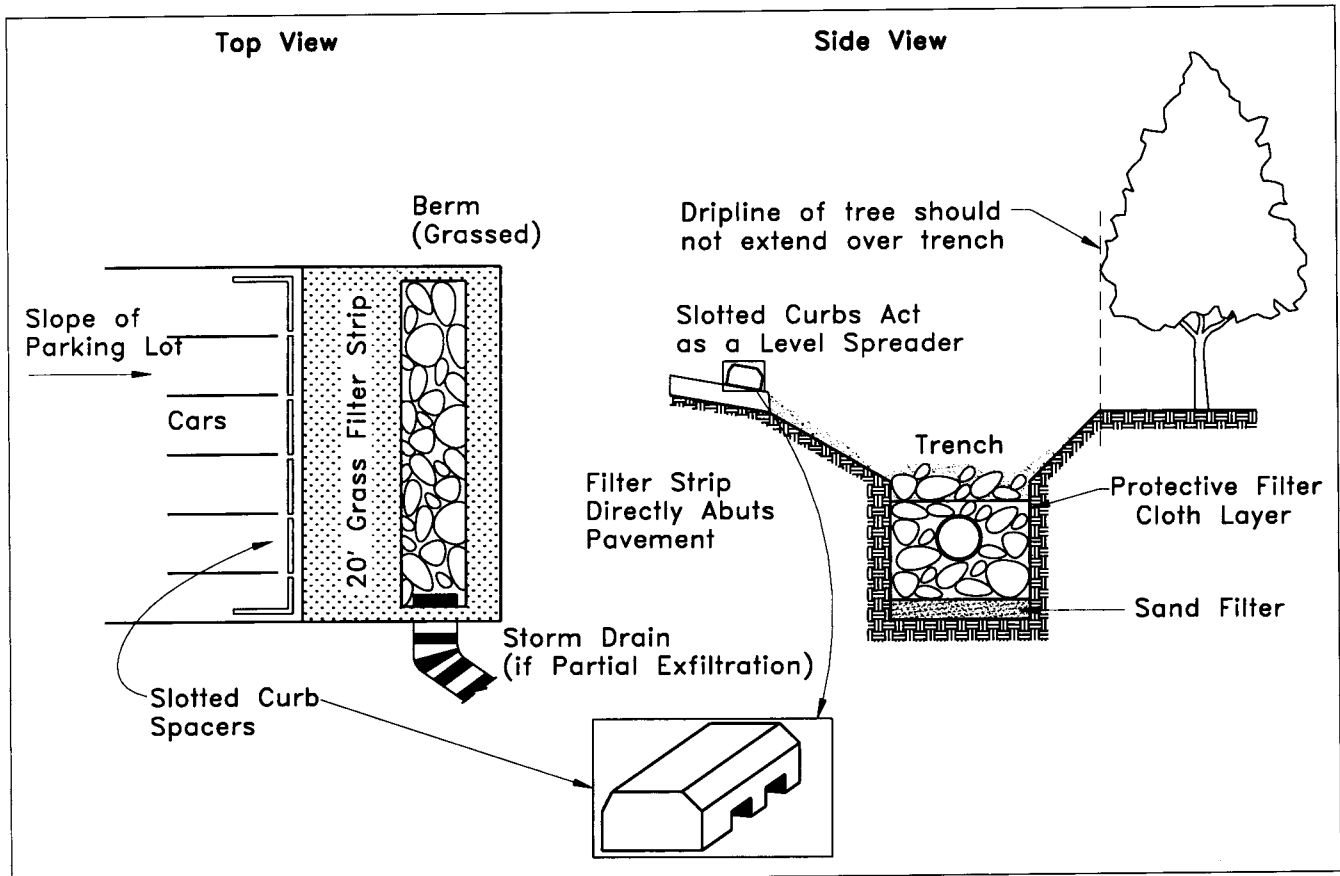


**FIGURE 22.34** Design schematic of a wet pond. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)





**FIGURE 22.35** Typical infiltration trench. (Courtesy of Engineers & Surveyors Institute and Northern Virginia Planning District Commission, 8401 Arlington Blvd. Fairfax, Va. 22031.)

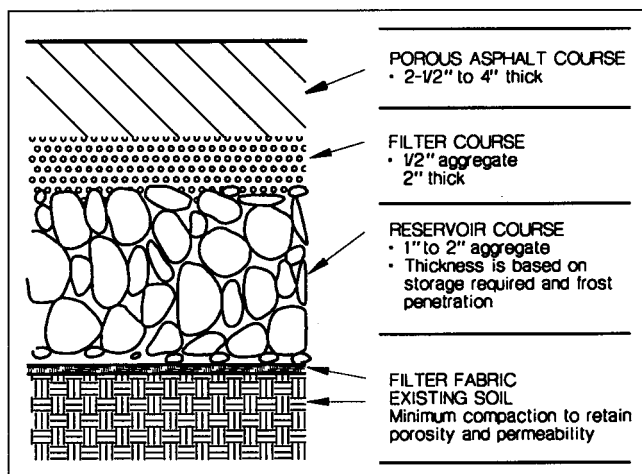


**FIGURE 22.36** Parking lot perimeter trench. (Courtesy of Schueler, Thomas R. 1987, *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Washington, DC: Metropolitan Washington Council of Governments, 777 No. Capital St. NE, Suite 300, Washington, DC 20002-4226, 202/962-3256.)

capacity equal to a prescribed design storm runoff volume and allow the runoff to infiltrate into the soil. These types of facilities are also capable of being modified to provide peak flow rate reduction by incorporating a spillway outlet system, as with a conventional dry pond. Runoff quantities in excess of the design storm volume pass through the basin via an emergency spillway outlet. One advantage of infiltration basins is that they can most closely reproduce pre-developed flow conditions by providing water quality improvement, groundwater recharge, and runoff volume and peak flow rate reductions. However, they are reported to have one of the higher failure rates of the different types of BMPs available.

All of the restrictions regarding soil types, sediment controls, depth to water table and bedrock, proximity to buildings and drinking wells, and the information regarding functioning of these facilities, is identical to those listed in the previous discussion on infiltration trenches. In addition, construction and maintenance of an embankment and emergency spillway for an infiltration basin should be in accordance with the requirements discussed in the SWM section for quantity control.

**Porous Pavement.** (Figure 22.37). Like infiltration trenches, porous pavement allows runoff to be stored in an excavated, stone-filled reservoir, from which infiltration into the subsoil takes place. It is generally used for managing runoff from low volume parking areas, although some additional runoff from other areas such as rooftops or adjacent parking areas may be handled in some cases. High-volume traffic areas should be paved by conventional methods, and the runoff can be directed to the porous pavement areas if necessary. Porous pavement will perform comparable to conventional pavement when properly designed and installed. Further information on installing porous pavement may be found in the publication *Porous Pavement: Principles of Development and Design Manual*, published in 1978 by the Franklin Institute Press, Philadelphia, PA.



**FIGURE 22.37** Schematic of typical porous pavement section. (Courtesy of Engineers & Surveyors Institute and Northern Virginia Planning District Commission, 8401 Arlington Blvd. Fairfax, Va. 22031.)

The advantages of porous pavement include saved costs, because of reduced or eliminated curbs and gutters, and safer pavement for driving, because of less accumulated water, which can cause skidding and hydroplaning. One disadvantage is that if clogging occurs, the maintenance required to clean out the deposited sediment may be difficult and costly. Also, the construction must be performed with good workmanship to assure proper functioning of the porous pavement.

To prevent clogging, it is recommended that the surface of porous pavement be vacuum swept and washed with a high pressure hose at least four times per year. In addition, coarse grained particles such as sediments are not intended to be removed by porous pavement infiltration systems.

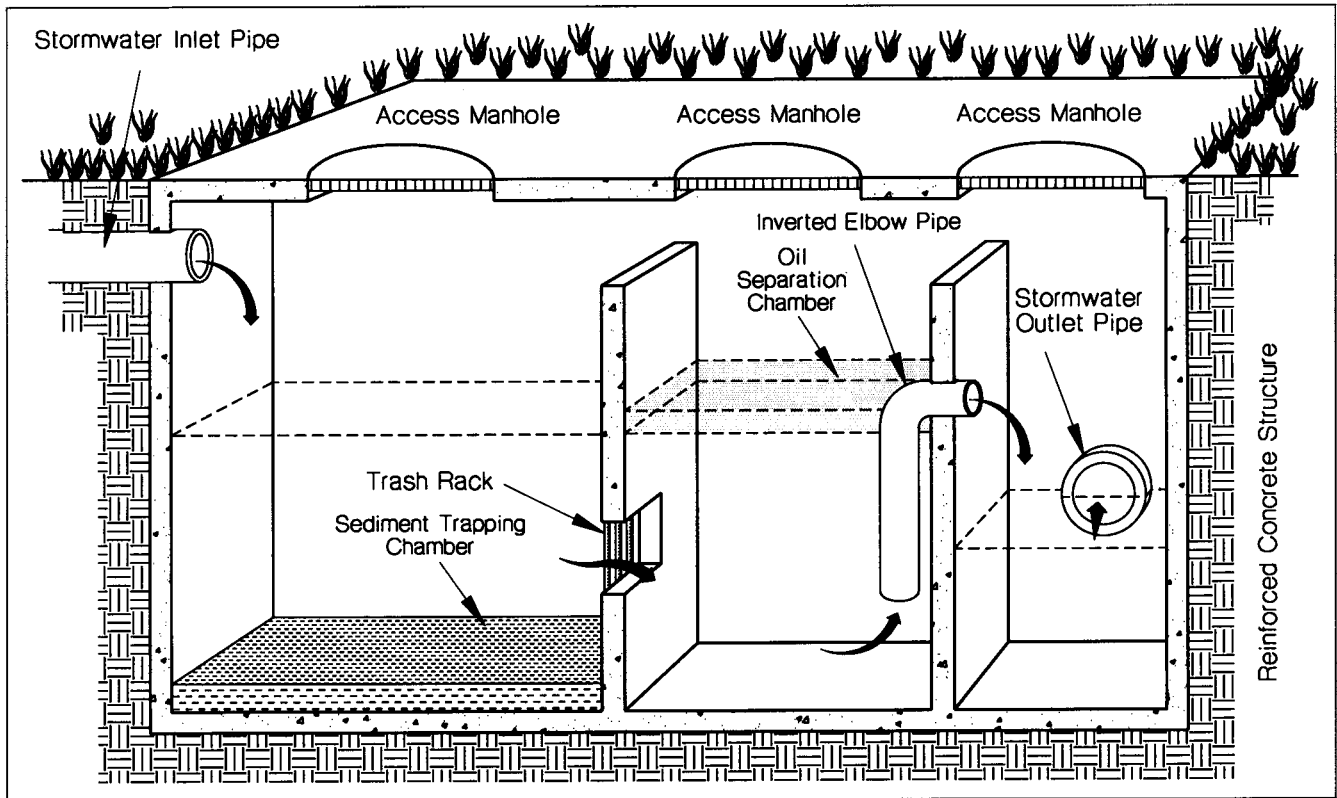
All of the restrictions regarding soil types, sediment controls, depth to water table and bedrock, proximity to buildings and drinking wells, and the information regarding functioning of these facilities, is identical to those listed in the previous discussion on infiltration trenches. During construction, it is important that measures be taken to prevent compacting the subgrade by construction equipment. If the subgrade becomes compacted, the infiltration rates may be reduced to unacceptable levels.

### Water Quality Inlets

Water quality inlets are also known as oil/grit separators and are intended to separate out oil and other hydrocarbons, grit, and coarse sediments from runoff. They are often used to pre-treat runoff from parking lots before it enters an infiltration device or storm sewer system. Because of the short retention time of runoff, they are effective only in reducing the amounts of coarse grained sediments, debris, oil, and grease present in runoff. Water quality inlets must be cleaned out frequently, at least twice a year, to remove collected grit and pollutants for disposal.

A schematic diagram of a typical three-chamber water quality inlet is shown in Figure 22.38. Runoff enters the first chamber through a storm drain pipe or curb inlet, passes through two screened orifice openings to the second chamber, then passes through an inverted pipe elbow to the third chamber. Flow exits the third chamber through a storm drain outlet pipe. The first two chambers both contain a permanent pool of water three to four ft deep, and the third chamber should as well, if possible, to maximize the opportunities to settle out pollutants.

The first chamber is used to trap floatable debris and settle coarse particles such as sediment. The second chamber is used to separate out oil, grease, and other hydrocarbons that float to the surface while the runoff is withdrawn from the opening of the elbow several feet below. The floating hydrocarbons remain in this chamber until removed at clean-out or adsorbed by sediment and settled out. The third chamber will have a permanent pool if the outlet pipe is elevated above the floor. Additional sediments may settle out in this chamber. An overflow opening is included in each chamber for large stormwater flows to bypass the orifice openings between chambers.



**FIGURE 22.38** Schematic of a water quality inlet. (Courtesy of Engineers & Surveyors Institute and Northern Virginia Planning District Commission, 8401 Arlington Blvd. Fairfax, Va. 22031.)

It is recommended that water quality inlets be sized to provide at least 400 ft<sup>3</sup> of permanent wet storage for each acre of contributing watershed. Access for clean-out is available through manhole openings. A variation in this basic design is to provide weep holes in the bottom of the chambers with permanent pools, so that the runoff may drain out of the chambers, through a gravel layer, and infiltrate into the soil. However, it is likely that the holes would soon become clogged by sediment and debris, and the inlet would then function like the standard. Also, some risk of groundwater contamination exists if infiltration into the soil occurs.

### Sand Filters

Another type of BMP rapidly gaining acceptance is the intermittent sand filter (see Figure 22.39). Austin, Texas and Florida have been using sand filters to treat stormwater runoff since the early 1980s. Other localities, such as Washington, D.C., Alexandria, VA and Delaware, have adopted sand filters for use as BMPs. Although the sand filters for each locality vary in design and configuration, the basics of the operations are the same.

The typical sand filter has a gravel layer overlain on a layer of sand. The two filter medias are separated with a geotextile fabric to prevent the sand from intermixing with the gravel. Beneath the filtering media is a collection pipe system to convey the filtered runoff to the discharge point.

Many pollutants, such as heavy metals and suspended solids, are physically removed as the water is “strained” through the filter media. However, other pollutants, such as phosphorus, nitrogen and BOD, are removed through chemical processes and adsorption.

### Vegetative Practices

Various vegetative BMPs can provide some measure of water quality enhancement for a site, as well as improve aesthetics. Included as vegetative practices are grassed swales and filter strips.

Grassed Swales are utilized to convey stormwater, often as an alternative to a curb and gutter or storm sewer. They are less expensive than other conveyance systems but have only a limited range of flows/velocities over which they may be used. Large flows with erosive velocities should be handled by other methods.

Pollutants may be removed from runoff in grassed swales by the filtering action of the grass, deposition in low-velocity areas, and by infiltration into the subsoil. To enhance the efficiency of pollutant removal, swales should be constructed with the minimum slope practical to allow positive drainage. Side slopes of the swale should also be minimized and should be no steeper than 3:1 to allow mowing. A dense cover of grass should be maintained to filter pollutants and prevent erosion. Check dams can be placed along the swale to slow flow velocities and promote infil-

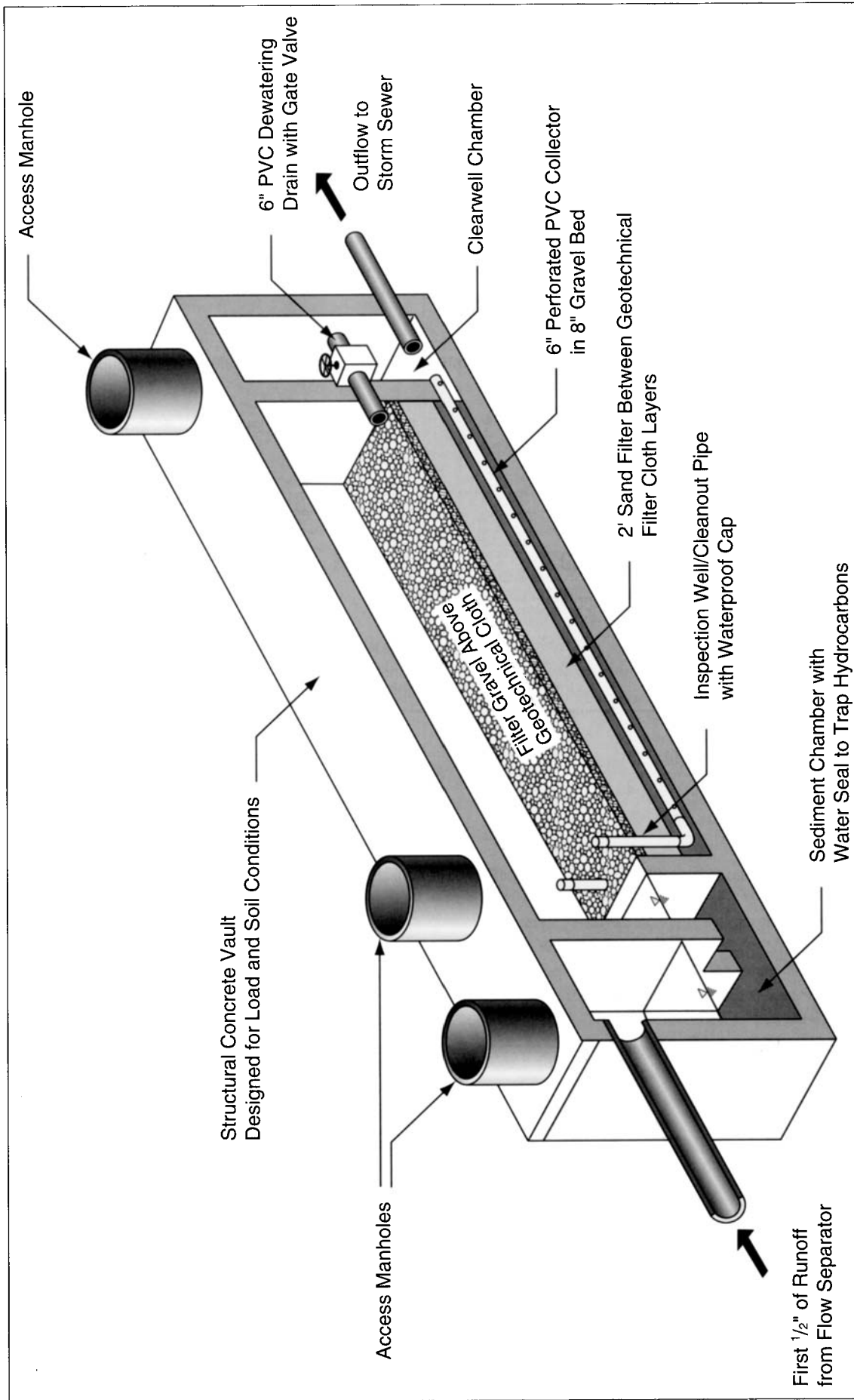


FIGURE 22.39 Sand filter.

tration. Additional measures such as riprap may be necessary on the downstream side of the check dams to prevent erosion.

Filter strips are vegetated strips along a slope or flowpath designed to convey sheet flow and filter out pollutants. They may be used to filter runoff before it enters a stream or other stormwater conveyance system and are often used to filter runoff before entering an infiltration-type BMP facility. Pollutant removal occurs by the same mechanisms as for grassed swales, and the design guidelines for enhancing pollutant removal for grassed swales basically apply to filter strips as well. It is important that flow over filter strips flows as sheet flow and not concentrated flow, as the latter would effectively short-circuit the filter strip, and little pollutant removal would likely occur. Filter strips can be virtually cost free if preserved before any development activities take place, and they can be effective at filtering out pollutants with either grassed or forested vegetation.

To ensure that sheet flow occurs across filter strips, they should be graded to a uniform, even, and relatively flat slope. Level spreaders may be used to spread concentrated flows out before entering a filter strip. If a cross slope exists along the strip, berms may be placed perpendicular to the strip to prevent flows from concentrating. The contributing drainage area to a filter strip should be relatively small to further inhibit concentrated flow from forming.

The pollutant removal efficiencies for filter strips are not easily quantified, but forested filter strips are reported to have greater pollutant removal capability than grass strips, due to the greater uptake and long-term retention of nutrients in forest biomass. However, because the amount of vegetative cover is less in forested areas than in grassy areas, a greater filter strip length is recommended for forested areas than for grass.

There are no standard rules for the preferred length of filter strip required for pollutant removal. Strips should be no less than 20 ft in length, but it is recommended that 50 to 75-ft lengths be used. For very short lengths, flat slopes are mandatory for pollutant removal to occur. A rule of thumb for steeper slopes is to add 4 ft beyond the 50 to 75 ft for every 1% increase in slope. Grades of 5% or less are recommended for filter strip application, while slopes of 15% or greater will preclude the pollutant removal function of filter strips.

A summary of the effectiveness of various types of BMP facilities appears in Table 22.15.

### BMP Design Example

Using the same 50-acre parcel of the detention example, determine the annual phosphorus load from the site for both pre-developed and post-development conditions. What type(s) of BMP facility should be used to maintain the pre-development pollutant load level? Size a BMP facility to reduce the post-development phosphorus load by 40%.

The average annual rainfall is 46 in. The imperviousness of the site for pre-developed conditions is estimated at 2%, and the total phosphorus concentration is estimated at 0.15 mg/l (from Table 22.9 under total phosphorous for hardwood forest). Using Eq. (22.46), the pre-development annual phosphorus load is

$$\begin{aligned} L_{\text{pre}} &= \left( \frac{(46)(0.9)(0.05 + 0.009(2))}{12} \right) (0.15)(50)(2.72) \\ &= 4.8 \text{ lb/yr} \end{aligned} \quad (22.44)$$

For the post-development conditions, the estimated imperviousness is 10% for residential areas and 75% for commercial areas, and the phosphorus concentration is 0.26 mg/l. A weighted impervious value is

$$\text{Weighted } I = \frac{10\% (40 \text{ acre}) + 75\% (10 \text{ acre})}{50 \text{ acre}} = 23\% \quad (22.45)$$

The post-development phosphorus load is

$$\begin{aligned} L_{\text{post}} &= \left( \frac{(46)(0.9)(0.05 + 0.009(23))}{12} \right) (0.26)(50) 2.72 \\ &= 31.3 \text{ lb/yr} \end{aligned} \quad (22.46)$$

and this results in a phosphorus load increase of 26.5 lb/yr. To achieve no net increase in phosphorus load, the post-development phosphorus load, must be reduced by 26.5 lb/yr or approximately 85% decrease relative to the pre-development load level. According to the data presented in Schuler (1987), no single BMP facility is capable of 85% removal efficiency. A combination of two or more facilities in series might be a consideration, such as an infiltration trench followed by an ED pond. Recognize that this would use more land. Regardless of the number of facilities used in series, most of the soluble pollutants will not be captured.

Another approach offered by local jurisdictions is to implement "regional" pollutant removal goals where required pollutant removal is not determined by pre- and post-development conditions of a particular site but by average watershed conditions and/or a preset value. For the required 50%, phosphorus removal efficiency equates to reducing the post-development annual phosphorus load by 50%.

Using the methodology of Schuler (1987), an extended detention pond sized to detain runoff volume produced by 1 in. of rainfall for 24 hr will provide removal efficiency on

**TABLE 2.2.15 Comparative Effectiveness of Urban BMP Practices**

<b>URBAN BMP OPTIONS*</b>	<b>RELIABILITY FOR POLLUTANT REMOVAL</b>	<b>LONGEVITY*</b>	<b>APPLICABLE TO MOST DEVELOPMENTS</b>	<b>WILDLIFE HABITAT POTENTIAL</b>	<b>ENVIRONMENTAL CONCERNS</b>	<b>COMPARATIVE COST</b>	<b>SPECIAL CONSIDERATIONS</b>
STORMWATER WETLANDS	Moderate to high depending on design.	20+ years expected.	Applicable to most sites if land is available.	High.	Stream warming; natural wetland alteration.	Marginally higher than wet ponds.	Recommended with design improvements and with the use of micropools and wetlands.
EXTENDED DETENTION PONDS	Moderate, but not always reliable.	20+ years, but frequent clogging and short detention common.	Widely applicable, but requires at least 10 acres of drainage area.	Moderate.	Possible stream warming and habitat destruction.	Lowest cost alternative in size range.	Recommended with design improvements and with the use of micropools and wetlands.
WET PONDS	Moderate to high.	20+ years.	Widely applicable, but requires drainage area of greater than 2 acres.	Moderate to high.	Possible stream warming, trophic shifts, habitat.	Moderate to high compared to conventional.	Recommended, with careful site evaluation.
MULTIPLE POND SYSTEMS	Moderate to high; redundancy increases reliability.	20+ years.	Widely applicable.	Moderate to high.	Selection of appropriate pond option minimizes overall environmental impact.	Most expensive pond option.	Recommended.
INFILTRATION TRENCHES	Presumed moderate.	50% failure rate within 5 years.	Highly restricted (soils, groundwater, slope, area, sediment input).	Low.	Slight risk of groundwater contamination.	Cost-effective on smaller sites. Rehab costs can be considerable.	Recommended with pretreatment and geotechnical evaluation.

**T A B L E 2 2 . 1 5 Comparative Effective of Urban BMP Practices (Continued)**

<b>URBAN BMP OPTIONS<sup>a</sup></b>	<b>RELIABILITY FOR POLLUTANT REMOVAL</b>	<b>LONGEVITY*</b>	<b>APPLICABLE TO MOST DEVELOPMENTS</b>	<b>WILDLIFE HABITAT POTENTIAL</b>	<b>ENVIRONMENTAL CONCERNS</b>	<b>COMPARATIVE COST</b>	<b>SPECIAL CONSIDERATIONS</b>
INFILTRATION TRENCHES	Presumed moderate, if working.	50% failure rate within 5 years.	Highly restricted (soils, ground water, slope, sediment input).	Low.	Slight risk of groundwater contamination.	Cost-effective on smaller sites. Rehab costs can be considerable.	Recommended with pretreatment and geotechnical evaluation.
INFILTRATION BASINS	Presumed moderate, if working.	60–100% failure within 5 years.	Highly restricted (see infiltration trench).	Low to moderate.	Slight risk of groundwater contamination.	Construction cost moderate, but rehab cost high.	Not widely recommended until longevity is improved.
POROUS PAVEMENT	High (if working).	75% failure within 5 years.	Extremely restricted (traffic, soils, groundwater, slope, area, sediment input).	Low.	Possible groundwater impacts; uncontrolled runoff.	Cost-effective compared to conventional asphalt when working properly.	Recommended in highly restricted applications with careful construction and effective maintenance.
SAND FILTERS	Moderate to high.	20+ years.	Applicable (for smaller developments).	Low.	Minor.	Comparatively high construction costs and frequent maintenance.	Recommended, with local demonstration.
GRASSED SWALES	Low to moderate, but unreliable.	20+ years.	Low-density development and roads.	Low.	Minor.	Low compared to curb and gutter.	Recommended, with checkdams, as one element of a BMP system.
FILTER STRIPS	Unreliable in urban settings.	Unknown, but may be limited.	Restricted to low-density areas.	Moderate if forested.	Minor.	Low.	Recommended as one element of a BMP system.

WATER QUALITY INLETS	Presumed low.	20+ years.	Small, highly impervious catchments (<2 acres).	Low.	Resuspension of hydrocarbon loadings. Disposal of hydrocarbon and toxic residuals.	High, compared to trenches and sand filters.	Not currently recommended as a primary BMP option.
----------------------	---------------	------------	---	------	--	--	--

Source: Courtesy of Galli, John. 1982, *Analysis of Urban BMP Performance and Longevity in Prince George's County, Maryland, Washington, DC*. Metropolitan Washington Council of Governments, 777 No. Capitol St. NE, Suite 300, Washington, DC 20002-4226. 202/962-3256.

\*Based on current designs and prevailing maintenance practices.



the order of 40%. The runoff volume from 1 inch of rainfall is

$$\begin{aligned} \text{Vol} &= \frac{P(R_v)(A)}{12} && (22.47) \\ &= \frac{(1)(0.05 + 0.009(39))(50)}{12} \\ &= 1.67 \text{ acre-ft of storage} \end{aligned}$$

Recommendation for this example is to use an ED pond that detains 1.67 acre-ft of runoff and slowly release it over 24 hr to achieve the 40% annual phosphorus reduction for the post-development conditions. Note that the post-development total phosphorus load from the site after release from the extended detention facility is

$$L_{\text{post}} = 27.3 \text{ lb/yr} \left( \frac{40}{100} \right) = 10.9 \text{ lb/yr} \quad (22.48)$$

which is an increase over the pre-development phosphorus load of 6.1 lb/yr.

This example for sizing a SWM facility for quality control, and the previous example for sizing a facility for quantity control, were presented separately to better illustrate the mechanics of the procedures. If the same facility is to serve both purposes, the volume for water quality would be determined first.

## REFERENCES<sup>1</sup>

American Public Works Association, *Urban Stormwater Management*, Special Report No. 49, Chicago, Illinois, 1981.

American Society of Civil Engineers, *Stormwater Detention Outlet Control Structures*, American Society of Civil Engineers, New York, 1985.

Bell, Warren, *Intermittent Sand Filter BMPS for Stormwater Quality*, Engineers and Surveyors Institute Stormwater Management Council, Stormwater Management Symposium, Sept. 8, 1994.

Brater, E.F., and H.W. King, *Handbook of Hydraulics for the Solution of Hydraulic Engineering Problems*, 6 ed. McGraw-Hill, New York, 1976.

Chow, V.T., *Open-Channel Hydraulics*, McGraw-Hill, New York, 1959.

Chow, Ven Te, David R. Maidment, and Larry W. Mays, *Applied Hydrology*, McGraw-Hill, New York, 1988.

Driver, Nancy E. and Gary D. Tasker, *Techniques for Estimation of Storm-Runoff Loads, Volumes, and Selected Constituent Concentrations in Urban Watersheds in the United States*, U.S. Geologi-

cal Survey Water-Supply Paper 2363, U.S. GPO, Washington, D.C., 1990.

Federal Highway Administration, *Debris-Control Structures*, Hydraulic Engineering Circular No. 9, Washington, D.C., 1971.

Federal Highway Administration, *Drainage of Highway Pavements*, Hydraulic Engineering Circular No. 12, Washington, D.C., 1984.

Federal Highway Administration, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular No. 14, Washington, D.C., 1983.

Federal Highway Administration, *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5, Washington, D.C., 1985.

Federal Emergency Management Agency, *Flood Insurance Study Guidelines and Specifications for Study Contractors*, Washington, D.C.

Federal Emergency Management Agency, *Appeals, Revisions, and Amendments to Flood Insurance Maps, A Guide for Community Officials*, Washington, D.C., 1985.

Fread, D.L., *DAMBRK, the NWS Dam-Break Flood Forecasting Model*, National Weather Service, Silver Spring, Md., 1979.

Galli, John, *Analysis of Urban BMP Performance and Longevity in Prince Georges's County, Maryland*, Metropolitan Washington Council of Governments, Washington, D.C., 1982.

Garman, Gayle D., Gregory B. Good and Linda M. Hinsman, *Phosphorus: A Summary of Information Regarding Lake Water Quality*, Illinois Environmental Protection Agency, Springfield, Illinois, 1986.

Gundlach, D.L., and W.A. Thomas, *Guidelines for Calculating and Routing a Dam-Break Flood*, U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1977.

Hammer, Mark J., and Kenneth A. MacKichan, *Hydrology and Quality of Water Resources*, John Wiley & Sons, New York, 1981.

Heaney, J.P., W.C. Huber, and S.J. Nix, *Stormwater Management Model: Level I—Preliminary Screening Procedures*, EPA Rep. No. 66/2-76-275, U.S. Environmental Protection Agency, Washington, D.C., October, 1976.

Huber, W.C., J.P. Heaney, S.J. Nix, R.E. Dickinson, and D.J. Polmann, *Storm Water Management Model User's Manual*, Version III, EPA-600/2-84-109a, U.S. Environmental Protection Agency, Athens, Ga., 1981.

Hydrologic Engineering Center, *Urban Storm Water Runoff: STORM, Generalized Computer Program 723-58-L2520*, U.S. Army Corps of Engineers, Davis, Ca., 1975.

Johanson, R.C., J.C. Imhoff, and H.H. Davis, *Users Manual for Hydrological Simulation Program-FORTRAN (HSPF)*, EPA-600/9-80-015, U.S. EPA, Athens, Ga., 1980.

Kibler, David F., ed., *Urban Stormwater Hydrology*, Water Resources Monograph 7, American Geophysical Union, Washington, D.C., 1982.

Linsley, Ray K., and Joseph B. Franzini, *Water Resources Engineering*, 3d ed., McGraw-Hill, New York, 1979.

Metropolitan Washington Council of Government, *Urban Runoff in the Washington Metropolitan Area*, Final Report, Washington, D.C. Area Urban Runoff Project; EPA Nationwide Urban Runoff Program, prepared by Metropolitan Washington Council of Governments, December, 1983.

National Weather Service, *Five to 60-Minute Precipitation Frequency for the Eastern and Central United States*, (NWS Hydro-35), Washington, D.C., 1977.

<sup>1</sup> Some of the references listed below are government documents. In addition to the individual agencies, some of the government publications may be available through the National Technical Information Service (NTIS) or the Government Printing Office (GPO) for a minimal charge. For availability and price, contact: National Technical Information Service (703) 487-4650 and U.S. Government Printing Office (202) 783-3238.

- Ponce, Victor Miguel, *Engineering Hydrology. Principals and Practices*, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1989.
- Sartor, J.D., and G.B. Boyd, *Water Pollution Aspects of Street Surface Contaminants*, Environmental Protection Series, Environmental Protection Agency, Report EPA-R-2-272-081, Washington, D.C., 1972.
- Schueler, Thomas R., *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*, Metropolitan Washington Council of Governments, 1987.
- Schueler, Thomas R., *Design of Stormwater Wetlands Systems Guidelines for Creating Diverse and Effective Stormwater Wetland Systems in the Mid-Atlantic Region*, Metropolitan Council of Governments, Washington, D.C., 1992.
- Shelley, P. E., *Monitoring Requirements, Methods, and Costs, Area-wide Assessment Procedures Manual*, EPA-600/9-76-014, Water Planning Division, U.S. Environmental Protection Agency, 1976.
- Stahre, Peter, and Ben Urbonas, *Stormwater Detention for Drainage, Water Quality, and CSO Management*, Prentice Hall, Englewood Cliffs, N.J., 1994.
- U.S. Department of Agriculture, *Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years* (Technical Paper No. 40), Washington, D.C., 1963.
- U.S. Department of Agriculture, *Simplified Dam-Breach Routing Procedure*, Technical Release Number 66, Washington, D.C., 1979.
- U.S. Department of Agriculture, *Earth Dams and Reservoirs*, Technical Release No. 60, Washington, D.C., 1985.
- U.S. Department of Agriculture, *National Engineering Handbook*, Section 4, Hydrology, Washington, D.C., 1985.
- U.S. Department of Agriculture, *National Engineering Handbook*, Section 5, Hydraulics, Washington, D.C.
- U.S. Department of Agriculture, *Urban Hydrology for Small Watersheds*, Technical Release 55, Washington, D.C., 1986.
- U.S. Department of the Interior, *Geological Survey Guidelines for Determining Flood Flow Frequency*, Bulletin #17B of the Hydrology Subcommittee, Interagency Advisory Committee on Water Data, 1982.
- U.S. Department of the Interior, Bureau of Reclamation, *Design of Small Dams*, 3d ed., U.S. Government Printing Office, 1987.
- U.S. Department of Transportation and Environmental Services, *Alexandria Supplement to the Northern Virginia BMP Handbook*, Dept. of Transportation and Environmental Services, Alexandria, Va., 1992.
- U.S. Environmental Protection Agency, *Report to Congress: Non-point Source Pollution in the United States*, Washington, D.C., January, 1984.
- U.S. Environmental Protection Agency, *Results of the Nationwide Urban Runoff Program*, Final Report, Washington, D.C., December, 1983.
- U.S. Environmental Protection Agency, *Lake and Reservoir Restoration Guidance Manual*, First Edition, Washington, D.C., 1988.
- U.S. Geological Survey, *Roughness Characteristics of Natural Channels*, Water Supply Paper 1849, U.S. Gov't. Printing Office, Washington, D.C., 1967.
- Urbonas, Ben, and L.A. Roesner, eds., *Urban Runoff Quality—Impact and Quality Enhancement Technology*, Proceedings of a Conference at Henniker, N.H., Published by the American Society of Civil Engineers, New York, 1986.
- Viessman, Warren, Jr., and Mark W. Hammer, *Water Supply and Pollution Control*, 4th ed., Harper & Row, New York, 1985.
- Walesh, Stuart G., *Urban Surface Water Management*, John Wiley & Sons, Inc., New York, 1989.
- Wanielista, Martin P., and Yousef A. Yousef, John Wiley & Sons, New York, 1993.
- Wetmore, J.N., and D.L. Fread, *The NWS Simplified Dam Break Flood Forecasting Model for Desk-top and Hand-held Microcomputers*, Printed and distributed by the Federal Emergency Management Agency, undated.
- Whipple, William, Neil S. Grigg, Thomas Grizzard, Clifford W. Randall, Robert P. Shubinski, L. Scott Tucker, *Stormwater Management in Urbanizing Areas*, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1983.



## CHAPTER 23

# FLOODPLAIN STUDIES

*Philip C. Champagne, P.E. / Gilbert R. Jones, P.E.*

### INTRODUCTION

Historically, development has occurred near the water, whether it was a navigable stream or a coastline, for recreation or for commerce. Although the benefits of being on the water are numerous, so too are the potential problems created by being so close to such a powerful entity. Floodplain studies are used to understand the potential hazard of being near the water and to identify where the floodwaters will go after a major storm.

As discussed previously within the chapter on Stormwater Management, changing land use from vacant rural land to residential/commercial use inevitably results in a decrease in pervious surface and an increase in impervious surface. This change in basin imperviousness results in increased flow rates, runoff volumes, the frequency of flooding, and the degradation of surface water quality. In order to accommodate the increased runoff, systems consisting of curb and gutter, storm sewer, ditches and paved channels are developed to convey the runoff safely through the basin. These systems directly increase flow velocity and consequently decrease the basin time of concentration resulting in higher peak flow rates.

In a typical moderately-developed watershed, the increase in peak discharge could be from two to five times higher than pre-developed conditions. The volume of runoff can be increased by as much as 50%, and the time of concentration may be decreased by an even greater percentage (see Figure 22.1). Although such effects from a small individual site may seem inconsequential, the collective effects of numerous sites, throughout the watershed, can have substantial impact on the anticipated flood elevation, especially in the lower reaches of a watershed.

### FEMA AND THE NATIONAL FLOOD INSURANCE PROGRAM

#### History of the NFIP

Historically, the federal government responded to development in and around the floodplain by constructing flood-control projects (dams, levees, and seawalls) and providing emergency disaster relief funding to both communities and individuals. However, this approach did little to discourage unwise development, and in some cases, may have encouraged development in floodprone areas. In response to escalating flood-related losses and the burgeoning cost to the taxpayer, in 1968, the U.S. Congress created the National Flood Insurance Program (NFIP). The purpose of the NFIP is to encourage responsible development practices in flood-prone areas and to protect property owners through an insurance mechanism that is funded by those who are most at risk of flooding.

The NFIP, first established in 1968 with the passage of the National Flood Insurance Act, was broadened and strengthened by the Flood Disaster Protection Act of 1973, as well as the National Flood Insurance Reform Act of 1994. The NFIP was administered by the Department of Housing and Urban Development (HUD) until 1979, when it was absorbed by the newly created Federal Emergency Management Agency (FEMA). Currently, FEMA administers the NFIP primarily through the Federal Insurance and Mitigation Administration, which manages both the insurance aspects and the floodplain management aspects of the program.

The NFIP is a community-based program. A community's participation in the NFIP is voluntary (although some states require participation as part of a state-wide floodplain management program). It is the responsibility of each com-

munity to assess their flood risks and determine whether they would benefit from the flood insurance and floodplain management assistance provided through the program. At the time of publication, approximately 19,000 communities participate in the program nationwide. (See Table 23.1 for a list of FEMA regional offices.)

During the early years of the NFIP, to get as many communities in the program as quickly as possible, FEMA established the "Emergency Phase" of the program, which was designed to provide limited amounts of insurance coverage at less than actuarial rates. In general, no detailed floodplain studies have been conducted for communities in the Emer-

**TABLE 23.1 FEMA Regional Offices**

<p>REGION I (Connecticut, Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont) J. W. McCormack Post Office and Courthouse Building Room 442 Boston, Massachusetts 02109 (617) 223-9540</p>	<p>REGION VI (Arkansas, Louisiana, New Mexico, Oklahoma, and Texas) Federal Regional Center 800 North Loop 288 Denton, Texas 76201 (940) 898-5399</p>
<p>REGION II (New Jersey, New York, Puerto Rico, and Virgin Islands) 26 Federal Plaza Room 1311 New York, New York 10278 (212) 680-3600</p>	<p>REGION VII (Iowa, Kansas, Missouri, and Nebraska) 2323 Grand Blvd. Suite 900 Kansas City, Missouri 64108 (816) 374-2161</p>
<p>REGION III (Delaware, District of Columbia, Maryland, Pennsylvania, Virginia, and West Virginia) Liberty Square Building (Second Floor) 105 South Seventh Street Philadelphia, Pennsylvania 19106 (215) 931-5500</p>	<p>REGION VIII (Colorado, Montana, North Dakota, South Dakota, Utah, and Wyoming) Denver Federal Center Building 710 P.O. Box 25267 Denver, Colorado 80225-0267 (303) 235-4800</p>
<p>REGION IV (Alabama, Florida, Georgia, Kentucky, Mississippi, North Carolina, South Carolina, and Tennessee) 3003 Chamblee-Tucker Rd. Atlanta, Georgia 30341 (770) 220-5200</p>	<p>REGION IX (Arizona, California, Hawaii, and Nevada) Presidio of San Francisco Building 105 San Francisco, California 94129 (415) 923-7100</p>
<p>REGION V (Illinois, Indiana, Michigan, Minnesota, Ohio, and Wisconsin) 536 South Clark St. (6th Floor) Chicago, Illinois 60605 (312) 408-5500</p>	<p>REGION X (Alaska, Idaho, Oregon, and Washington) Federal Regional Center 130 228th Street, S.W. Bothell, Washington 98021-9796 (425) 487-4600</p>

gency Program, and the communities are required to adopt only limited floodplain management ordinances to control future use of their floodplains. Only about one percent of the communities participating in the NFIP are in the Emergency Phase.

The remainder of the participating communities are in the “Regular Phase” of the NFIP. These communities are generally provided detailed studies of their floodprone areas and are required to adopt more comprehensive floodplain management ordinances in exchange for higher amounts of flood insurance coverage.

### Types of FEMA Flood Maps

One of the primary tools in administering the NFIP is maps that identify floodprone areas. Because the NFIP is a community-based program, the maps are generally produced for individual communities, although FEMA has recently started updating the maps for select areas to a county-wide format.

FEMA has produced different types of flood hazard maps over the history of the NFIP. The two primary types of maps used in the program are:

- Flood Hazard Boundary Maps (FHBMs)
- Flood Insurance Rate Maps (FIRMs)

FHBMs show the floodplain boundaries based upon approximate data and limited analyses. They were issued during the Emergency Phase of the NFIP and generally have limited information regarding the floodprone areas.

Communities that have been converted from the Emergency Phase to the Regular Phase of the NFIP are provided with FIRMs. FIRMs are generally based upon detailed studies of the floodprone areas and show more detailed information regarding the floodprone areas than the FHBMs.

### Regulations and Terminology

Under the NFIP, the mortgages for any insurable structures that are federally-backed (which almost all are) must have flood insurance if they are located within a *Special Flood Hazard Area* (SFHA). The SFHA is defined as the land area that would be inundated by a flood having a one-percent chance of occurring in any given year. The one-percent annual-chance flood is often referred to as the “base” or “100-year” flood. The elevation of the one-percent annual-chance flood (referred to as the *base flood elevation* or “BFE”) is typically shown on the FIRM for those SFHAs that have been studied in detail.

To identify the SFHA and other areas on the FIRMs, different zone designations are used. Non-coastal areas within the SFHA are designated as Zone A, AE, AO, AH, A1-A30, or A99. Coastal SFHAs, where wave action is a concern, are designated as V, VE, or V1-V30. Areas outside the SFHA include Zones B, C, D, and X. Because each zone designation represents different classes of risk, the insurance premium for a structure is based upon the zone designation. (See Figure 23.1.)

Generally, placing obstructions in the riverine floodplains, such as buildings or other structures, inhibits the flow of floodwaters downstream and results in an increase of the upstream flood elevations. To ensure that development in the floodplain does not result in unacceptable increases in the base flood elevation, as a condition of participating in the NFIP, the community must adopt a regulatory *floodway* into its ordinances. The floodway is composed of the actual stream channel plus the portion of the overbank area that must be kept free from encroachment in order to convey the one-percent annual-chance flood without increasing the BFE by more than a specified amount.

Under the NFIP, the allowable increase, or *surcharge*, within the floodway is one foot; however, many communities and states have adopted more stringent floodway surcharge limits. For example, New Jersey has established a maximum surcharge limit of 0.2 ft, resulting in a wider floodway than that needed to convey the one-percent annual-chance flood with a one-foot surcharge, while other more conservative communities have set a zero surcharge limit, effectively defining the entire floodplain as the floodway.

The area outside the floodway, but inside the SFHA, is commonly referred to as the *floodway fringe*. Although the floodway is typically shown on a community’s FIRM, between the mid-1970s and mid-1980s, separate Flood Boundary and Floodway Maps (FBFMs) were created for some communities that show only the floodways, but not the BFEs or flood risk zones. (See Figure 23.2.)

The full text of the NFIP regulations is contained in Parts 59 through 77 of Title 44 of the Code of Federal Regulations, which can be found on the internet at <http://www.fema.gov/nfip/laws> or ordered from the Government Printing Office by calling (202) 512-1800.

### Options for Map Revision

FEMA’s inventory of flood maps contains over 100,000 map panels that cover most of the floodprone areas in the nation. Maintaining these maps is an enormous challenge. Development and other man-induced and natural changes are constantly altering the floodplains throughout all areas of the country. FEMA conducts flood studies routinely in an effort to keep its maps updated. However, given its limited resources and the pace of the changes, FEMA is unable to conduct detailed studies of every area that is in need of restudy. Consequently, FEMA has established several different methods of amending and revising a FEMA flood map based upon data submitted by communities and property owners.

The options for revising FEMA’s flood maps are described below.

#### FEMA-funded Updates

**FEMA-funded Studies and Restudies.** When flood risks are identified in a community that does not have a flood map, or if flood maps have been prepared for a community

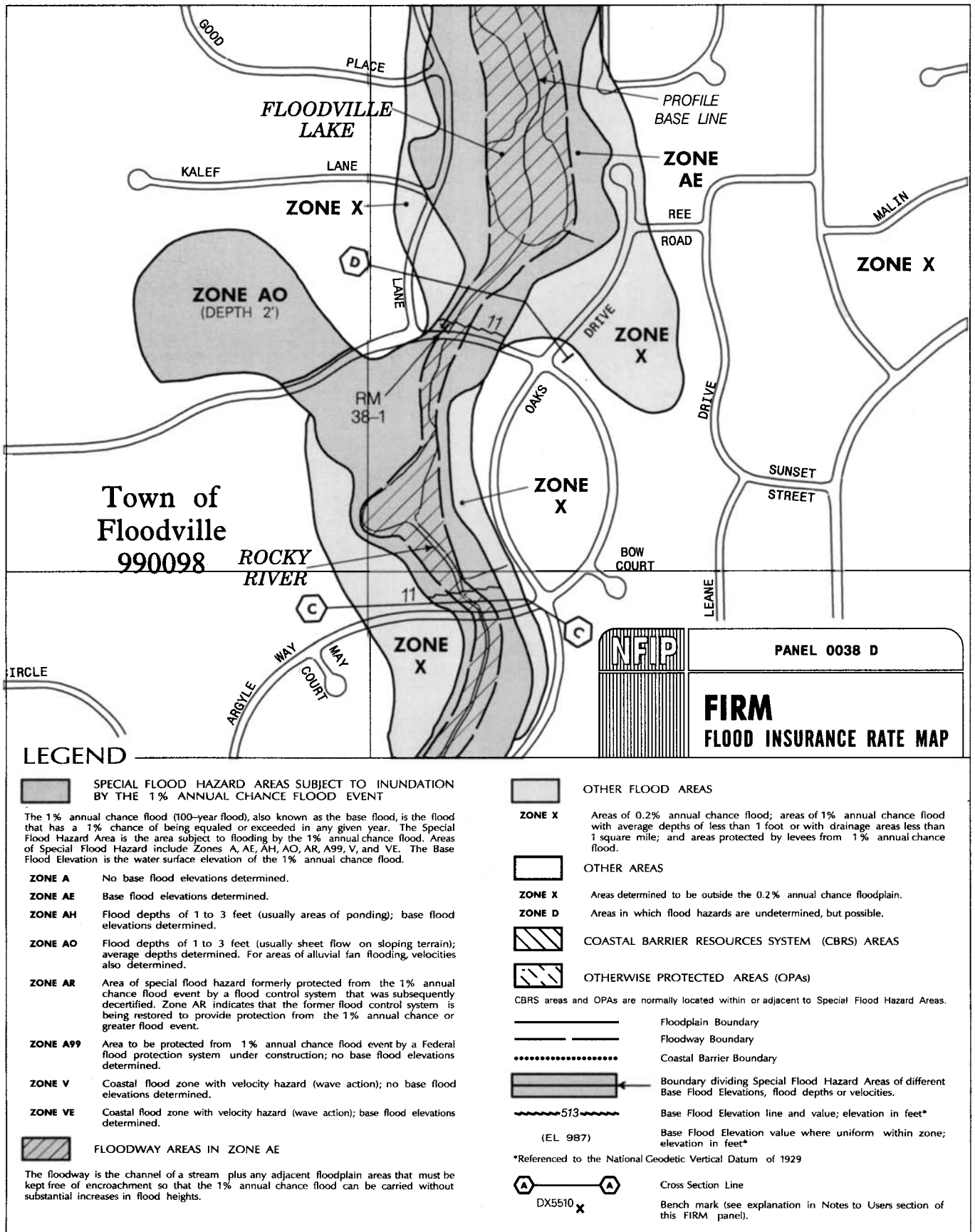
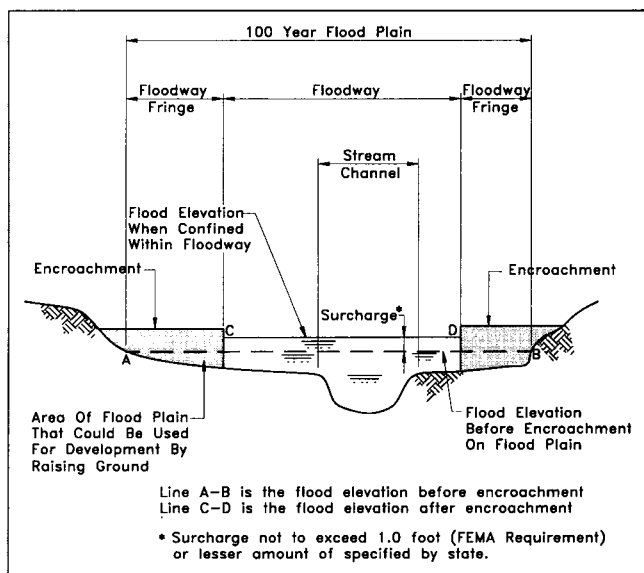


FIGURE 23.1 Sample flood insurance rate map.



**FIGURE 23.2** Showing a floodway and a floodway fringe.

but there are still floodprone areas on the map that do not have designated floodplains, FEMA may conduct a flood study to establish the SFHA boundary. Flood studies require detailed engineering analyses and are generally only conducted in areas where development warrants it. Restudies are detailed engineering analyses conducted in areas that were previously studied but have become outdated or inaccurate.

At the initiation of a study or restudy field surveys, site investigations, and/or aerial photography are conducted. Based upon this data, detailed hydrologic and hydraulic analyses are performed and draft work maps are prepared, which are submitted to FEMA for review and issuance. Traditionally, engineering firms, under contract to FEMA as "Study Contractors," would perform all field work, engineering analyses and work map preparation. Recently, in addition to using their own Study Contractors, FEMA has been partnering with communities and States that have mapping capabilities and resources to contribute to flood studies. (Information on FEMA's requirements for FEMA Study Contractors is provided in "Flood Insurance Study Guidelines and Specifications for Study Contractors (FEMA 37).") During the subsequent 90-day due process period, the "preliminary" maps are issued for public comment. Upon resolution of any appeals or protests received during this time, the maps are issued final. The community is then given a six-month compliance period to update their regulations to reference the new/updated maps.

#### **Limited Map Maintenance Program (LMMP) Revisions.**

LMMPs are engineering analyses that are limited in scope and detail, typically involving no more than one flooding source. In general, they are "quick fixes" to blatant mapping problems. However, unlike studies and restudies in which Study Contractors are selected independently for each project, FEMA contracts, with a single contractor, all LMMPs

within a given FEMA region for a set number of years. LMMPs are subject to the same due process and compliance periods as studies and restudies; however, because of the limited scope of the projects, these processes are typically much more streamlined.

**Existing Data Studies.** These studies rely on existing flood hazard information that was developed for another purpose by a state or another federal agency. These studies which do not require a Study Contractor, are subject to the same due process and compliance periods.

#### **Community/Property Owner-Initiated Amendments and Revisions**

**Letters of Map Amendment (LOMA).** Although NFIP maps are prepared using the best available topographic maps at the time they are prepared, due to limitations of scale and topographic definition, a property may appear to be in a SFHA even though it is actually above the 100-year flood elevation. If no fill<sup>1</sup> has been used to elevate the property, the property owner or lessee may submit a request to be removed from the SFHA via a letter. To obtain a LOMA, the requestor must submit technical data to FEMA certifying that the lowest ground elevation adjacent to the structure (*lowest adjacent grade* or "LAG") is above the BFE. If the building is located in a SFHA for which no BFEs have been determined (e.g., floodplains designated as Zone A), then FEMA will assist the requestor in calculating the BFE for that location. Upon review and acceptance of the data, FEMA will issue a LOMA that officially removes the property from the SFHA. The map is not republished when a LOMA is issued, and the letter does not affect the SFHA delineation or any other features of the NFIP map.

**Letter of Map Revision Based on Fill (LOMR-F).** If a property is located within the boundaries of an SFHA but has been elevated above the BFE by fill, the property owner may submit technical data to FEMA to request a LOMR-F removing the property from the SFHA. The submission must show that both the LAG and the lowest floor (including basement/crawlspace) is above the BFE. Fill used for structural support must meet specific engineering criteria for erosion, stability, and compaction. Please note: LOMR-Fs are not permitted in SFHAs designated as V-Zones.

**Letter of Map Revision (LOMR).** Often, revisions to NFIP maps are warranted due to construction or improvements in the floodplain, annexations that include unmapped floodplains, or other changes that affect BFEs and/or floodplain and floodway delineations. A community may request that the maps be revised to reflect such changes by providing technical data that support the revision. If the changes are not extensive and do not warrant republishing the maps, a LOMR may be issued. Because the community must adopt changes and revisions to the maps in their local

<sup>1</sup> "Fill" is defined to include any earthen material used to raise the ground elevation and includes both structural and non-structural fill.



floodplain ordinances, all such requests must be submitted by, or at the direction of, the community CEO.

**Physical Map Revisions (PMRs).** A PMR is performed if changes to BFEs, floodway or SFHA boundaries, or planimetric features warrant republishing the NFIP map. The decision to issue a LOMR or a PMR is made by FEMA upon receipt of a community's revision request. This decision is based upon a number of factors: the extent and scope of the changes; the cost of making the changes; the number of map panels affected by the changes; the number of previously issued LOMAs and LOMR-Fs; and whether a study/restudy is planned in the near future.

### Acceptable Flood Study Methodologies

To ensure that the studies and restudies meet the minimum standards required by FEMA, certain technical methods have been reviewed and accepted by FEMA. These methods include hydrologic and hydraulic models and computation methods used in the detailed engineering analyses conducted as part of most studies/restudies. See Chapter 41 for lists of acceptable methods at the time this book was published. Because FEMA occasionally reviews and approves new models and methods and updates to existing models and methods, FEMA should be contacted about the use of any method not listed in Chapter 41 prior to its use.

## PREPARING A FLOODPLAIN STUDY

### Introduction

A flood plain study graphically depicts an engineering estimate of the water surface elevation expected along a length of a stream for some specified design storm. Typically, a floodplain study is conducted as a design tool or as a regulatory requirement. As a design tool, they are often used to determine the limits of inundation, for some specified recurrence interval, to assure that new developments will lie beyond the floodplain. Similarly, they may be used to evaluate the changes in flood elevations due to changes or additions to structures within the floodplain. A floodplain study may also be conducted to evaluate the effectiveness of channel improvements or modifications.

Even if a floodplain study does not seem necessary for design, it is often a regulatory requirement. It is common for local or state governments to require that a floodplain study be performed and submitted in support of a land development plan. Regulations often contain a provision that a floodplain study is required for a reach of stream if the drainage area is greater than some minimum value (100 acres may be typical).

Structures or fill placed within the floodplain may restrict the flow, raising the flood elevation upstream of the encroachment. When new structures are proposed within the floodway, it is common to require a floodplain study to ensure that the new construction does not increase the water surface elevation more than the allowable surcharge above the base flood elevation.

### Hydrologic Data and Methodologies

**Information Search.** Before beginning a floodplain study, an information search should be conducted to determine if previous studies on the subject stream exist or if other studies have been performed that may provide some of the data needed for a study on the subject stream. Some federal agencies perform floodplain studies as an aid to identifying flood-prone areas. These agencies include the U.S. Geological Survey, the U.S. Army Corps of Engineers, and FEMA. In addition, some local or state organizations may have drainage plans or floodplain studies already established. If a development project is near a previously or concurrently developed site that is also affected by the subject stream, a floodplain study may have been developed in conjunction with the nearby development. These studies should be identified and examined to determine if they may be used outright or if some pieces of information may be used from the studies.

**Methodologies.** The first step in performing a floodplain study is the hydrologic analysis. A hydrologic analysis is performed in order to estimate the flood discharge at a location along the flooding source. As with other aspects of the floodplain study process, the first step in performing a hydrologic analysis should be to determine if there is a stream gauge on the stream in question. Typically, in the land development process, the drainage areas of the flooding sources in question are smaller (less than one square mile) and likely not gauged.

In trying to determine the flood discharge for an ungauged stream, several techniques can be employed. Often, more than one technique will be utilized in order to support your results. These techniques can be broken down into two categories, namely Regression Equations and Rainfall-Runoff Modeling. Federal, state, and local ordinances should be consulted to determine which technique is approved for use in your jurisdiction.

*Regression Equations:* The land development engineer should once again determine if any federal, state, or local agency has established flood discharges previously for the ungauged stream under consideration or should check the FEMA effective Flood Insurance Study. In the absence of such an analysis, the land development engineer shall use the most recent regional flood flow frequency report, published by the USGS. These reports are generally available on a statewide basis. Caution should be exercised when applying these equations, for they have been developed for conditions and locations for which they are recommended.

*Rainfall-Runoff Modeling:* Where USGS regional flood flow frequency reports have not been developed or are not applicable due to either rapid watershed development, storage, or stream regulation, the land development engineer can develop a rainfall-runoff model such as HEC-HMS or TR-20. These programs as well as the procedures for developing these models have been described in the Stormwater Management chapter (Chapter 22).

## Hydraulic Data and Methodologies

**Data Requirements.** The data required for a floodplain study will vary to some degree depending on the methods used, the level of detail required, and the desired results. However, the basic data to be gathered will normally include topographic mapping, the limits of stream to be studied, flow discharges (hydrology), cross sections, and Manning's "n" values.

First, topographic mapping of the area to be studied should be obtained. Sources of topographic mapping include local and state agencies, USGS Quadrangle maps, and site topographic maps for the land development project. This mapping will be indispensable for properly setting up the floodplain study. The mapping should cover the area to be studied, with plenty of additional coverage beyond the actual floodplain limits. There are several uses of the mapping, not the least of which is as a base map for plotting floodplain boundaries. Other uses for the topographic mapping will be discussed as appropriate.

Next, the limits of the stream to be studied must be determined. The limits will be determined based on the desired results of the study. If the reason for performing a floodplain study is due to a regulatory requirement related to the land development project, then the entire length of stream through the project limits will likely be studied. If the reason for performing the study is to determine the floodplain boundary in a certain location, or to determine the necessity for channel improvements, etc., then the limits of study must be determined such that the portion of the stream impacting the development will be studied. Once the necessary limits have been determined, some additional distance upstream and downstream should also be included to insure that the appropriate reach is covered and to account for downstream and upstream impacts. The topographic mapping should be used to lay out the limits and to investigate the stream upstream and downstream of the section to be modeled. The limits should be set such that no hydraulic control sections in the floodplain occur beyond the limits, such as a severe constriction, a bridge, or a weir, that will have a significant impact on the reach being modeled.

The appropriate discharges must be determined for the floodplain analysis. An analysis of the hydrology may be required, as discussed earlier in this chapter. Also, appropriate recurrence intervals should be chosen to provide the level of risk or protection desired. The 100-year floodplain is the most common for analysis, because of regulatory requirements from local jurisdictions and from FEMA. In addition, the incremental changes to the flow discharges may need to be made along the stream reach where tributaries or storm sewers discharge or where the drainage area increases significantly.

A floodplain study usually requires analyzing cross sections for flood depth, width, velocity, etc., and interpolating the results between cross sections. The selection of cross-

section locations is therefore a very important step in setting up a floodplain model. The topographic map should be used in conjunction with field information to determine the best locations for cross sections. Cross sections should be located at points along the stream where the shape of the section changes, where the slope of the stream changes, at bends in the stream, and at locations where the floodplain conditions otherwise change from the conditions upstream or downstream. Consideration should be taken of the entire floodplain, not just the channel portion, when locating cross sections. In addition, cross sections should be located upstream and downstream of bridges and culverts, and downstream of tributary inflows where the stream flow discharge should be adjusted. Cross sections should also be placed at locations where conditions may change due to the land development project, or at locations where floodplain elevations are desired for a specific purpose. After cross sections have been located as described above, additional cross sections may be placed to provide additional, more detailed information for interpolating between cross sections. Also, local regulations may dictate a maximum allowable distance between cross sections for a floodplain study.

At each cross-section location, the cross-section geometry is required. This may be determined from a topographic map, if the map provides an acceptable level of detail for the study. Often, field surveys will be required to obtain the cross-section geometry necessary for a floodplain study. Using a greater number of cross sections will increase the level of detail of the study, but will also increase costs for data collection and modeling. Also, a greater level of detail in cross sections may be unwarranted if other aspects of the study have a greater level of uncertainty. For example, if the flow discharges are highly questionable, then detailed modeling will still provide highly questionable results. Therefore, considerable judgment must be used in selecting cross-section locations.

Roughness coefficients must be determined for each reach of stream. Most floodplain models use Manning's "n" values to estimate the roughness of the stream channel and overbanks. Selection of  $n$  values is very important because of the influence of roughness on water surface elevations. Furthermore,  $n$  value selection is the one parameter that cannot simply be measured in the field but, instead, must be determined largely from engineering judgment. Proper selection of  $n$  values will provide a greater level of confidence in the floodplain model. Manning's  $n$  values for several types of channels are presented in Table 23.2.

**Methodology.** Flood elevations for riverine areas are normally determined by step-backwater computation models, like HEC-RAS, WSPRO, or WSP-2. The most common model is the USACE HEC-RAS (which supercedes HEC-2) program, although the land development engineer should consult the local regulations to determine which of the above noted programs is acceptable for use. The models accepted by FEMA are listed in Chapter 41.

**TABLE 23.2 Values of the Roughness Coefficient  $n$**

TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM	TYPE OF CHANNEL AND DESCRIPTION	MINIMUM	NORMAL	MAXIMUM
Excavated or Dredged							
<i>a.</i> Earth, straight and uniform							
1. Clean, recently completed	0.016	0.018	0.020	8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
2. Clean, after weathering	0.018	0.022	0.025	<i>b.</i> Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
3. Gravel, uniform section, clean	0.022	0.025	0.030	1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
4. With short grass, few weeds	0.022	0.027	0.033	2. Bottom: cobbles with large boulders	0.040	0.050	0.070
<i>b.</i> Earth, winding and sluggish				2. Flood plains			
1. No vegetation	0.023	0.025	0.030	<i>a.</i> Pasture, no brush			
2. Grass, some weeds	0.025	0.030	0.033	1. Short grass	0.025	0.030	0.035
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040	2. High grass	0.030	0.035	0.050
4. Earth bottom and rubble sides	0.028	0.030	0.035	<i>b.</i> Cultivated areas			
5. Stony bottom and weedy banks	0.025	0.035	0.040	1. No crop	0.020	0.030	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050	2. Mature row crops	0.025	0.035	0.045
<i>c.</i> Dragline-excavated or dredged				3. Mature field crops	0.030	0.040	0.050
1. No vegetation	0.025	0.028	0.033	<i>c.</i> Brush			
2. Light brush on banks	0.035	0.050	0.060	1. Scattered brush, heavy weeds	0.035	0.050	0.070
<i>d.</i> Rock cuts				2. Light brush and trees, in winter	0.035	0.050	0.060
1. Smooth and uniform	0.025	0.035	0.040	3. Light brush and trees, in summer	0.040	0.060	0.080
2. Jagged and irregular	0.035	0.040	0.050	4. Medium to dense brush, in winter	0.045	0.070	0.110
<i>e.</i> Channels not maintained, weeds and brush uncut				5. Medium to dense brush, in summer	0.070	0.100	0.160
1. Dense weeds, high as flow depth	0.050	0.080	0.120	<i>d.</i> Trees			
2. Clean bottom, brush on sides	0.040	0.050	0.080	1. Dense willows, summer, straight	0.110	0.150	0.200
3. Same, highest stage of flow	0.045	0.070	0.110	2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
4. Dense brush, high stage	0.060	0.100	0.140	3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
Natural Streams				4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
1. Minor streams (top width at flood stage <100 ft)				5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
<i>a.</i> Streams on plain				3. Major streams (top width at flood stage >100 ft). The $n$ value is less than that for minor streams of similar description, because banks offer less effective resistance.			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033	<i>a.</i> Regular section with no boulders or brush	0.025	—	0.060
2. Same as above, but more stones and weeds	0.030	0.035	0.040	<i>b.</i> Irregular and rough section	0.035	—	0.100
3. Clean, winding, some pools and shoals	0.033	0.040	0.045				
4. Same as above, but some weeds and stones	0.035	0.045	0.050				
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055				
6. Same as 4, but more stones	0.045	0.050	0.060				
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080				

Source: Courtesy of Chow, V.T., *Open-Channel Hydraulics*, New York: McGraw-Hill, 1959. Reproduced with permission of McGraw-Hill, Inc.



### Format and Presentation

The end product of a floodplain study should consist of three items: (1) a plan view of the stream being studied, usually on a topographic base, with the floodplain boundaries delineated along the stream; (2) a profile (longitudinal cross section) of the stream, with the stream invert elevations and flood elevations delineated; (3) technical documentation regarding the determination of the floodplain elevations and limits. This documentation may include calculations and/or computer modeling results, a written description of the methods used, sources of data, special or unusual items that affected the modeling, assumptions made, and limitations of the model.

Figure 23.3 shows a typical plan and profile sheet for a flood study. If a floodplain study is prepared to determine and document the changes to flood elevations or floodplain boundaries due to a development project, both pre-developed and post-developed floodplains should be presented. If a floodway is required, the floodway boundaries should also be presented. A typical plan and profile sheet of a floodplain study should include the following information: cross-section location, orientation, and numbering; floodplain and floodway boundaries; water surface profiles; water surface elevations; locations of bridges and culverts; locations of incoming tributaries; floodplain tie-in to floodplains on tributaries; and location and elevation of any critical structures within or near the floodplain.

### REFERENCES<sup>2</sup>

Bedient, Philip B., and Wayne C. Huber, *Hydrology and Floodplain Analysis*, Addison-Wesley, Reading, Mass., 1988.

<sup>2</sup> Some of the references listed below are government documents. In addition to the individual agencies, some of the government publications may be available through the National Technical Information Service (NTIS) or the Government Printing Office (GPO) for a minimal charge. For availability and price, contact: National Technical Information Service (703) 487-4650 and U.S. Government Printing Office, (202) 783-3238.

Brater, E.F., and H.W. King, *Handbook of Hydraulics for the Solution of Hydraulic Engineering Problems*, 6th ed., McGraw-Hill, New York, 1976.

Chow, V.T., *Open-Channel Hydraulics*, McGraw-Hill, New York, 1959.

Chow, Ven Te, David R. Maidment, and Larry W. Mays, *Applied Hydrology*, McGraw-Hill, New York, 1988.

Code of Federal Regulations, Title 44, Chapter 1, Parts 59–72 (National Flood Insurance Program).

Federal Highway Administration, *Debris-Control Structures*, Hydraulic Engineering Circular No. 9. Washington, D.C., 1971.

Federal Emergency Management Agency, *Flood Insurance Study Guidelines and Specifications for Study Contractors*, Washington, D.C.

Federal Emergency Management Agency, *Appeals, Revisions, and Amendments to Flood Insurance Maps, A Guide for Community Officials*, Washington, D.C., 1985.

French, Richard H., *Open-Channel Hydraulics*, McGraw-Hill, New York, 1985.

Linsley, Ray K., and Joseph B. Franzini, *Water Resources Engineering*, 3rd ed., McGraw-Hill, New York, 1979.

National Weather Service, *Five to 60-Minute Precipitation Frequency for the Eastern and Central United States*, (NWS Hydro-35), Washington, D.C., 1977.

U.S. Army Corps of Engineers, *HEC-2, Water Surface Profiles, User's Manual*, Hydrologic Engineering Center, Davis, CA, 1991.

U.S. Department of the Interior, Geological Survey, *Guidelines for Determining Flood Flow Frequency*, Bulletin #17B of the Hydrology Subcommittee, Interagency Advisory Committee on Water Data, 1982.

U.S. Geological Survey, *Roughness Characteristics of Natural Channels*, Water Supply Paper 1849, U.S. Gov't. Printing Office, Washington, D.C., 1967.

U.S. Geological Survey, *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, Water Supply Paper 2339. U.S. Government Printing Office Washington, D.C., 1989.

## CHAPTER 24

# GRADING AND EARTHWORK

John Matusik, P.E. / Revised/Updated: Daniel Deible, R.L.A.

### INTRODUCTION

Grading is configuring the surface of the land by removing or adding earthen material to shape the land to best suit the project. It is accomplished with both large machines, such as bulldozers, pans, and dump trucks, and men wielding rakes and shovels. Grading is a major component of the function and success of a land development project (see Figure 24.1).

In this chapter, a general overview of the grading process will be followed by an explanation of how grading is represented and worked with on plans. A more specific breakdown of grading strategies and requirements follows, providing all the tools needed by the designer to produce an effective grading plan.

A good design integrates the natural landforms of the site with the proposed program to create an aesthetically pleasing, yet functional and cost-effective site plan. Because a grading scheme must consider function and utility, as well as aesthetics, it is both a science and an art. The grading of a site serves three basic purposes:

1. *Grading re-forms the land surface to make it compatible with the intended land use.* Subsequently, the relative elevations and gradients of streets, buildings, parking areas, and pedestrian/vehicle accesses must be mutually compatible if they are to function as a system. Similarly, they must be compatible with the surrounding existing terrain. Incompatibility with the existing terrain, which leads to excessive earthwork, the use of retaining walls, and drainage problems, increases construction costs.
2. *Grading establishes and controls the new drainage patterns.* In order to be cost effective, the grading de-

sign should allow for the efficient collection, conveyance, and detention of stormwater runoff. Proper grading prevents wet basements, damp crawl spaces, foundation damage, eroding hillsides, and muddy stream waters.

3. *Grading helps define the character and aesthetics of the site.* Site design is the foundation upon which many other elements of development depend. Proper grading should be cost-effective to the developer, appealing to the user, and responsive to the opportunities and restraints offered by the site. In this way, it enhances property values and contributes to the success of a land development project.

Often, the word “grade” refers to the slope, as in “the mountain road has a steep grade.” However, the word is also used as a reference to elevation, as in “what is the grade at the top of the driveway?” Both uses are correct, in that grading changes the ground elevation and, therefore, the inclination of the ground surface.

### CONTOUR GRADING

#### Description of Contour Lines

Contour lines are a method for depicting three dimensions on two-dimensional media, while maintaining a uniform scale in all directions. A contour line is an imaginary line connecting points of equal elevations and is formed by the intersection of a horizontal plane with the ground surface. The spacing and shape of contour lines indicate the shape and the inter-relationships of these land forms. A natural example of a contour line is the shoreline of a still body of water.



**FIGURE 24.1** Grading the site. (Photo Courtesy of Christer Carshult)

The vertical distance between successive contour lines is the contour interval. Most topographic maps, especially those associated with a land development project, have a constant contour interval. For instance, every contour may indicate a 2-ft change in elevation. Typically, the contour line at every fifth or tenth contour interval is shown as a heavier or darker line to make the map easier to read. In the rare instances where extremely flat areas and extremely steep slopes are shown on the same map, the addition or deletion of contour lines may be warranted. However, this depends on the scale of the drawing and the desired level of detail. The addition or deletion of intermediate contour lines results in an inconsistent contour interval for the map.

Topographic maps are an integral part of the design process of land development projects. The contour interval and level of detail depends on the purpose of the task and the scale of the drawing.

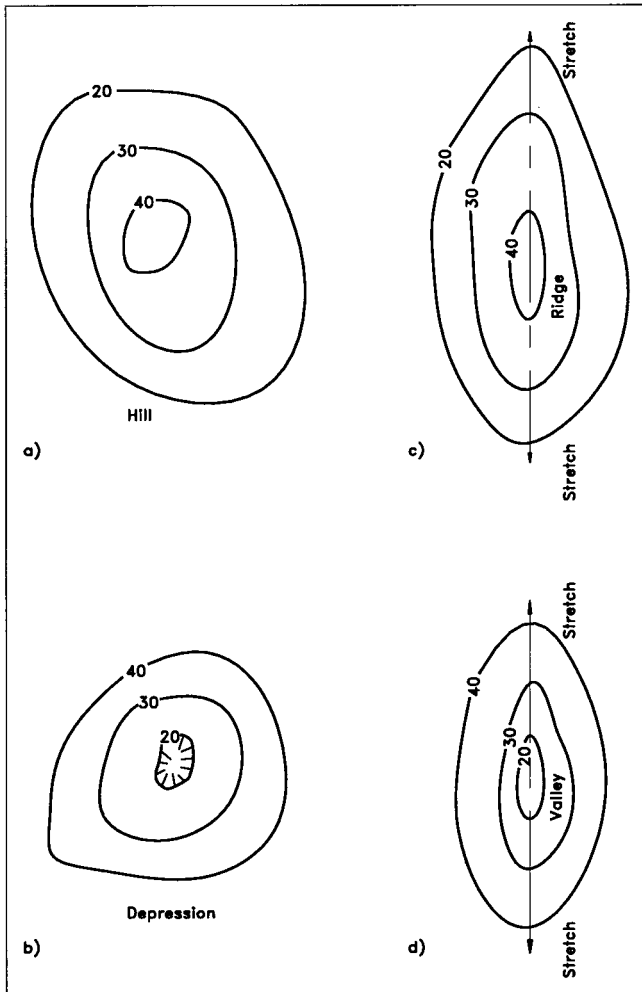
Large scale topographic maps, with 5 to 10-ft contour intervals, are usually available at a reasonable cost from government agencies and are used for feasibility studies of the project, where detailed information is not as critical. Because these maps are not constantly updated, their accuracy may be suspect, especially in areas of rapid development, so always qualify the information presented when using them.

Smaller scale maps, with 2-ft contour intervals, are used for final design and detailed studies. These maps are usually produced from recently collected data and provide a more accurate basis for design. State and local agencies, through ordinance and design standards, often require specific contour intervals for drawings submitted for review.

### Characteristics of Contour Lines

A key to conceptualizing and executing a grading plan is the ability to visualize the two-dimensional information depicted on the plan in three dimensions. In order to engineer a land development plan, the designer needs to have an understanding of contour lines and be able to recognize the land features associated with them. The following list briefly describes the fundamental characteristics of contour lines.

- *All contour lines eventually close on themselves if traced in their entirety.* Any apparent break in a contour line is due to the limitations of the map. Contour lines that extend beyond the limits of the subject area terminate at the map edge.
- *A series of contours that close on themselves within the mapped area indicate either a localized hill or localized depression.* Figure 24.2-a shows a hill and Figure 24.2-b shows a depression. In Figure 24.2-a, the elevations shown on the contour lines increase up to the summit. Conversely, in Figure 24.2-b, the elevations decrease towards the bottom of the depression.
- *Valleys and ridges are indicated by contour lines configured in V-shapes.* Imagine the contours of the hill in Figure 24.2-a being stretched in one direction. The result is an elongated hill, depicted by contours that are shaped like Vs at either end, as indicated in Figure 24.2-c. The tips of this V-shaped hill, when connected, depict a ridge. Stretching the contours of the depression of Figure 24.2-b creates the valley configuration of Figure 24.2-d. Both stretched figures are identical in



**FIGURE 24.2** Contour pattern: (a) for hills, (c) for ridges, (b) for depressions and (d) for valleys.

appearance except for the direction of increasing elevations. To distinguish the ridge from a valley, notice the direction of the apex of the V. On ridges, the V points down ridge (i.e., downhill), while the V points upstream (i.e., uphill) in valleys. Additionally, stream valley contours typically have a sharper V-shape, whereas ridges may be in a rounded U-shape.

- *Spacing of contours indicates general steepness of the ground.* While closely spaced contours indicate steep slopes, as the ground slope becomes flatter, the distance between the contours increases. Most natural hills and depressions are convex or concave in shape. A slope is convex-shaped if there is an increase in spacing between contour lines near the crest of the hill. Conversely, a slope is concave-shaped if there is an increase in spacing between the contour lines near the bottom of the slope.

- *Technically, contour lines never cross.* If a contour line represents a single elevation, then intersecting contour lines indicate two distinct elevations at the same point, a physical impossibility. However, in the case of

an overhanging cliff, where contour lines may appear to cross, this does not indicate dual elevations, since the two contours are actually in different horizontal planes.

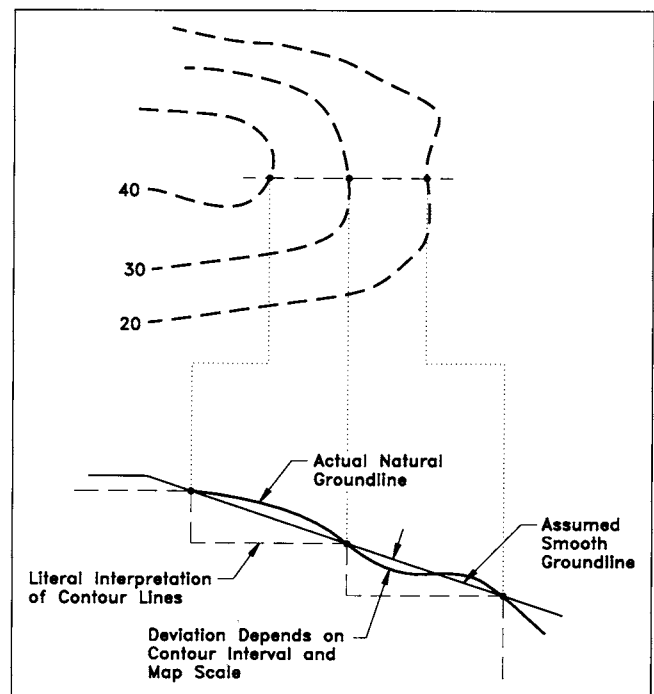
A contour line cannot split, nor can several lines join to form one line. This implies a knife-like edge, which is an unnatural occurrence.

In general, irregularly shaped contour lines designate rough, rugged landforms, while parallel, equally spaced contour lines indicate a smooth, uniform slope—often a machine-graded slope. Note: the steepest slope and also the path of flowing water is perpendicular to the contour lines, an important consideration when establishing drainage divides.

On a relatively large scale, the natural ground line is considered smooth and continuous. Relatively few ground features show sharp jagged or abrupt changes in ground relief. This smoothness is carried over to the concept of contour lines. Contour lines indicate distinct elevations, with the actual ground line between contour lines having local areas of irregular depressions and mounds, which may deviate (up to  $1 \pm$  ft) from the assumed smoothed ground line, as schematically shown in Figure 24.3. When determining an elevation between contour lines, the ground is assumed to be straight. Machine-graded slopes tend to be more uniform than natural ground and consequently have fewer irregularities.

### Ground Slope

Ground slope is the rate of change in elevation with respect to the horizontal distance, commonly expressed as either a



**FIGURE 24.3** Irregularities in natural ground.



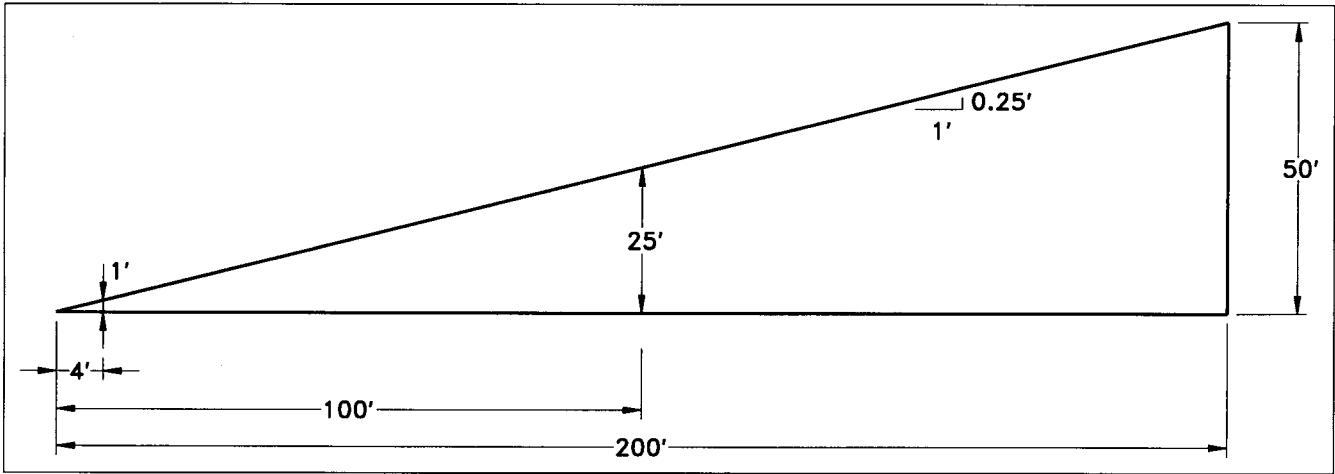


FIGURE 24.4 Various ways to denote slope gradients: (a) 4H:1V, (b) 25%.

percentage or a ratio. The percent slope describes the uniform change in elevation for a 100-foot horizontal distance. For example, if elevation changes 25 ft over a distance of 100 ft, the slope is expressed as 25%. Mathematically, this slope is defined as the ratio of  $\Delta v/\Delta x$ , where  $\Delta v$  is the change in vertical direction and  $\Delta x$  is the change in horizontal position (i.e., a distance,  $h$ ). A simple way to remember this basic tenant is in terms of “rise/run.” As a ratio, this slope would be expressed as 4 ft horizontal to 1 ft vertical, or simply “4h to 1v.” For example, the rate of change in elevation between two points that are 200 ft apart, with corresponding elevations of 100 ft and 150 ft (Figure 24.4), is

$$\begin{aligned} \text{rate of change in elevation} &= \frac{\text{change in elevation}}{\text{horizontal distance}} && (24.1) \\ &= \frac{\Delta v}{h} = \frac{150 - 100}{200} \\ &= 0.25 \text{ feet/foot} \end{aligned}$$

Hence, for every 1 ft of horizontal distance, the ground changes 0.25 ft in elevation. As a percentage, this equates to a slope of 25%; as a ratio this is equivalent to a 4h:1v slope (or 1v:4h). Figure 24.4 illustrates these various ways for designating slope gradients.

On most civil site drawings, the ratio is shown as horizontal to vertical,  $h:v$ . However, the order is sometimes reversed, (i.e.,  $v:h$ ) in other professional disciplines (e.g., architecture). Because of the significant difference between the two, always verify the order. To that end, it is good practice to include the  $h$  and  $v$  designation (e.g., 4h:1v) or indicate in a note on the plans the order of the  $h$  and  $v$  in the ratio.

The specific elevation of any point that lies between two points of known elevation is found by interpolation, assuming the ground slope between the two known points is linear. Hence, the newly determined elevation is based on the average ground slope between the two end points.

Given two end points (A and B) at positions  $x_A$  and  $x_B$  apart, with elevations  $El_A$  and  $El_B$ , the average ground slope,  $S_{avg}$ , between the two points is

$$S_{avg} = \frac{El_B - El_A}{x_B - x_A} = \frac{\Delta v}{\Delta x} \quad (24.2)$$

The distance  $\Delta x$  ( $h$ ) may be the scaled distance from the drawings, or it may be obtained through coordinate geometry if coordinates of points A and B are known.

$$El_P = El_A + (S_{avg} \times h') \quad (24.3)$$

In Figure 24.5, the elevation at any point P between A and

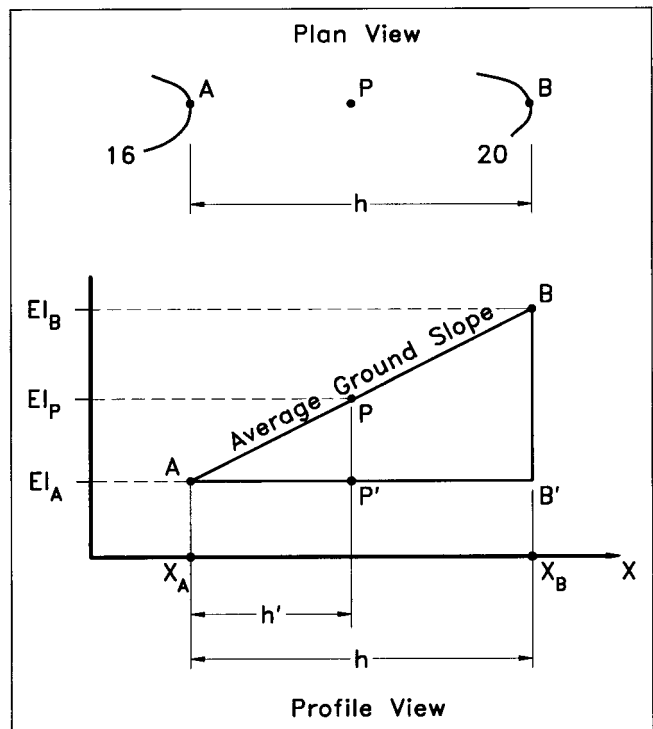


FIGURE 24.5 Interpolation using similar triangles.

B and relative to point A is where  $h'$  is the horizontal distance from A to point P. The relative elevation of any point P along  $h$  is accounted for by the algebraic sign of  $S_{avg}$ , as determined by Equation 24.2. In this instance, the slope increases in going from A to B.

Another way to compute the elevation at any point P is to recognize that triangles APP' and ABB' are similar and the ratios of their corresponding sides is a constant, as shown in Figure 24.5. Hence from the following geometric ratios

$$\frac{h}{h'} = \frac{(BB')}{(PP')} \quad (24.4)$$

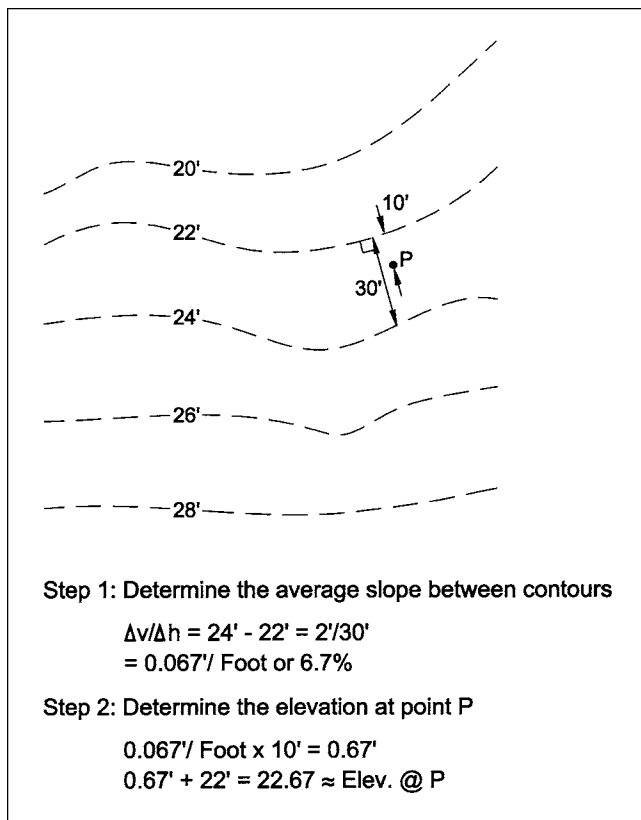
the elevation is found by adding (or subtracting depending on the relative slope) the distance PP' from the elevation.

Typically, the  $h$  measured is the shortest distance between the two points and, as a result, produces the steepest gradient between the two points. However, the horizontal distance measured between the two points may be more circuitous if the situation warrants. For instance, the average slope may be desired along a curb return or along a winding stream channel.

Figure 24.6 provides an example of interpolating between contours to determine the elevation of a point.

**Contour Line Patterns for Constructed Surfaces**

The concepts of contour lines can be applied to constructed surfaces just as easily as they are used on natural features.



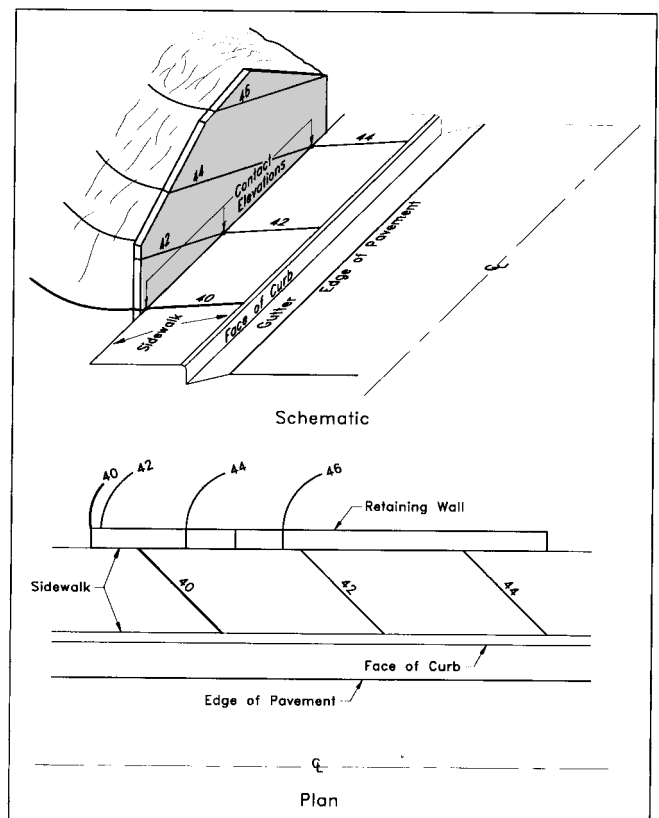
**FIGURE 24.6** Working with contours.

For instance, swales and mounds have distinct patterns similar to valleys and ridges. Reading the contours identifies important information such as drainage patterns and road gradients. The following discussion focuses on the contour line patterns of the most common types of constructed features.

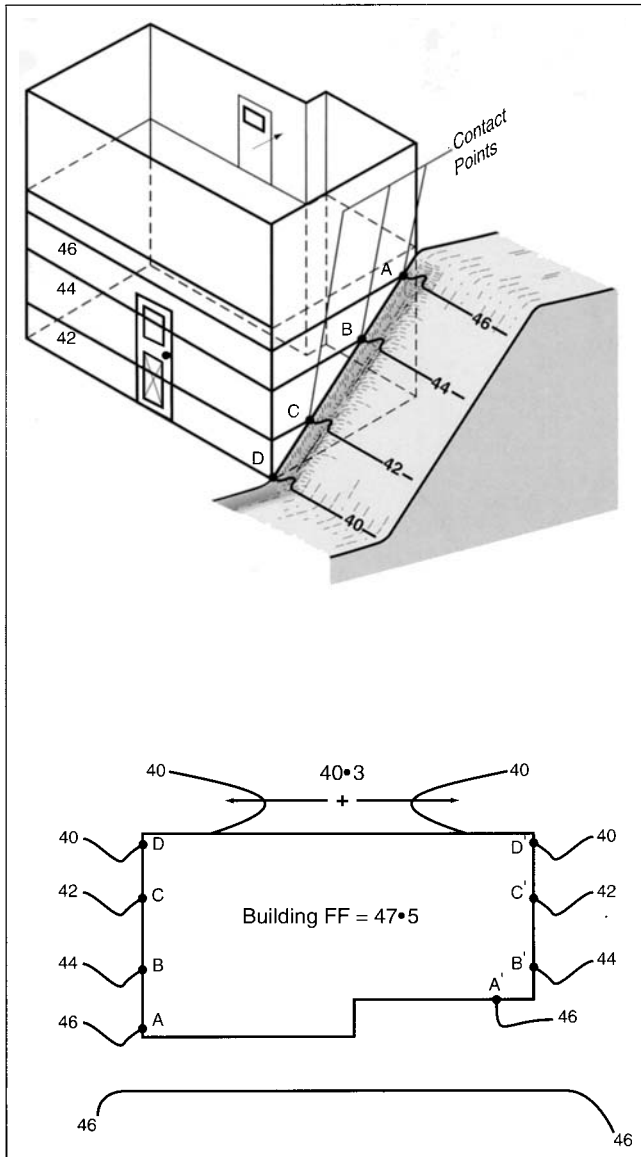
**Retaining Walls.** Although it is physically impossible for several contours to join and form a single contour line, retaining and exterior building walls can appear in plan view to do just this. A machine-graded uniform slope produces equally spaced, parallel contour lines. For the same contour interval, the spacing of the contour lines decreases (in the plan view) as the inclination of the slope increases. For a vertical slope (wall), the space between the contour lines disappears. Consider the face of a wall as a series of contour lines stacked one on top of another, as shown in Figure 24.7.

In tracing the contour line around a wall, the contour line intersects the wall at the contact elevation (i.e., where the ground meets the wall) and then continues along the face of the wall. The contour line leaves the wall where the wall intersects the ground at that elevation.

**Exterior Walls of Buildings.** These serve as earth retaining walls, as well as structural support. Similar to the conventional retaining wall, the contour line enters the wall where the ground surface intersects the wall at the prescribed elevation, shown as points A, B, and C in Figure 24.8. The



**FIGURE 24.7** Schematic and plan view of retaining wall with contour lines superimposed.



**FIGURE 24.8** Schematic and plan view of exterior building wall with contour lines superimposed.

contour line then follows the exposed face of the building wall until it reaches the point where the ground surface is the same elevation, shown as points A', B' and C' in the plan view of Figure 24.8. Since a contour line is continuous, if it "enters" the wall, it must "exit" the wall.

**Conveyance Channels.** Four frequently used conveyance channel sections are trapezoidal, V-ditch, rectangular, and semicircular. In the plan view of Figure 24.9, each of the channels is two ft deep. The depth of the channel is evident by comparing the elevation at the top of bank with the elevation at the bottom of the channel. On each type of channel section, point A, the top of bank, has an elevation of 06 ft. A line drawn perpendicular to the flow line, through the top of bank, intersects the flow line at elevation 04 ft, representing the elevation of the channel invert.

Hence, the depth of two ft. The V-ditch section shows the V-apex pointing upstream—a quick indication of the direction of flow. Direction of flow in the other channel sections is evident from inspection of the contour elevations.

Another indication of the depth of the channel is how far upstream the contour line runs. The spacing of the contour lines along the sides of the channel is an indication of the steepness of the bank. Figure 24.10 shows a 2-ft-deep and 4-ft-deep V-ditch with the same longitudinal slopes and the same top-widths,  $w$ . Note how the contour lines extend farther upstream for the 4-ft-deep channel as compared to the 2-ft-deep channel. Because the top widths are the same, the 4-ft-deep channel has steeper side slopes, evidenced by the contour line spacing on the sides of the channel.

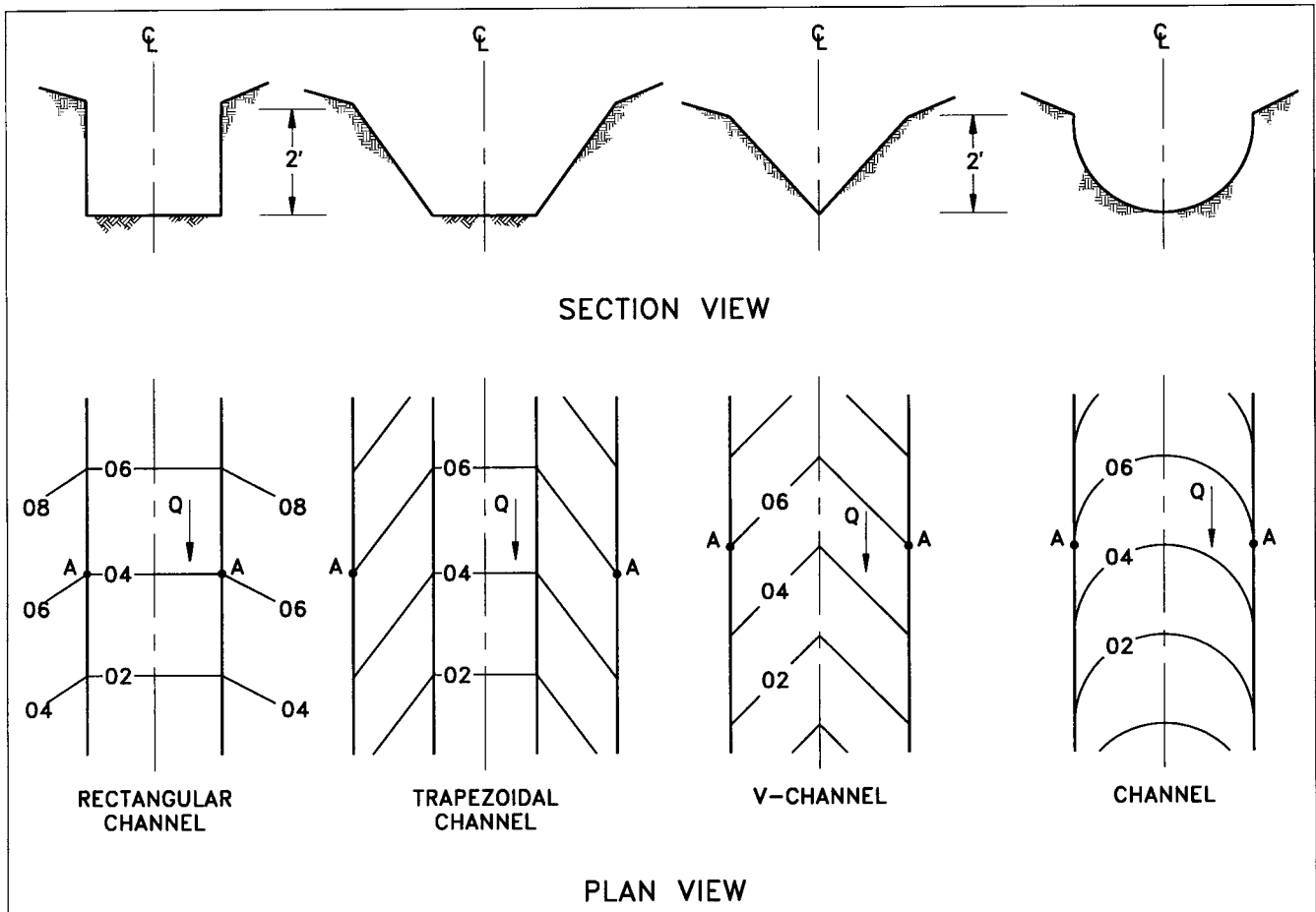
**Streets.** Two types of streets commonly used in development projects are the crown street with curb and gutter and the crown street with shoulder and ditch. Figure 24.11 shows the contour line pattern for a curb and gutter street section with longitudinal slope  $S$ . For a normal crown section street, the elevation decreases on a line perpendicular to the centerline due to the cross slope of the pavement. From the prescribed elevation on the centerline (point A), the contour line follows a straight line path that leads uphill until it meets a point at the edge of the gutter (point B) equal in elevation to the centerline elevation.

The break in the contour at the edge of the gutter (point B) results if the cross slope in the gutter pan differs from the cross slope of the street. From the edge of the gutter, the contour line continues uphill to the point on the flow line with the same elevation (point C). The contour line then follows the face of the curb downhill to the point on the top of the curb with the same elevation (point D). In the plan view, the contour line from C to D does not appear since the top of curb, the bottom of curb, and contour are all superimposed. The contour line intersects the outside edge of the sidewalk at the prescribed elevation. Typically, sidewalks are inclined towards the street. This is apparent from the contour line's downhill direction (points D to E) across the sidewalk. A similar trace of the contour line is shown in the shoulder/ditch type of street of Figure 24.12.

Knowing that water flows perpendicular to the contour line, it is easy to determine the direction of the longitudinal gradient for crown section streets from a quick glance of the contour pattern. Typically, surface drainage on the pavement flows toward the curb and gutter (or ditch); therefore, water flows in the direction perpendicular to the contour lines towards the curb or ditch.

### Plotting Street Contours

Knowing the concepts and mechanics for plotting contour lines is essential in land development design. Manual plotting of contour lines is rarely done for large street projects, given the technology of computers, software and digitizing. However, there is occasion to manually plot contours on short street sections and throughout other areas of the site.



**FIGURE 24.9** Contour line pattern for conveyance channels.

Typically, on a subdivision street with normal crown section, the crown is coincident with the centerline of the street. For this discussion, the focus will be on a straight segment of a crown section street with a uniform longitudinal slope.

The longitudinal reference line plotted in the profile view, identifying the design elevations of a street, is the profile grade line (PGL). The PGL may be the centerline, a line following the top of curb or median, or any other conveniently established line. The PGL is the reference line that has a known horizontal and vertical relationship to any other part of the street. For example, the top of curb elevation is known, given the pavement width and cross slope from the PGL centerline.

**EXAMPLE 1:** The PGL line is the centerline of a straight segment of a crown section street that has curb and gutter and sidewalk on the left side, and shoulder and ditch on the right side.<sup>1</sup> Figure 24.13 shows the typical section. From the profile view (not

given), the longitudinal slope of the street is  $-4.5\%$  in the direction of increasing stationing and the elevation of station  $10 + 50 = 125.50$  ft. Plot the contour lines at 2-ft intervals between stations  $10 + 00$  and  $11 + 50$ .

**SOLUTION:**

1. On the plan view, locate the stations for the elevations of the contour lines to be plotted along the PGL line. This can be done by mathematically calculating the elevations and their locations<sup>2</sup> or by scaling the location of the elevations<sup>3</sup> from the profile view. Depending on the longitudinal slope of the street, the scale of the plan and profile views and the desired precision, the latter may not be the acceptable method. The station of contour line 126 is determined by Eq. (24.2).

<sup>2</sup> Most engineers and designers have access to computer programs for hand-held calculators or microcomputers that provide elevations at incremental stations along the PGL.

<sup>3</sup> Scaling elevations is not recommended especially if the elevation can be obtained mathematically. Scaling is done only when a mathematical relationship is not known.

<sup>1</sup> This particular street section is used to illustrate the mechanics of plotting contour lines and is not a type of street section typically used in street design.

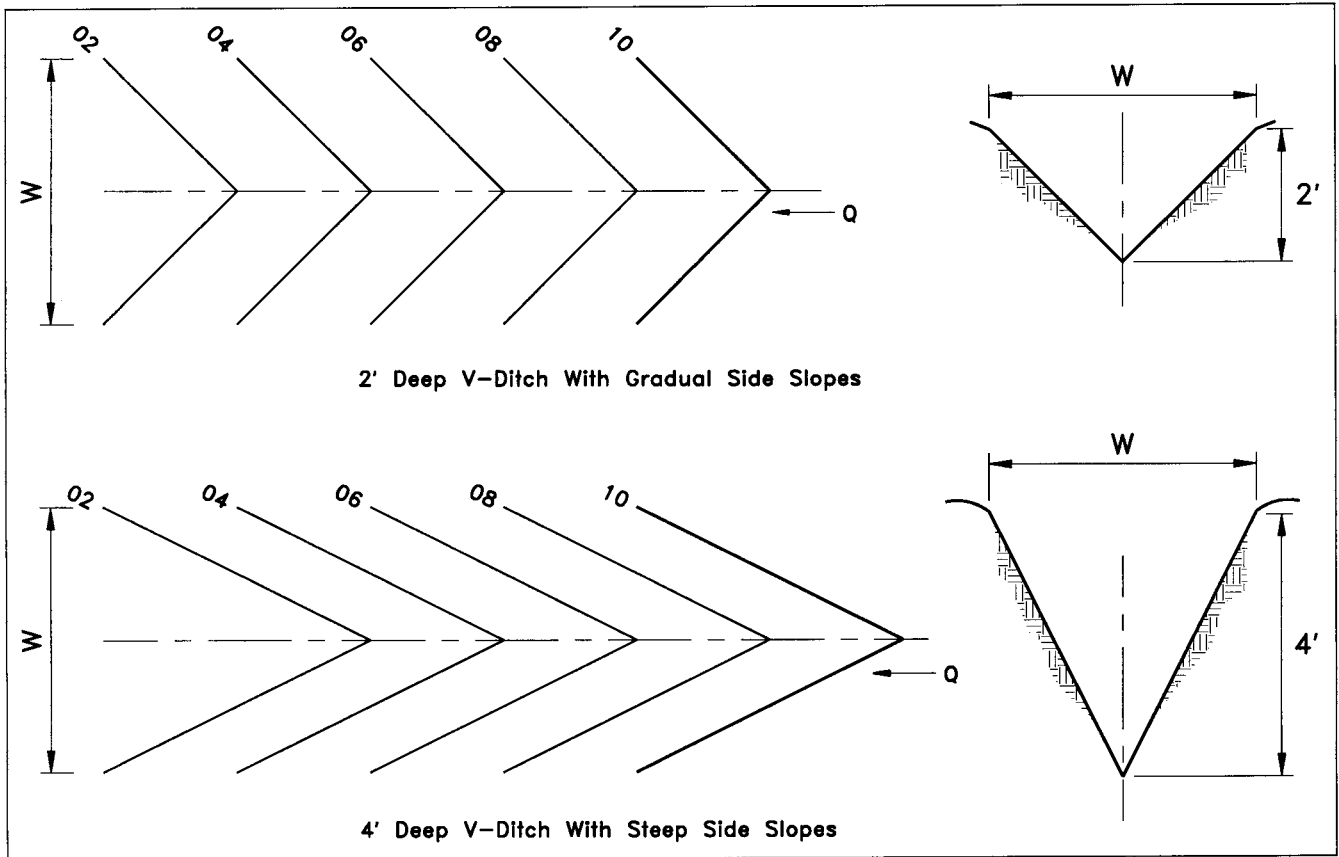


FIGURE 24.10 Example of 2-ft and 4-ft-deep channels with gradual and steep side slopes.

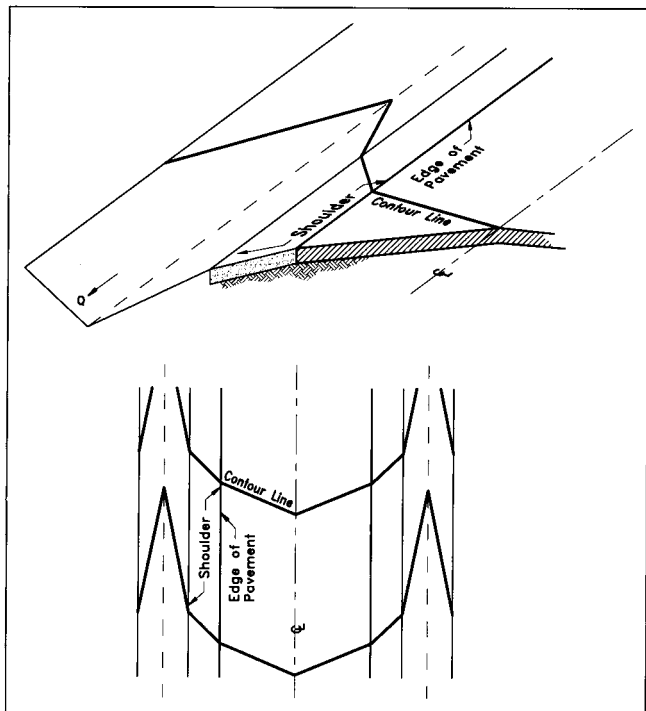


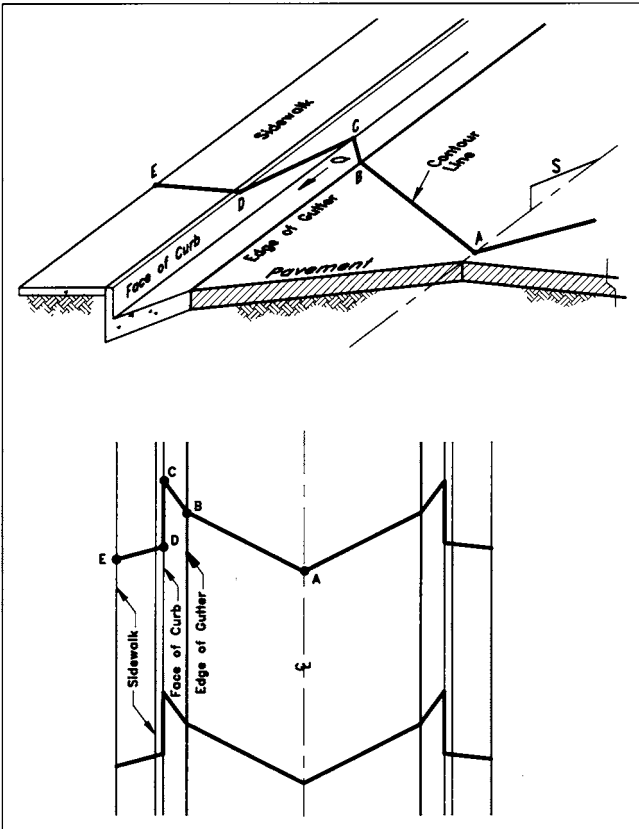
FIGURE 24.11 Contour line pattern for curb and gutter street section.

$$1050 - \frac{(126.0 - 125.5)}{0.045} = 1039 \quad (24.5)$$

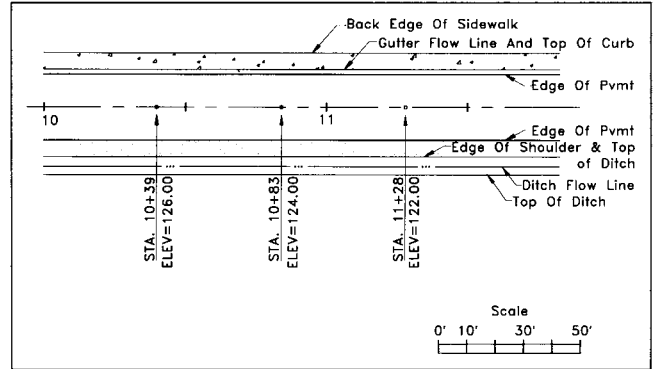
A uniform gradient of 4.5% corresponds to 2-ft change in elevation every 44.4 ft from Eq. (24.2). Therefore, contour lines 124 and 122 are located at stations 10 + 83 and 11 + 28 respectively (shown in Figure 24.14).

2. Calculate the elevation difference between the PGL and any other points of interest (e.g., the edge of pavement, the bottom and top of curb, and the back edge of the sidewalk). A 2% cross slope and 12-ft pavement width results in the edge of pavement being 0.24 ft lower than the PGL ( $0.02 \times 12 \text{ ft} = 0.24 \text{ ft}$ ). Table 24.1 lists the elevation difference for the other specific points.

3. Since the elevation difference between the PGL and edge of pavement is  $-0.24 \text{ ft}$ , elevation 124 at the edge of pavement is located at the station (on the PGL) where the elevation is 124.24, namely sta. 11 + 77.67. Mark the point on the edge of pavement that is perpendicular to the PGL from station 11 + 77.67. Similarly, the 124 contour at the flow line of the gutter occurs at the point perpendicular to the PGL point with elevation 124.41, namely sta. 11 + 92.33. The



**FIGURE 24.12** Contour line pattern for shoulder and ditch type of street.

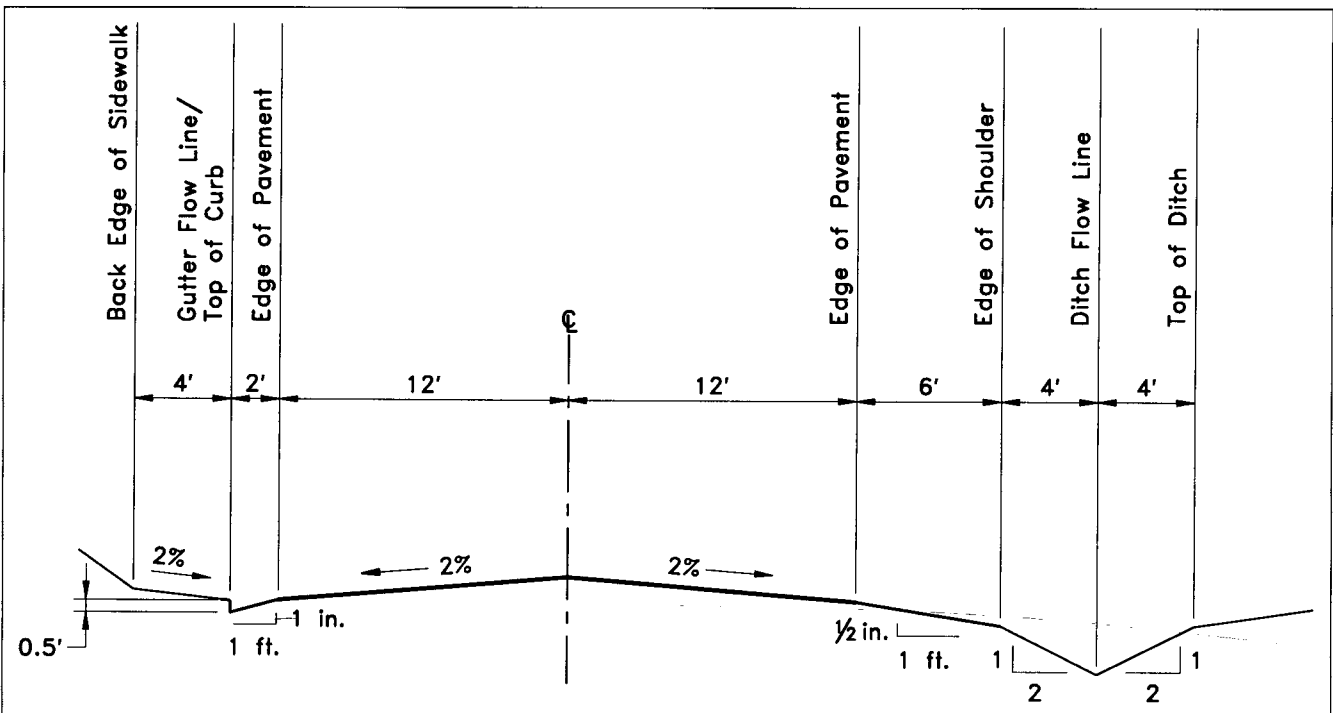


**FIGURE 24.14** STEP 1: Plot the location of the contour line elevations along the PGL.

124 contour at the top of curb will occur at a point perpendicular to the centerline point with elevation 124.09, namely sta. 11 + 30. This is done for all points of interest in the typical section (see Figure 24.15).

4. Connect all plotted points with a straight line. A constant longitudinal slope on a straight section of street results in uniformly spaced parallel contour lines (see Figure 24.16).

5. Since this straight section of street has a uniform longitudinal slope (i.e., it is not within a vertical curve), the other contour lines will all be parallel to this one. However, if the profile of the PGL is in a ver-



**FIGURE 24.13** Typical section for Example 1.

**TABLE 24.1 STEP 2: Compute Elevation Differences Between PGL and Points on Street Section**

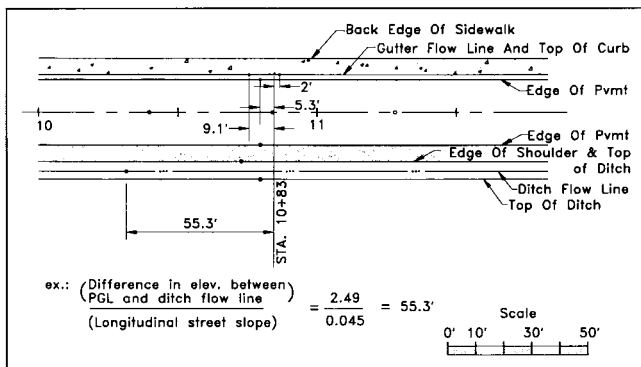
POINT ON STREET SECTION	ELEVATION RELATIVE TO PGL (FT)
Edge of Pavement/Gutter	-0.24
Gutter Flow Line	-0.41
Top of Curb	+0.09
Edge of Sidewalk	+0.17
Edge of Pavement/Shoulder	-0.24
Edge of Shoulder/Top of Ditch	-0.49
Ditch Flow Line	-2.49

tical curve, only steps 1 through 3 apply. Similarly, for streets with non-uniform cross slopes, new relative elevations need to be calculated (see Figure 24.17).

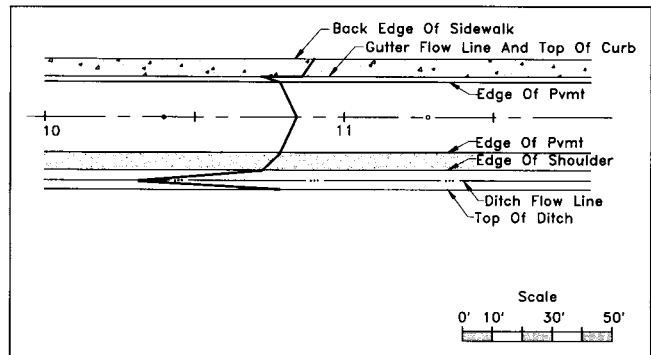
**Spot Elevations**

Often, contour lines alone cannot provide sufficient grading information to detail the existing ground conditions. As a result, the level of precision needed to construct the proposed features, detailed on land development plans, is not afforded by contours alone. Therefore, spot elevations are used to identify specific elevations at precise locations. For this reason, spot elevations take precedence over contour lines when determining grades.

A spot elevation is indicated in the plan view by a "+" symbol with the elevation written next to it. Spot elevations identify discontinuous or abrupt grade breaks in the ground surface, where straight-line interpolation between contours does not give the intended elevation. Therefore, spot elevations are used when the uncertainties associated with scaling distances and interpolating between contours cannot be tolerated.



**FIGURE 24.15** STEP 3: Plot location of the 124 contour lines at other points of interest on the street.



**FIGURE 24.16** STEP 4: Draw the contour line.

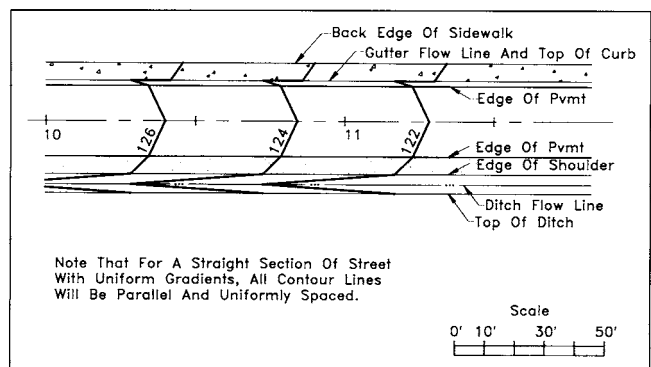
Typically, spot elevations are used for:

- Paved areas at points critical to construction, such as curb returns and parking lot corners
- Precise information regarding the tops of drainage and sewage structures
- Identification of high and low points in the grading scheme
- Description of retaining walls (i.e., top and bottom of wall elevations)
- Elevations at building entrances and corners

Note: spot elevations show up in a profile as an abrupt acute angle at the spot elevation location. The house-grading plan of Figure 24.30 illustrates the liberal use of spot elevations. Abbreviations are written next to the spot elevation when the elevation pertains to a specific feature, for instance, TC = 105.5. Selected abbreviations are given in Table 24.2.

**Cut and Fill**

The term "cut" refers to an area where soil is removed, while the term "fill" refers to the area where soil is added. Additionally, "excavation" refers to the removal of soil and material from an area, and "embankment" is used to reference



**FIGURE 24.17** STEP 5: Add the remaining contour lines following a similar procedure.

**TABLE 24.2 Spot Elevation Nomenclature**

ABBREVIATION	MEANING
TW/BW	Top/Bottom of wall
TC/BC	Top/Bottom of curb
FF El.	Finished floor elevation
BF El.	Basement floor elevation
HP/LP	High/low point
Inv. El.	Invert elevation
MH El.	Manhole elevation

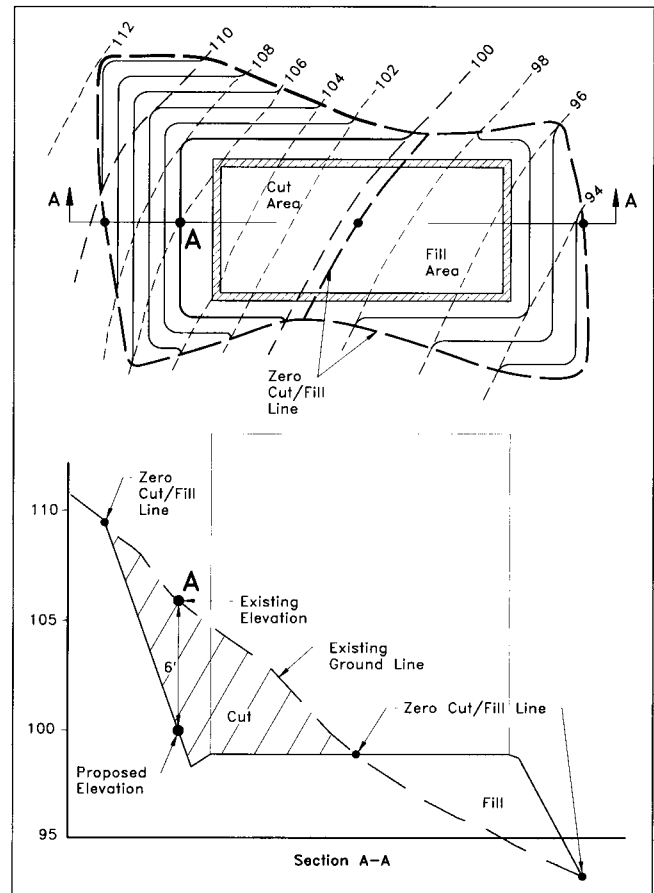
the addition of soil onto an area to bring it to grade. For example, a cut area is evident by the upward sloping ground along the sides of a road, while fill areas are evident as the downward slopes away from a building or street.

Cut and fill areas are indicated on the grading plan by comparing the existing contour lines to the proposed contour lines at a specific location. Where the proposed contour line elevation is higher than the existing contour line elevation, the area is a fill. Conversely, a cut area is one in which the proposed elevation is lower than the existing elevation.

Figure 24.18 shows a grading plan of a building with cut and fill areas. As an example of how to determine the depth of cut from comparison of contour lines, consider point A, where the existing 106 contour line intersects the proposed 100 contour line. The section view shows the 6-ft depth of cut at this point.

The left side of the building is a cut area and the right side is a fill area. The plan view shows a line around the cut and fill areas known as the zero cut/fill line. This line connects the points where no fill or cut occurs and separates the cut areas from the fill areas. Additionally, notice that the line also follows the points where the proposed contour lines connect to the existing contour lines around the perimeter of the graded area. Since the basement floor elevation has been established at elevation 99.0 ft, the zero line follows the existing 99.0-ft contour line through the center of the building. Although this grading plan shows only one cut and one fill area, other projects may have several areas of both types. The zero cut/fill lines are helpful for determining earthwork quantities, which will be discussed later in the chapter.

Consideration of cut and fill quantities is very important when developing a grading plan. For many reasons, a balance of cut vs. fill is usually desirable. Balance is achieved when the quantity of cut is roughly equal to the amount of fill. For example, in order to create a flat area on a hillside, it is most effective to cut into the hillside and use the excess



**FIGURE 24.18** Plan and section view showing cut and fill areas.

soil as fill on the lower portion of the site. This concept is illustrated in Figure 24.18.

One of the most compelling reasons to achieve a balance of earthwork is its effect on project cost. Moving soil around a site is less costly than either importing fill to the site or hauling excess cut from the site. Generally, importing fill onto the site costs more than hauling excess material away. Balancing the earthwork helps keep costs under control and often will help the finished site appear more in harmony with its surroundings.

The relationship between cut and fill is simple in concept, but other factors must be considered that complicate the equation. These factors include:

- Construction qualities of the specific soils on site. It cannot be assumed that on-site material may be used as fill, especially in areas where a load, such as a building or wall, is to be placed. The designer must verify the engineering characteristics of on-site soils rather than assuming they may be used as fill material. (See the chapter entitled *Soils* for further discussion).
- Soils generally shrink when used as fill, and the shrinkage factor can vary greatly between different soils. A grading plan that may appear to produce a



balance of materials may come up short on fill due to the shrinkage of the fill volume.

- A site developer may have several concurrent projects with different cut and fill needs. It may be desirable to produce intentionally a need for fill on one project in order to dispose of excess fill from another nearby project (hauling fill long distances is impractical due to high costs).

## DEVELOPING A GRADING STRATEGY

Now that the basics of grading, plan graphics, and grading mechanics have been discussed, it is time to combine these elements and develop a grading plan. Grading experience makes most of these considerations second nature, and enables the designer to concentrate on an overall strategy that produces the best possible plan.

### Grading and Drainage

Grading is necessary to enable a site to accommodate a proposed use. Perhaps the most important aspect of grading is to ensure proper drainage for the site. If a developed site traps water, causes it to flow to undesirable locations, or causes erosion, the grading has failed in its most basic requirement.

The designer must take into account the runoff that starts on the site as well as the runoff that flows onto the site from off-site areas. The drainage shed analysis serves as the basis for the design of all the proposed drainage structures and often influences the very layout of the site plan. For instance, the analysis may show that a large swale located on the site carries a considerable amount of water during a storm. The drainage structure required to convey the runoff underground proves to be too costly to construct; hence, the decision is made to leave the channel open and not construct site improvements over it.

The drainage study sets the basic parameters for the grading design. The following is a list of goals for a site-grading plan from a drainage perspective:

- Collect runoff and direct it safely to adequate outfall points at non-erosive velocities.
- Quickly convey runoff away from buildings to protect them from foundation damage and wet basements.
- Prevent the formation of unintentional wet areas that cause maintenance problems.

Experience and imagination both play a role in developing the final grading scheme. As the designer begins to work through the process, relationships between proposed improvements and existing conditions begin to coalesce. Important relationships begin to dictate patterns, such as existing elevations at site entry points compared to ground elevations at proposed building sites.

Drainage is conveyed either overland or underground. While the design of drainage facilities for concentrated

flows of runoff is covered thoroughly in the chapter entitled *Storm Drain Design*, this chapter will display a broader perspective, with a mixture of guidelines and rules, beginning with small flows.

Overland flow in its most benign form is called sheet flow, where little or no concentration of water exists as it moves across uniform, fairly level areas. Sheet flow is an ideal way to convey water because it helps absorption and is non-erosive. It is difficult to maintain sheet flow for large areas, however, due to water's tendency to concentrate. Left on its own, water quickly gathers into swales and ditches. The designer's job is to manipulate this transition from sheet flow artfully to shallow concentrated flow using sound engineering principles that accomplish the goal without obliterating the site. Although it may be possible to direct all runoff to its outfall points via overland flow, often it is advantageous to pipe it underground. Guiding the designer to either method are general rules-of-thumb that help simplify the process. A basic premise of grading is that a minimum slope of 2% needs to be maintained to drain runoff across non-paved areas. Decreasing the slope below 2% may result in sluggish drainage and standing water. It also can slow down the construction process. Very flat slopes are difficult to achieve in the field, even given the precision of modern construction equipment. Flat areas also remain wet for a longer time, and cause longer construction delays following rain. Yet, site constraints often require slopes of less than 2%. The use of flat slopes should be carefully considered and simply avoided if possible.

In some instances, the minimum desirable slope exceeds 2%, such as the grading adjacent to a building, where the objective is to move the water away quickly. The ground elevation at the building is referred to as the parge grade.<sup>4</sup> Dropping the proposed elevation at least six inches below the parge grade within ten ft of the building (5%) is ideal. Figure 24.19 shows the three basic methods for directing surface drainage away from a building. The ground beyond the parge grade directs the runoff to eventual points of collection. Although grades can vary beyond this point, a minimum slope of 2% helps prevent drainage problems.

Conversely, proposed grades that are too steep can lead to erosion and maintenance concerns. Steep slopes increase the velocity of water and therefore its energy, so that even relatively small amounts of runoff can erode large quantities of mulch or soil off a hillside. If grass is established on the slope, mowing is dangerous if slopes exceed 3h to 1v. While slope stability depends on many factors, they are all exacerbated with steeper slopes, especially in a fill condition. Local policy may also dictate the maximum (and minimum) slopes required for specific situations, and this makes a working knowledge of pertinent ordinances a valuable tool.

<sup>4</sup> A coat of masonry cement (parge) is applied to the part of the building walls below grade as a deterrent to moisture penetration. The parge grade is the elevation of the ground around the building sufficient to cover the parge coating.

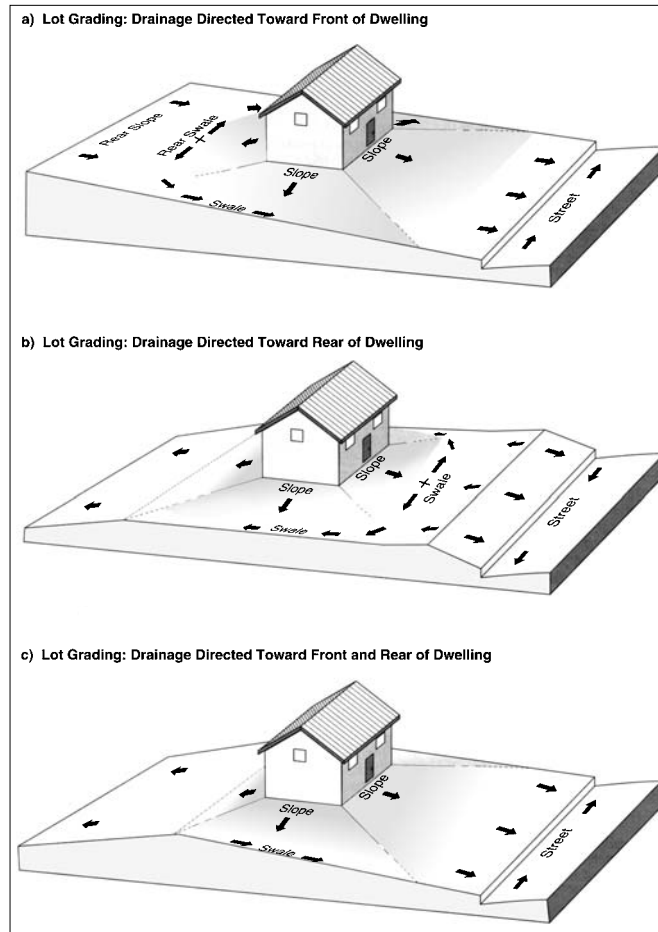


FIGURE 24.19 Schematic diagram for grading for drainage around a building.

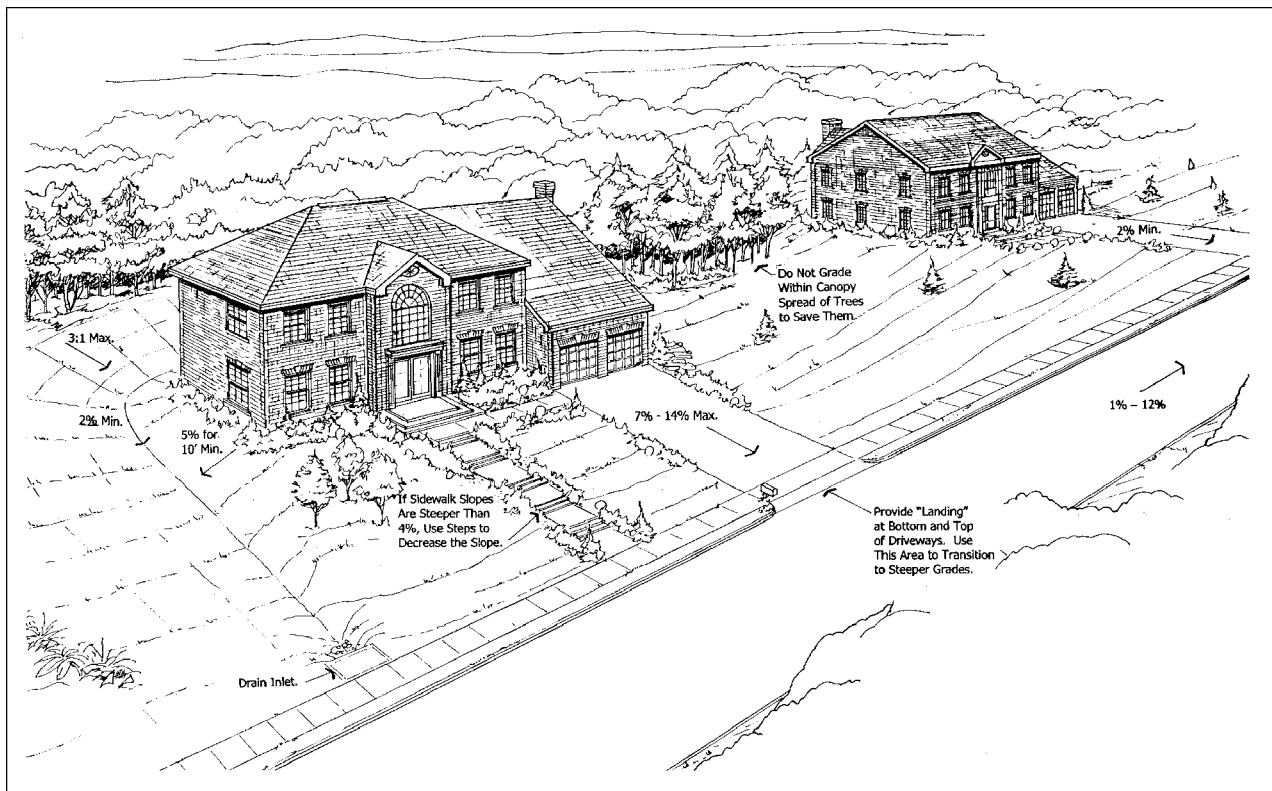


FIGURE 24.20 Illustration of grading strategies for subdivisions.

Minimum and maximum slopes are also an important consideration when designing roads, driveways, parking areas, sidewalks, and trails. Road design is a science unto itself, and the grading of roads is explained in the chapter entitled *Suburban Street Design*. More rules-of-thumb apply when planning for the movement of people across a site. Since the driveway entrance is the transition from the public or private road onto the site, the grading must facilitate its function. The following discussion explains grading for private and commercial driveways.

**Private Driveways.** A house set 25 to 35 ft from the street typifies the private driveway entrance. The slope of the driveway should be kept in the range of 2% minimum to about 7% to 14% maximum, depending upon whether the driveway is inclined up or down. The following points should be considered for driveway slopes exceeding about 5%:

- In snowy climates, steep driveways can become slippery and dangerous.
- If a walkway is proposed from the street to the house, a steep slope may require long flights of stairs, which are expensive to construct and can make the entrance to the house awkward.
- The transition from the street to the drive must be lengthened so that car bottoms don't drag.
- A landing with a slope of 2% should be provided in front of the garage that transitions to the steeper grade of the driveway.

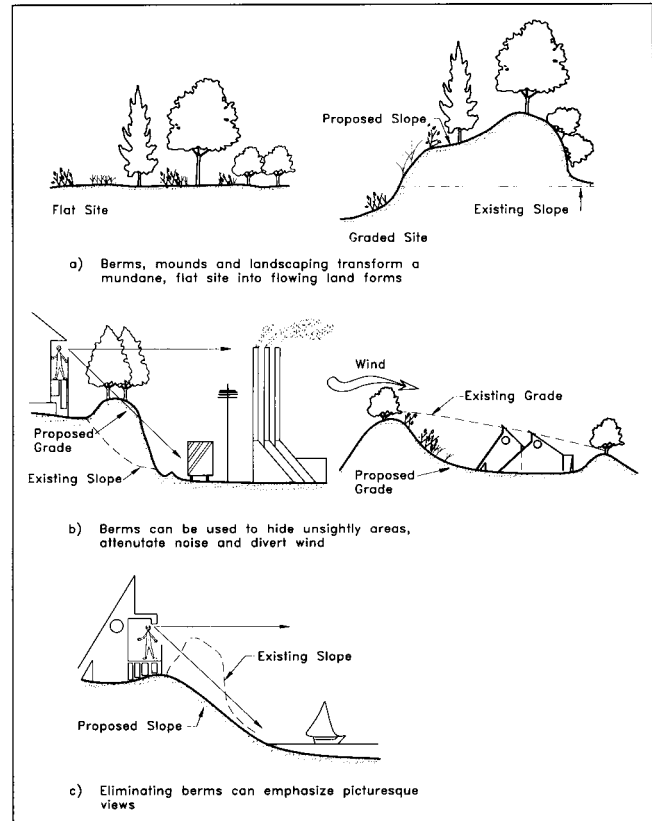
Such points of potential concern are illustrated in Figure 24.20.

**Public Driveways.** The higher traffic volumes encountered at public entrances dictate more conservative guidelines than private driveways. A landing at least 20 ft long with a maximum slope of 4% should be provided at the entrance. Grades on site should be in the 2% to 8% range. Local ordinances and requirements for access by physically challenged individuals (discussed below) must be incorporated into the plan.

### Grading and Aesthetics

Rules, regulations, and formulas have been discussed extensively above, and the mastery of the mechanics of grading will ensure a functionally sound site. But a functionally sound site is usually not enough in the competitive world of land development. A site must appeal to a user's aesthetic sense in order to be successful. Grading can transform a flat, featureless site into a visually pleasing series of rolling landforms that enhance the user's experience and create a higher demand for the property.

Figure 24.21 illustrates how grading can be used to enhance aesthetics. The grading itself can become the feature, as in the creation of landforms where none exist (24.21a). Grading can also be used to influence what we see by hid-



**FIGURE 24.21** Grading to improve aesthetics. (Courtesy of Nelischer, Maurice, ed., *Handbook of Landscape Architectural Construction* Vol 1, 2nd ed. Washington, D.C.: Landscape Architecture Foundation, 1985.)

ing a visually undesirable element, as shown in 24.21b, or opening up a view, shown in 24.21c. The landforms created for aesthetic reasons can simultaneously serve functional roles. The landform may aid in the balancing of earthwork by providing an on-site location to dispose of excess soil, or it may be used to direct wind away from buildings or outdoor-use areas.

However, using the grading to enhance the site aesthetically is not achieved by simply following formulas and rules; it must be coupled with a thorough knowledge of the site and the sensitivity to know what will work. Whereas a 20-ft berm on one site is appropriate, it might be totally inappropriate on another. Similarly, a small retaining wall used to help save a grove of trees may serve as the signature design element for a new project. Because most of these grading devices have an impact on project costs, the aesthetic gain must be compared to the extra construction expense. Sometimes the designer can justify the expense, but he/she must be prepared with lower cost solutions if the builder balks.

### Access for Disabled Individuals

Two legislative laws that may impact the grading scheme of certain types of construction are the Fair Housing Amend-

ments Act (FHA) of 1988 and the Americans With Disabilities Act (ADA) of 1990. Both acts deal with providing accessibility for individuals with disabilities. The FHA deals with multi-family housing of four or more dwelling units, and the ADA addresses accessibility to public places and commercial buildings such as hotels, stores, theaters, offices etc. Guidelines and standards adopted by the Department of Housing and Urban Development (HUD) for specific accessibility requirements of the FHA are given in Federal Register Vol. 56 No. 44 (March 6, 1991). Similar guidelines and standards for the ADA are developed by the Architectural and Transportation Barriers Compliance Board (ATBCB). The guidelines can be found in Federal Register Vol. 58 No.144 (July 26, 1991),

From the grading and site development perspective, HUD guidelines state that elevated buildings must have an accessible entrance on an accessible route, regardless of the terrain, unless site conditions and terrain make such accesses impractical. Because of the complexity of the requirements and its language, the reader is referred to the guidelines to make his or her own determination and interpretations. Local public works agencies are a good source of current requirements.

### Schematic Grading Analysis

Before detailed grading plans are underway, the designer should develop a schematic generalized grading scheme to determine any problem areas and get a feel for the type of limitations the site may have as design progresses. Developing a schematic grading plan consists of placing building footprints on the site and setting the building elevation, road grades, and parking area grades. Spot elevations and sketching the 5-ft, or sometimes 2-ft, contour lines helps determine the feasibility of the building elevations.

The road and parking area gradients for the schematic preliminary scheme are checked against any minimum requirements.

- Drainage patterns are analyzed
- Rough estimates of cut and fill quantities are calculated
- Steep slopes and retaining walls are identified
- Tree-save areas are identified

During this process, several grading schemes may result, each with its own advantages. Conferring with the client may help in establishing the favorable grading scheme.

### ESTABLISHING THE GRADING PLAN

A grading plan is established through refinement of different schemes through several trials. The first layout is rarely the best or accepted by the developer. During the first few trials, grades are adjusted to accommodate site constraints, earthwork, different building designs, and the preferences of the developer. The design is often done by hand on trac-

ing paper, but may be laid out in the computer in a digital format. The first few schemes may only involve siting the building, showing spot elevations at critical locations, and drawing several proposed contour lines. Additional detail is added to new iterations as the grading scheme is further refined. Once refined to the desired comfort level, the grading needs to be compiled in a reproducible format so that the designer can make work prints and distribute them to other members of the design team. Whether the grading was accomplished by hand drafting or through a digital application, it needs to adhere to general industry-wide graphic standards for both symbology and neatness to be effective.

In general practice, lighter, dashed lines are used for existing contour lines, while darker, solid lines are used to represent proposed contour lines. Note: the local jurisdiction may have specific graphic requirements for plans that should be considered as well. Additionally, drawing the existing contour lines on the back of the reproducible while placing the proposed contours on the front makes changing the proposed grading scheme easier. In a digital format, a variety of methods may be used to keep proposed information separate from existing. Either way, the goal is to create an easy-to-read plan that adheres to sound drafting principles.

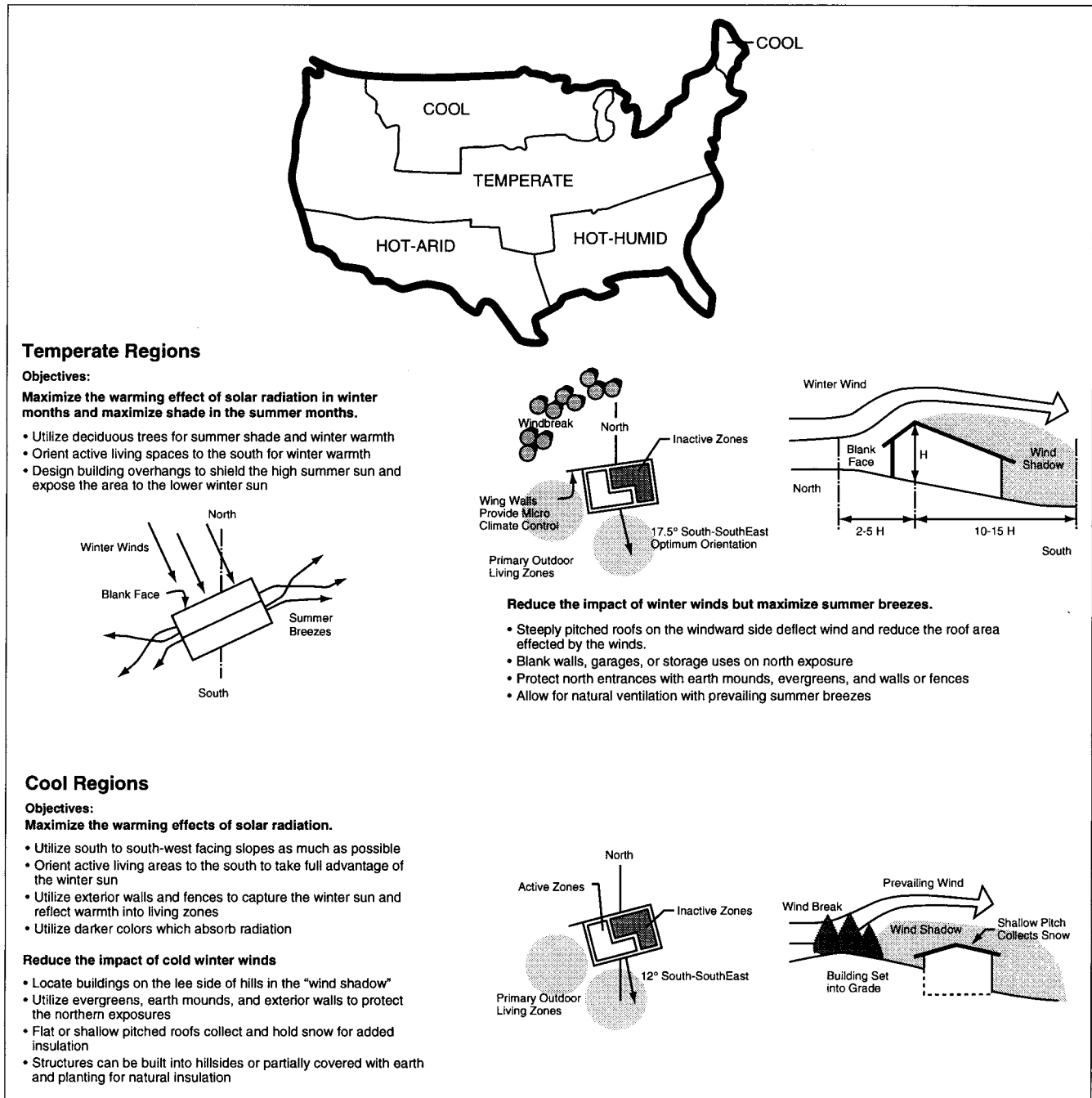
### Grading for Residential Purposes

Consumer appeal for a subdivision or a particular house type depends on numerous factors, one of which is appearance. The layout of the lots and houses, the style and type of house, as well as their spatial arrangement affect the overall appearance of the development and combine to form its character.

Frequently, residential land development projects incorporate several different house types and styles within a small price range. This practice is done for three reasons.

- The first reason is to accommodate the varying needs and the aesthetic tastes of buyers. The various house designs attract a wider range of consumers, thereby enhancing sales.
- The second reason is to take advantage of the varying topographical features of the lots. This provides the developer flexibility in the layout of the houses and reduces some of the construction costs.
- The third reason is for energy efficiency. Figure 24.22 identifies the considerations for room layout and house siting concerning climate conditions. Different house types offer varying responses to local climatic, slope, and orientation factors.

Professional and industry associations use different definitions in describing and categorizing houses. Some definitions overlap or conflict. Therefore, for the subsequent discussion the following definitions apply.



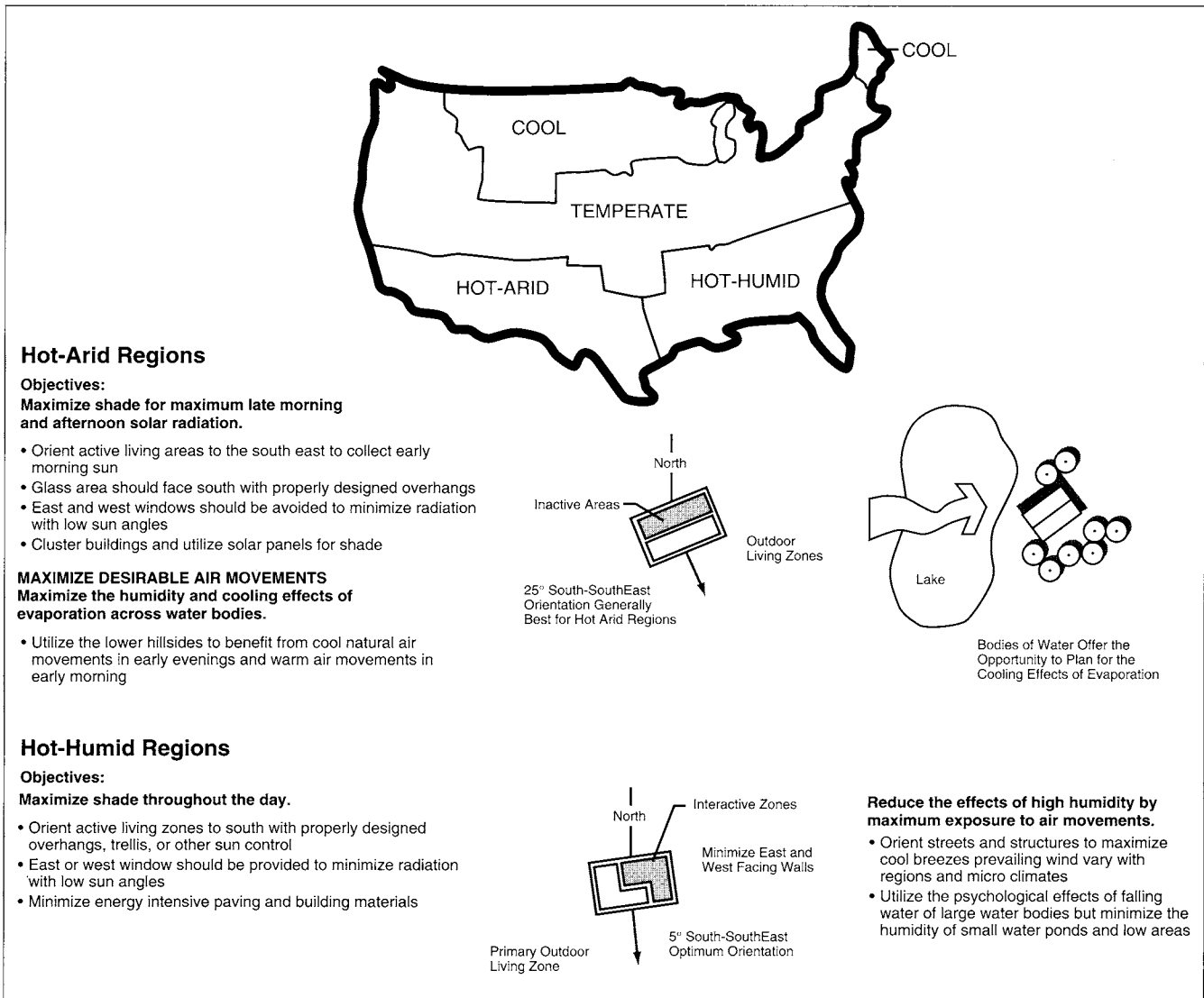
**FIGURE 24.22** Considerations for siting a building for different climate regions of the United States. (Courtesy of Robinette, Gary O., and Charles McClennon, *Landscape Planning for Energy Conservation*, New York: Van Nostrand Reinhold, 1983.)

**Architectural Style.** This is defined as the architectural features and configuration of a house that categorize it as belonging to a particular time period or region e.g., Colonial, Cape Cod or Victorian. (See Figure 24.25.)

**House Type.** This is the classification of a dwelling by the arrangement of floor elevations, location of entrances, location of walls, and property lines (e.g., 1-story, 2-story,

basement walkout and single-family attached and detached).

Not only is it imperative that the engineer developing the grading plan know the house types available in a subdivision, the engineer should also know the location and elevation of entrances, windows, garages, decks, patios, and roof lines. The engineer must consider each lot individually;



**FIGURE 24.22** (Continued) Considerations for siting a building for different climate regions of the United States.

however, he/she must also collectively consider the spatial arrangement of the houses along the street and within the development. In many suburban developments, the cost of land and the demand for greater living space forces the building of larger houses on smaller lots. The higher density produces a higher yield for the developer, and this in turn keeps housing costs down by reducing infrastructure costs on a per unit basis. However, smaller lots put houses in closer proximity, making house orientation a very important consideration. The designer must consider the views from one house to another, avoiding direct lines of site from one house into intimate areas of another house. For instance, no one wants to look out his front door and see into the dining area of the next house, or worse, to peer into the window of a second floor bedroom. Although grading can

be used to diminish the effects of poor siting, a good layout is paramount to a development's success.

Spatial arrangement is part of what is known as the streetscape. Setbacks, rooflines, utility corridors, street trees and landscaping, sidewalk and trail locations, and signage further define the streetscape. These elements work with the grading to lend the development a uniform character and create a cohesive, unified design.

A residential lot can be categorized according to the direction of the ground slope using the front property line as reference. A downhill lot is one in which the ground slope falls away from the front property line. An uphill lot has ground sloping upwards from the property line, while a side-to-side lot has slopes across its width. Efficient land use includes the selection of a house type that is compatible

with the terrain. Proper design and siting of the house minimizes earthwork and reduces the disturbed area, and this helps the environment and saves on construction costs.

For grading purposes, it is useful to separate the myriad of housing styles into one of the three following categories. The main difference between them is how their foundations are set up to deal with site grading.

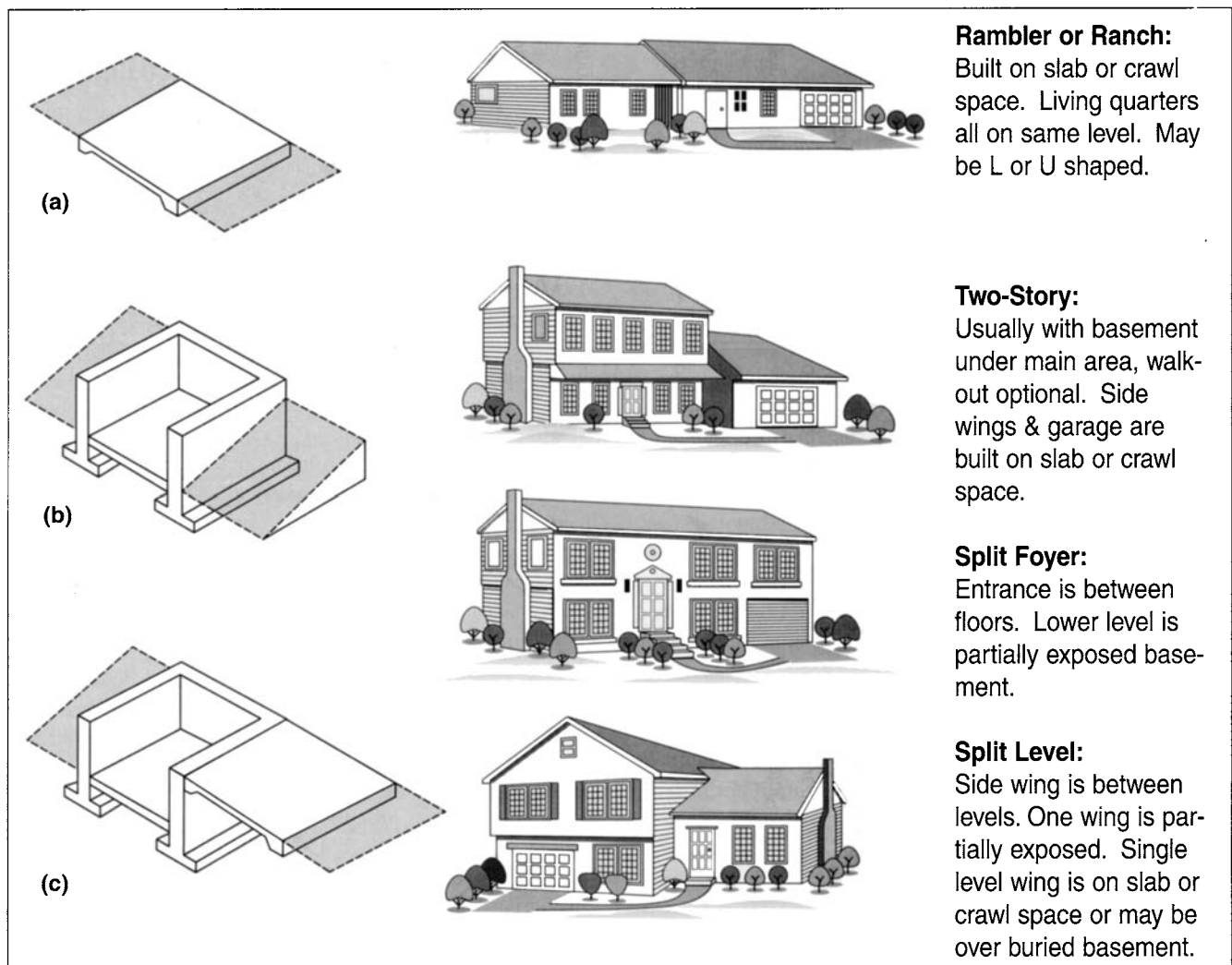
**No-Basement Units.** No-basement-type buildings are built on a concrete slab or crawl space and are best suited for flat areas with high water tables or where extensive rock lies near the surface. They are generally less expensive because of savings in excavation and construction costs. They are less suitable for hilly sites, since they require more grading to accommodate the building footprint. The no-basement unit is common in many regions of the country, and virtually every style of house can be constructed without a basement.

**Basement Units.** That part of the house that is wholly or partially buried is the basement. Basement units are useful on hilly lots because of their ability to accommodate grade

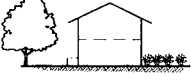
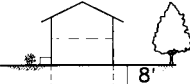
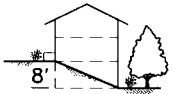
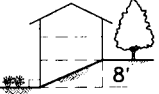
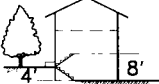

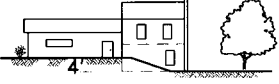
differences. For instance, the house shown in Figure 24.23b has a grade change of about 4 ft from front to back, and this may enable the proposed grade to tie-in with existing grade sooner, thereby reducing the disturbed area.

In addition to enabling the house to blend in with the site better, the use of basements can also generate small amounts of excess dirt that can be used elsewhere on a project. If a site is slightly deficient in fill dirt, basements can be incorporated to make up the deficiency. The use of basements also increases the living space of the house. This living space is enhanced by the presence of natural light, so often a partially buried basement that allows for window space is better than a completely buried basement. Another design element that enhances basements is the walk-out, which provides direct access outdoors from the basement. The typical site layout for a walkout is depicted in Figure 24.24.

**Split-Entry Units.** Floor elevations of a split-entry house are staggered such that their access from a preceding level is less than the full flight of stairs typical of the common



**FIGURE 24.23** Basic house types and their foundations.

HOUSE TYPE		COMMENTS
LEVEL UNIT		
 <p>2-Story House With Slab on Grade or Crawl Space</p>	 <p>2-Story House With Full Buried Basement</p>	1) Used on Flat or Mildly Sloping Lots.
FULL WALKOUT LOWER LEVEL (FRONT OR REAR)		
 <p>2-Story House With 8' Drop and Basement. Walkout From Rear</p>	 <p>2-Story House With Basement Entry from Front and 8' Rise to Rear of House</p>	2) Used to Make Up Grade Difference on Lots With Severe Front to Rear Slope Conditions.
SPLIT ENTRY UNIT ( $\pm$ 4' UP OR DOWN)		
 <p>2-Story Split Entrance With Rear Walkout From Lower Level</p>	 <p>2-Story Split Entrance With 4' Rise to Rear of House</p>	3) Used on Lots With $\pm$ 4' Fall or Rise Front to Rear
SPLIT LEVEL WITH SIDE TO SIDE DROP IN GRADE		
 <p>Side Split With Grade at One End of House 4' Higher Than the Other and Walkout From Lower Level</p>		4) Used on Lots With Side-to-Side Grade Differences of 2' to 4'

**FIGURE 24.24** Selection and location of building types that fit natural land forms.

2-story unit. Split-entry houses and their variations are ideal for all types of hilly lots. These houses work best when the grade difference across the lot is 2 to 4 ft. Houses in this category are frequently referred to as split-levels and split-foyers. Split-level houses have a combined slab and basement foundation system as shown in Figure 24.23. Similarly, the basement of a split-foyer is partially exposed. Figure 24.23 shows the basic house types and their corresponding foundations. Figure 24.25 illustrates more contemporary examples of the basic house styles.

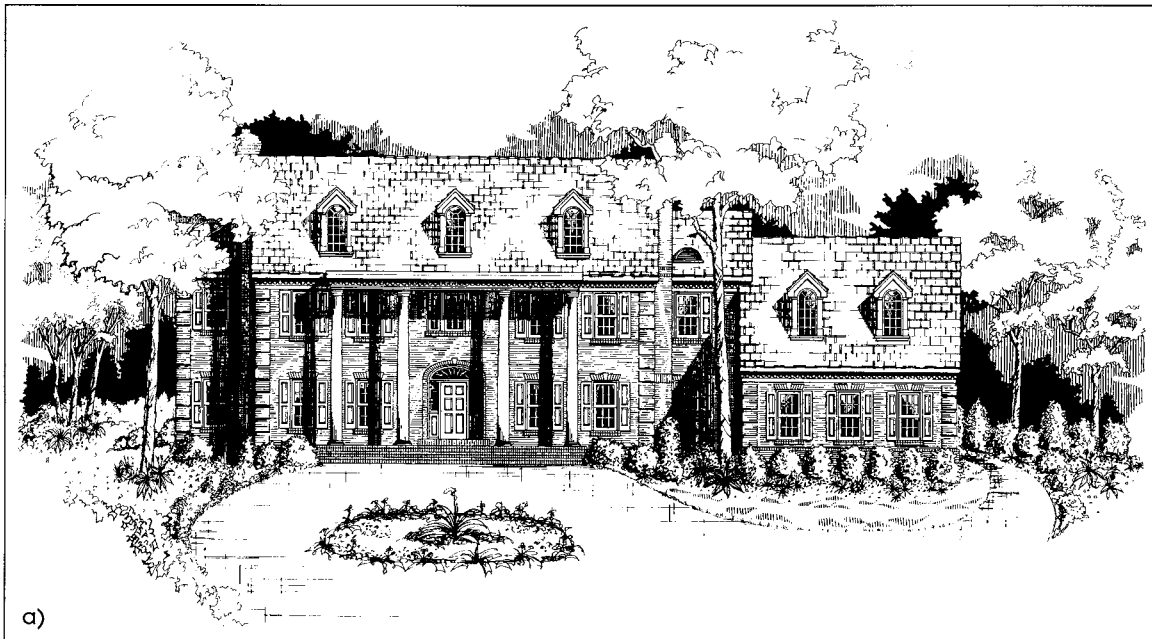
**Siting the House and Grading the Lot**

When siting a house on a lot, one must position the house horizontally and vertically in a manner harmonious with

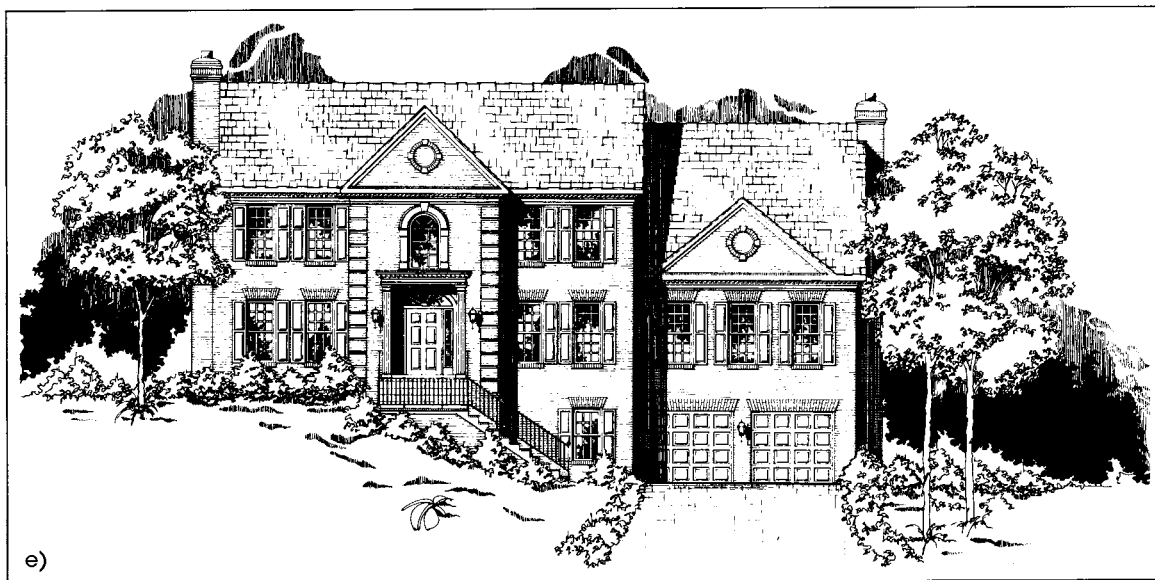
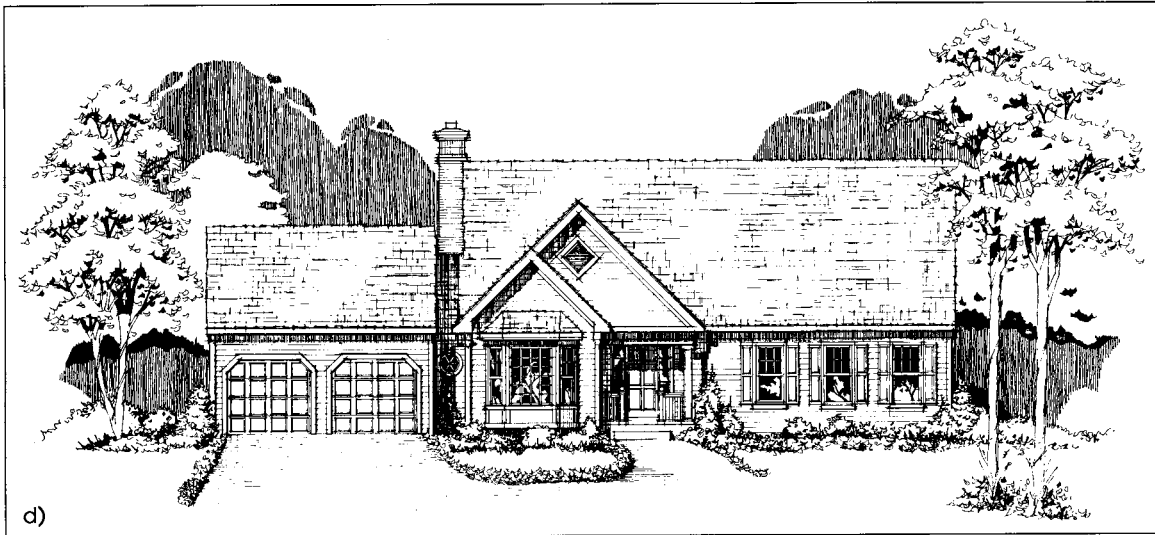
the surrounding structures and terrain, while ensuring compliance with all appropriate codes and ordinances. In most residential, single-family-detached subdivisions of moderate density, siting a house is limited to the lot layout of the subdivision and the orientation of the street. Presumably, solar exposure and compass orientation was considered when the lot and street layout were established in the schematic design stages, since there is not much practicality in orienting the house for energy efficiency if it is in conflict with the street and lot layout.

The house type is selected based on the topography of the lot, adherence to all applicable setbacks, and any constraints by the client (see Figure 24.24). Using generic building footprints, the engineer produces the block-





**FIGURE 24.25** Examples of different house types: (a) Southern Colonial (b) Victorian (c) Tudor. (Courtesy of Sutton, Sullenberger Yantis Architects.)



**FIGURE 24.25** (Continued) Examples of different house types: (d) Rambler (e) Side-to-side/Uphill (f) Split Foyer.

grading plan, which defines the general grading patterns for groups of lots. This plan verifies the site's feasibility regarding a fairly specific program and is often used by a developer to market the land to builders. The goal is to produce finished lots that are then sold to the builder. The builder can then use the block-grading plan to check the feasibility of a very specific program. Usually, the engineer portrays an exact footprint of the builder's house on a proposed lot, precisely measures its distance to the property lines, and produces a lot-grading plan, the final step in siting a house on a lot.

The above scenario illustrates the two types of grading schemes: lot grading and block grading. Lot grading involves only one building and one lot. Grading is limited to the constraints at the boundaries of the lot, and any grading beyond the lot requires permission from the adjacent owner. Typically, lot grading applies to infill projects and commercial projects. Block grading involves grading a group of lots, a frequent occurrence in single-family residential projects. Block grading is not bound by the rigid constraints of lot grading since the whole group of lots is owned by one entity. Additionally, the spatial arrangement and the drainage pattern can be integrated much more easily.

In addition to the relationship of the house to its lot and surroundings, another relationship that affects the siting of a house is the elevation and location of the sanitary sewer. As shown in Figure 24.26, the house should be sited in

such a manner as to provide a gravity sewer for the lowest level of the house. A quick way to determine approximate finished floor based on the sewer line elevation is:

- Determine the invert of the sewer main at the connection point.
- Multiply the distance from the connection point to the farthest point of the house by  $0.0208 \times (1/4 \text{ in./ft})$ , and add to the invert elevation (the fall,  $f$ , in Figure 24.26).
- Add the diameter of the sewer main (distance  $d$  in Figure 24.26).
- Add an allowance for clearance under the foundation (shown as \* in Figure 24.26).

Typically, the house sewer runs under the footing for shallow footings and foundations. Therefore, the elevation of the lowest level should be raised an additional 6 in to 1.5 ft (depending on local construction practices) to allow for clearance beneath the footing and floors. In situations where it is not feasible to run the house sewer line under the footing, the house sewer line can be run through the footing. Figure 24.26 shows this concept.

When a basement, because of its depth related to the sewer main in the street, cannot be sewerd using a gravity line, the sewer is referred to as a hung sewer. A pump may be installed to eject sewage from the basement to the sewer

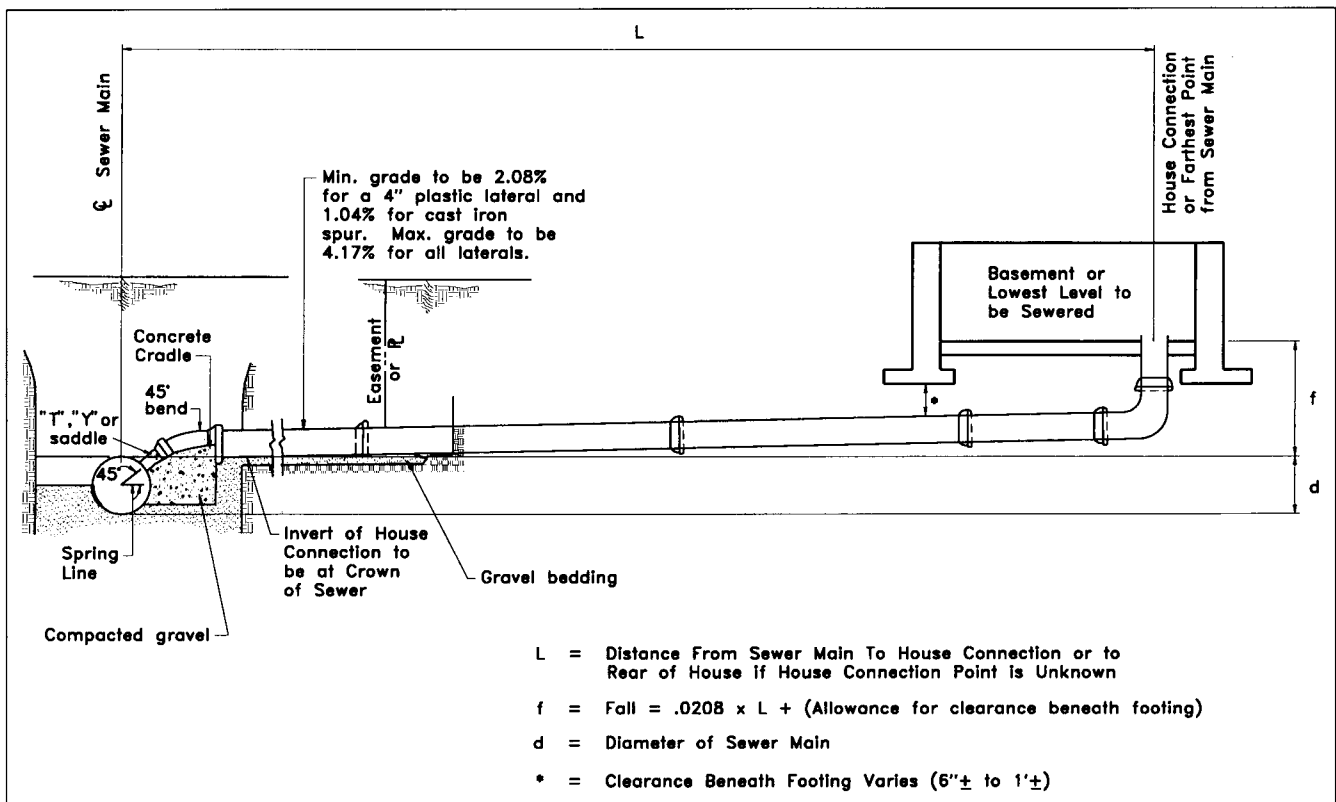


FIGURE 24.26 Sewer line from house to sewer main.

main if it will not flow by gravity, but this practice may come under special regulations in many jurisdictions. Other solutions, such as lowering the sanitary main, may be feasible, but the engineer must consider the impact of such a solution on the rest of the project. Careful planning while designing the grading plan will help avoid awkward situations such as basements that won't sewer by gravity.

Another controlling factor for setting the house elevation is the street elevation. Typically, the first floor elevation is above the street, but limitations on driveway grades, as discussed above, also control the maximum elevation of the house. The relationship of the house to the street is a major factor in its "curb appeal," and any deviation from general standards (e.g., a very steep driveway or slope that drains to the foundation of the house), looks out of place and lowers the curb appeal.

Finally, the development of any grading scheme includes the assimilation of other existing data and the constraints that data may impose. These existing constraints are combined with the data available on the proposed building structure. Ideally, these constraints are obtained from the approved<sup>5</sup> architectural and mechanical drawings. The location and relative elevations of doors, garages and windows, other appurtenances such as decks, patios, stoops, walkways, and any utility connections are necessary to develop an effective grading plan. This information becomes part of the constraints for grading. From this architectural information, develop a footprint or template of the building showing all entrances, windows and other appurtenances at or near the ground level that affect the grading. The relative elevations of these items with respect to the floor elevations are labeled on the template. Figure 24.27 depicts examples of house templates for three different house types used on a project. The building template is placed on the grading plan and oriented such that it best suits the topography, ordinance setback requirements, and utility constraints. The specific footprint of the desired house is then transferred onto the grading plan after the best orientation of the template is obtained. Since a typical subdivision has several house styles, the use of templates allows the designer to shuffle the house types around the lots for quick analysis.

Additionally, siting of buildings on corner lots must allow for site distances of the intersecting streets. This includes high walls and steep slopes resulting from the grading that might impede the required sight distance. However, in most cases the required zoning building setbacks and minimum radii of curvature allow for adequate sight distances.

The height of the earth fill around the house depends on the exterior surface and type of foundation system. Soil and accompanying moisture accelerates decay of some types

of construction materials and provides a haven for insects that can damage the house. Figure 24.28 shows the relationship of the exterior grade for selected foundation systems. The architectural plans should be consulted for the relationship of the flooring to the foundation wall, since setting the first floor elevation dictates the maximum ground elevation. The ground elevation around the perimeter of the house is adjusted to account for windows, doors, garages, and other architectural features, all of which are indicated on the house template.

Once the house is positioned on the lot, the elevations at all critical points are shown with spot elevations. Typically, stoop, patio and ground elevations outside of doors are six in (minimum) lower than the first floor elevation. Additionally, elevations are shown at windows located near ground level to ensure they are not buried. Walkways leading from the front door to the driveway or street should not be steeper than 5%. If walkways become too steep and steps cannot be used, then the overall house elevation may have to be raised or lowered to account for such constraints.

When grading out a rear yard, the designer should seek to provide an area sufficiently level as a place for lounging and family recreation. A space of 15 to 20 ft by 30 to 40 ft graded to a maximum of 5% to 6% slope immediately behind the house is reasonably adequate. In areas of steep slopes, where a flat yard of this size would require expansive fill slopes, the size of the back yard may be reduced, with perhaps a large deck proposed to provide the outdoor living space.

As shown in Figure 24.29, spot elevations and drainage direction arrows are used liberally on the grading plans to show the contractor the intentions of the engineer for the design. Note that the house types of Figure 24.27 correspond to the grading plan of Figure 24.29.

After all of the houses are set on the lots, the engineer should notify the developer of the proposed mix of house types since many developers know, from experience, what mix will be successful. In fact, sometimes the developer gives the engineer a range on the mix before any grading plan is started. Nonetheless, the engineer should verify the mix of house types to be sure no changes in the developer's program have occurred before proceeding with the project.

### Grading in Townhouse Development

Townhouse developments consist of parking areas, private streets, open space, and blocks of "attached" residential units. Typically, the blocks or sticks of residential units contain approximately three to nine homes. The interior units share a wall with either neighbor, while the end units have only one wall common with an adjoining unit. Typically, the property line runs down the center of the common wall.

Although a block of townhouses may have several house types, variations in architectural style and different lengths, usually they have a constant width. These variations help avoid the visual monotony that would occur if all the houses aligned and had the same style (see Figure 24.30).

<sup>5</sup> There are frequent changes in drawings during design and it is difficult to disseminate this information to all people involved in the design of the project in timely fashion. Final coordination of the architectural drawings with the civil site design must be checked when all plans are finalized.

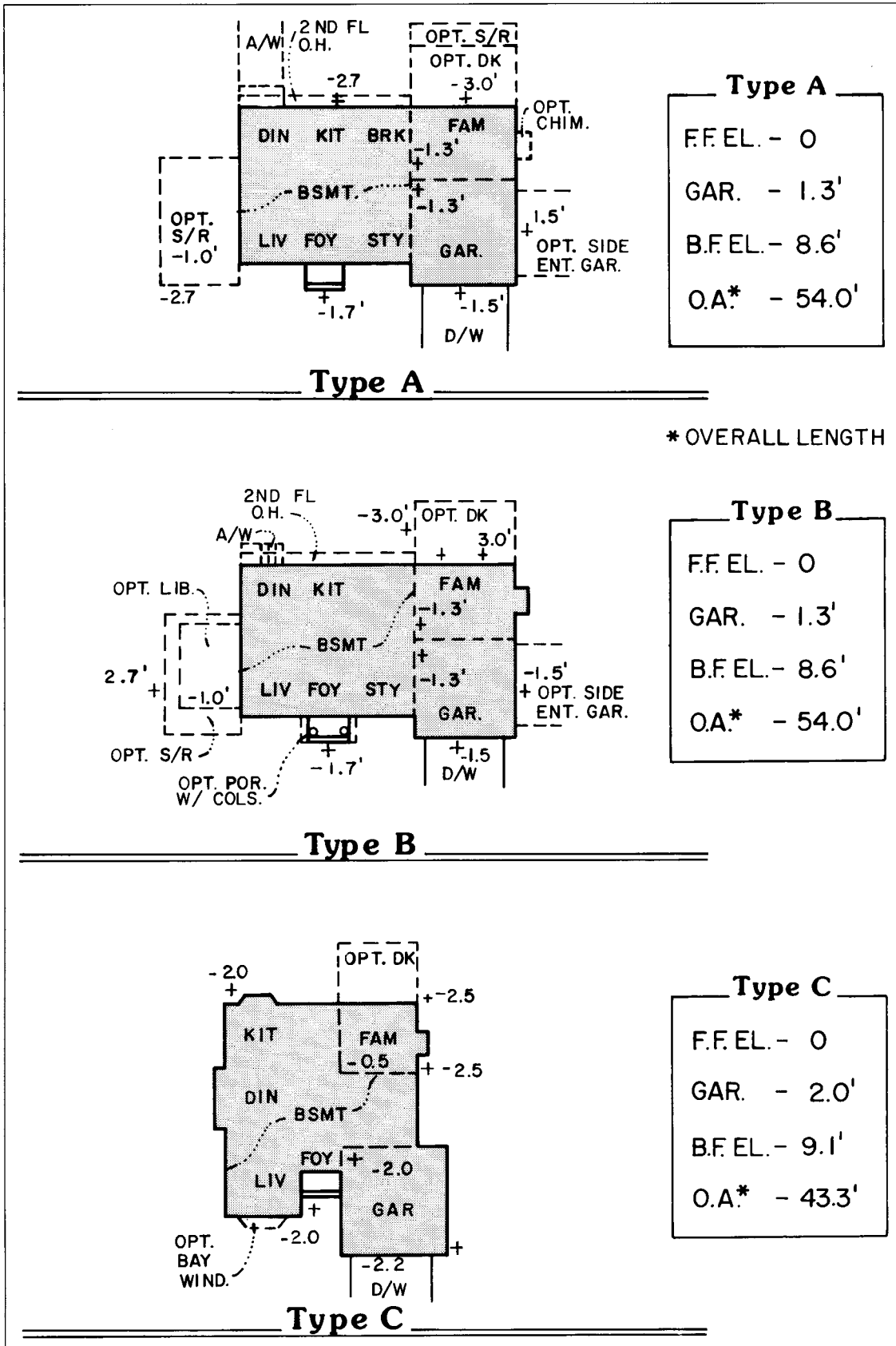
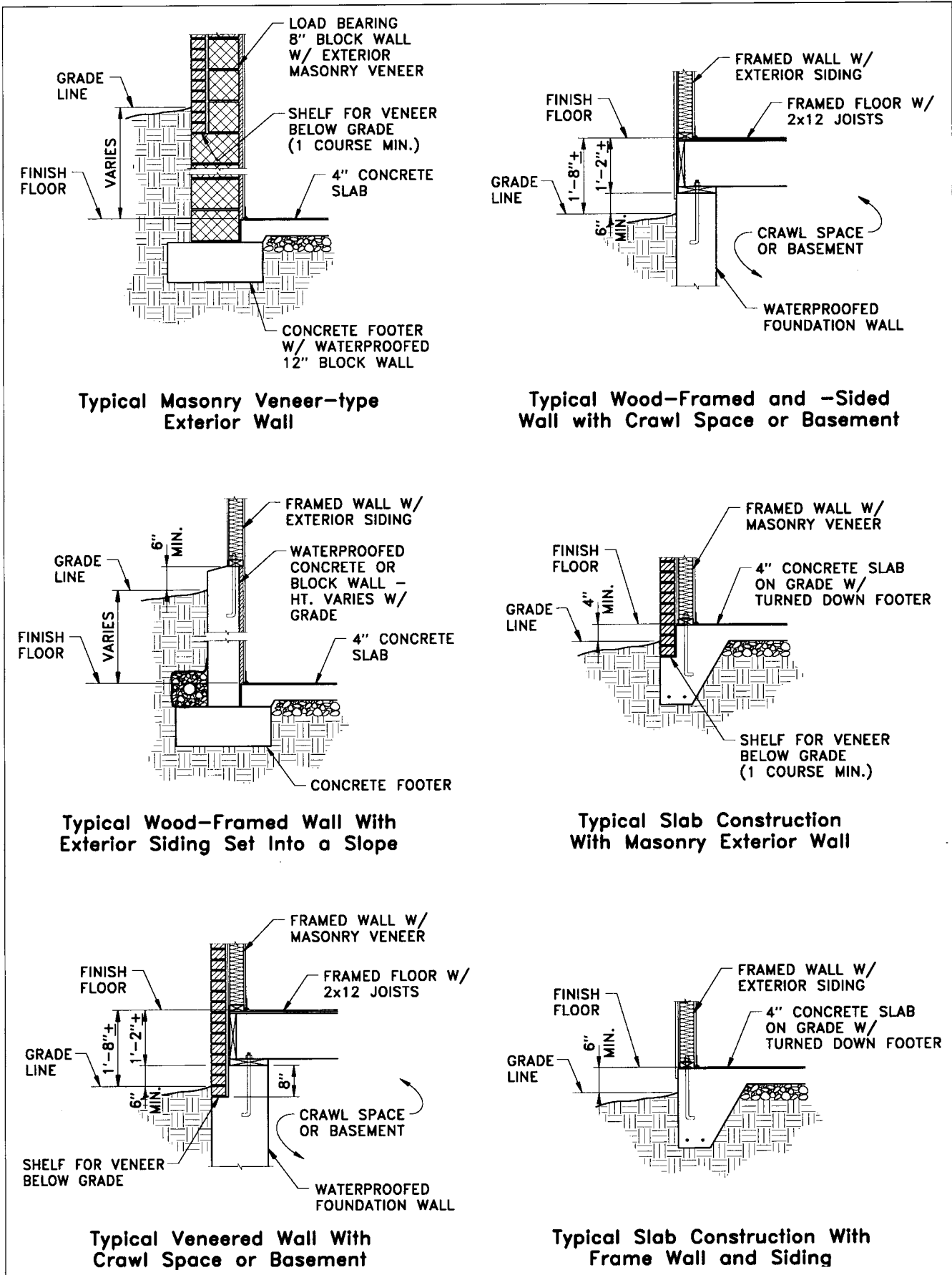
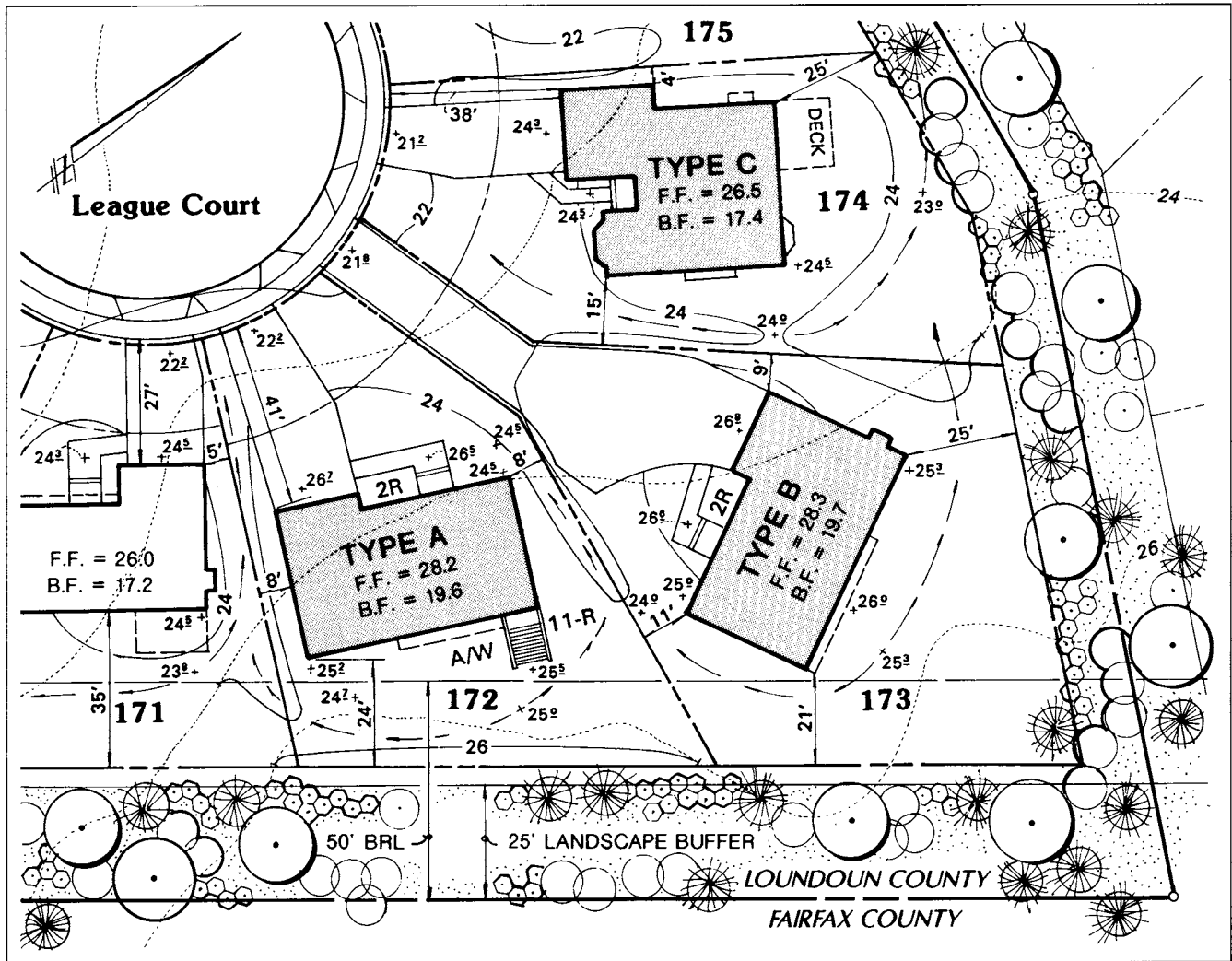


FIGURE 24.27 House template.



**FIGURE 24.28** Foundation systems, exterior surfacing and grading. (Reprinted with permission of C. Harris and N. Dines, 1988, *Time Saves Standards for Landscape Architecture*. New York: McGraw-Hill.)



**FIGURE 24.29** House grading plan.

Other variations include vertical and horizontal staggering, which provides flexibility in the layout and helps the sticks conform to sloping sites.

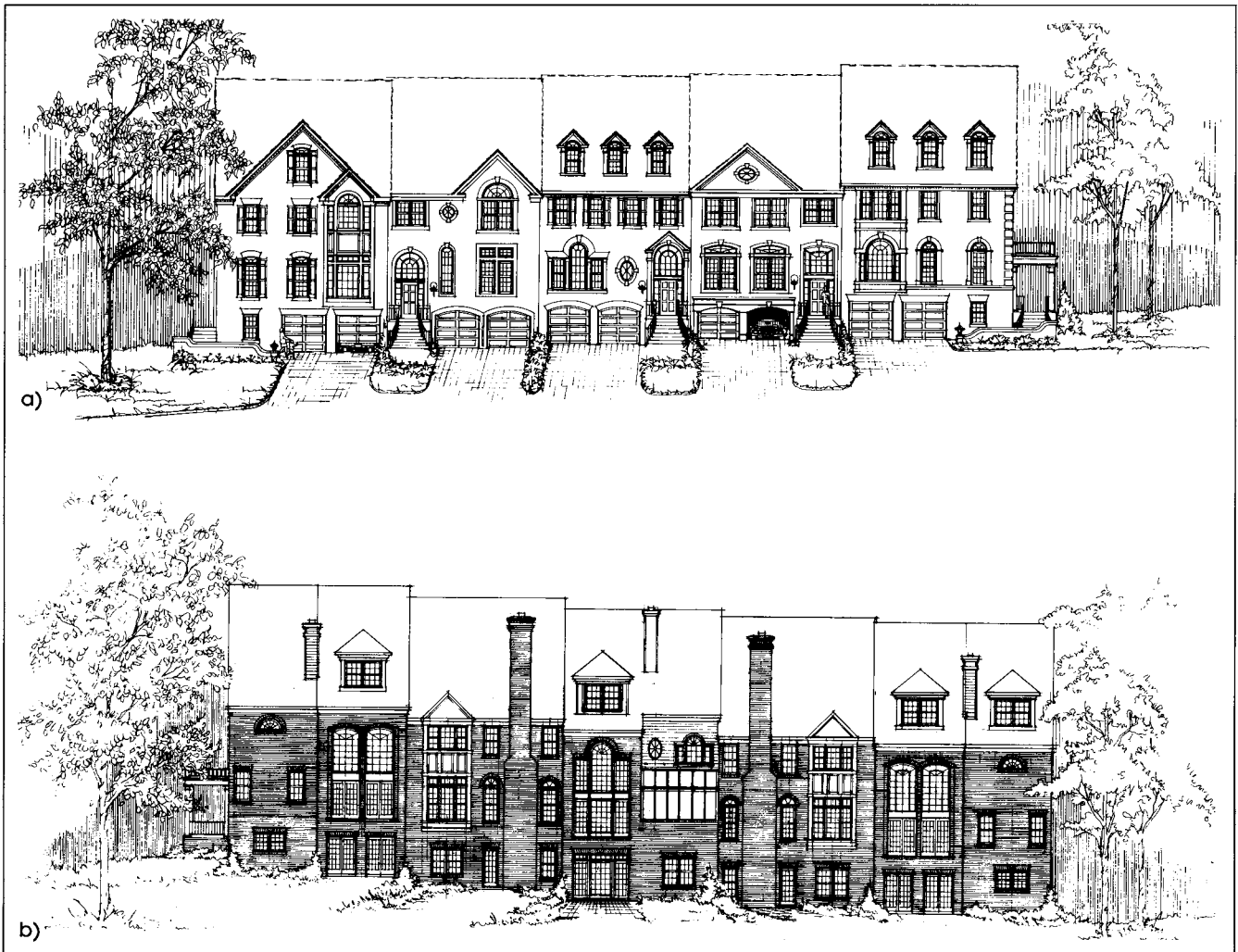
Since each unit is attached to another unit within a block, there must be coordination in the structural design with adjacent units (e.g., roof lines, wall framing, etc). The minimum or maximum variations in horizontal and vertical relationships may be dictated by local ordinance and building codes. Certainly, restrictions on staggering and vertical orientation will be limited to the structural and architectural design.

From an engineering aspect, the units have to coincide with the site constraints. Yet, from a sales perspective the units have to be appealing to the consumer. For this reason, the layout and design of a townhouse project requires extensive communication and coordination between the site designer, developer, and architect.

During the preliminary phase of design, the site designer obtains general criteria regarding the size, limitations on horizontal and vertical placement, location of garages, pat-

ios, decks and other constraining information about the units, all of which impacts the grading design. During the preliminary-design phase, several layouts and rough grading plan designs might be necessary to optimize the site. The preliminary lot layout provides information on the number of blocks, the number of units in each block, and a rough idea of the vertical arrangement of the units. From these preliminary studies, the architect develops the final arrangement of the units. During final design, the site designer determines the floor elevations of the units. The architect's drawings guide the site designer in setting the floor elevations.

Some refinement of the elevations shown on the preliminary plan is necessary at the final design stage. Whereas the preliminary study may have shown elevations to the nearest foot or half foot, final design drawings show floor elevations to the nearest hundredths of a foot. Therefore, deviations from the preliminary design are expected. In addition, the architect's drawings may also have to be refined during final engineering design. Occasionally, the architect



**FIGURE 24.30** Townhouse block showing vertical staggering to accommodate the longitudinal slope: (a) Front elevation; (b) Rear elevation. (Courtesy of Sutton Sullenberger Yantis Architects.)

allows some flexibility in adjusting the vertical and horizontal orientation of the units. Adjusting units several inches or even a foot or two may present no critical dilemma. When vertical separation between units is two ft or more, small retaining walls are required.

Setting the floor elevations for townhouses is more involved than single-family detached houses. A certain limited structural relationship, evident from the architectural plans, exists among the units of a block. In some townhouse designs, there is flexibility in adjusting the stagger distance and the vertical relationship of floor elevations of adjacent units. This flexibility is needed to allow for better coordination between the housing units and the site conditions.

To set the townhouse elevations properly, the engineer needs a complete and current set of architectural plans. From these plans, the site designer determines the relative location of the units in a block. As with single-family homes, a template of the townhouse block may simplify the process. This template shows the relative floor elevations of

the units and the location of stoops, patios, decks and other appurtenances. When vertical adjustment between units is necessary, it is recommended, and usually mandatory with brick veneer, to raise or lower floor elevations in eight-in increments. This recommendation is based on the dimensions of masonry units used for construction. Using 8 in. or a multiple of eight simplifies the construction while still providing flexibility to the designer.

Depending on the experience of the site designer and architect, the site designer may be the one who sets the pad elevations and staggers the setbacks. This information is then sent to the architect, who then develops the block designs. Whether the site designer suggests the initial unit requirements to the architect first, or vice-versa, communication between the site designer and architect (through the developer) is imperative to exchange the interdisciplinary information necessary for design.

Another problem encountered in setting townhouse elevations is the number of risers (steps) from the street to



the door. The site designer needs to verify the horizontal and vertical distance necessary to accommodate the full run of the stairs. Zoning ordinances may not allow the steps and stoop to encroach into the building setback limits. The site designer must verify that the vertical distance from the door (or stoop) to the ground, and the horizontal distance from the door (or stoop) to the property line, is capable of accommodating the required number of risers. A safe assumption is to allow a tread width of 1 ft; therefore, if ten risers are needed, the minimum setback to the stoop is 10 ft. Furthermore, additional distance may be needed between the sidewalk and the start of the steps. Although riser heights can vary with the maximum height typically limited to  $7\frac{1}{2} \pm$  in., building codes require the riser height to be constant for the run of steps. Therefore, the vertical distance from the sidewalk/street to the entrance must be a multiple of the riser height.

### Grading in Commercial Sites

Commercial projects (e.g., high and low rise offices, retail plazas, houses of worship, and schools) mainly consist of buildings, parking areas, and access points. Due to the high premium rates for commercial land, rarely are there large areas of natural ground to allow for extensive flexibility in the grading scheme. On residential projects, groups of lots are worked on simultaneously, and the grading can extend beyond the limits of a single lot, since the group of lots has only one owner. On commercial projects, rarely does one owner develop several parcels simultaneously; therefore, the limits of the grading cannot extend off site. Grading on an adjacent parcel, with a different owner, requires permission in the form of a letter of agreement or an off-site grading easement, either of which is often difficult and expensive to obtain. Therefore, assume that proposed grades must tie-out at the property line. When this practice requires expensive retaining walls or severely curtails the development program, then it is appropriate to explore the off-site permission. On small sites, this limitation makes developing a grading plan more challenging, especially when steep slopes exist.

The controlling factors for grading commercial sites are basically the same as other sites, and include drainage, slopes in parking and pedestrian areas, and access points to the building structures. From an aesthetic point of view, the grading concept for commercial sites is often tied to the visual goals that the architect (and developer) set for the proposed building.

The necessity for large expanses of paving on commercial projects leads to rules-of-thumb somewhat unique to this genre. Remember the requirements for commercial entrances discussed above. From the entrance, the recommended pavement slope in the travel lanes varies from 1% to 5%. The pavement is sloped to direct runoff to curb inlets, sump areas, or ditches off the edge of pavement. Placement of all drainage structures should take into consideration the movement of pedestrians and vehicles. Inlets

should not be placed in areas of heavy pedestrian use, such as crosswalks and curb-cut ramps. Additionally, the designer should locate inlets in areas where people can access their vehicles without stepping around the inlet. Figure 24.31 shows recommended placement of inlets in parking areas.

Another rule-of-thumb is that long runs of sheet flow on steep slopes in parking areas should be avoided, especially in colder climates where the sheet flow can freeze and create hazardous conditions for pedestrians and drivers. Additionally, runoff should be directed away from sidewalks and pedestrian travelways, as this not only makes it easier for walking but also reduces the splash from passing traffic.

When devising the grading scheme in larger parking areas, consideration should also be given to paving operations. Parking areas that have extensive wash-board effects are difficult to pave, especially if there are numerous grade breaks and relatively steep slopes.

Many commercial buildings have ramps that lead to underground parking or to loading areas. The grading should direct the runoff away from the ramp areas, while inlets at the bottom of the ramps carry the small amount of runoff that does fall in the ramp and loading area.

### Grading and Software

The process of grading explained in this chapter has developed over many years; most of the concepts presented have been in use long before the advent of computers. These concepts change very little, and they must be thoroughly understood before trying to grade a site. Only when the designer has this understanding should tools such as computer programs be employed.

Both AutoCAD and Microstation have software packages designed to perform many tasks associated with site grad-

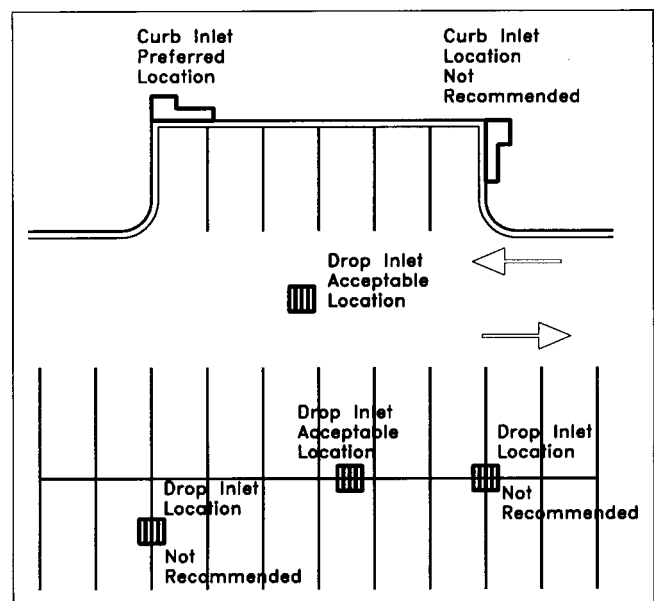


FIGURE 24.31 Recommended inlet locations in parking areas.

ing. By assigning a third dimension to the two-dimensional line, a  $z$  value, a three-dimensional entity is created, and this allows grading to be accomplished in the digital world. The use of computers theoretically leads to increased precision and productivity.

Most of the more mundane tasks associated with grading have been automated. Today, programs are available that not only aid in site grading but may actually design the site grading. Despite the level of sophistication within these programs, it is still imperative that the person applying the technology know what to tell the machine and what the machine is telling him. Only then can the grading plan truly represent the best solution.

### Conclusion

The person designing the site grading plan must combine several skills to accomplish the task. Knowledge of plan graphics and the mechanics of grading provides the means to represent the scheme on paper. Identifying the natural and manmade constraints and knowing how to work with them ensures feasibility. Manipulating the grades so that the proposed uses are facilitated and enhanced contributes to the plan's viability. Creation of an attractive site that is enhanced by the grading helps marketability. Accomplishing all of the above requires a combination of art and science rarely equaled in any other aspect of the site development process.

### EARTHWORK

Calculating the amount of displaced material is referred to as an earth take-off (ETO). Although rarely the case, the ideal scenario is to have a balance of earthwork, where all of the required excavation is used to backfill and to bring the lot to finished grade. In order to attain any semblance of an earthwork balance, the excavated material must be of adequate quality to be reusable. In projects with excessive rock, loam soils, or expansive soils, an earthwork balance, in all likelihood, is unattainable. If a balance cannot be attained, the next best alternative is to have excess soil, since hauling the excess away from the site is typically less expensive than hauling in borrow material.

A grading plan may have to go through several iterations before an acceptable earthwork balance is obtained. Frequently, a rough grading plan is developed as a first approximation. This rough grading plan shows the buildings, streets, and parking areas with spot elevations at critical points and contours lines with 2- or 5-ft contour intervals. An ETO is performed to determine the net earthwork quantity. Since this rough grading plan serves as an unpolished first guess for design, the earthwork analysis cannot be extraordinarily detailed. As the rough grading plan is refined and approaches final design, the detail and accuracy of the earthwork analysis increases.

Most earthwork calculations are performed with the aid of earthwork software, although occasion does arise when manual methods are more effective. Of the several methods

presented herein, the one selected depends on the type of site and the way the project is set up. For many projects, the ETO is developed from the contour grading plan, or in the case of roads, the cross-section drawings. Of major interest to the developer or contractor is the net amount of cut and fill material. Therefore, in order to get a final quantity, cut and fill values must be adjusted for such things as topsoil, pavement thicknesses, undercut, large conduits, soil shrink-swell, and other factors. Many of these factors are fully discussed in Chapter 40, Soils.

### Cross-Section Method

The cross-section method is used to calculate earthwork quantities for roads, utility trenching, and other projects when the length is greater than the width. A contour grading plan is not necessarily needed to use this method since cross sections for streets and prismatic channels can be obtained from the typical section and the profile. Street cross sections are easily plotted manually, although computer software is available to produce street cross sections more easily.

If the cross-section method is used for a building site, the grading plan is used to develop the cross sections. A baseline or reference line is drawn on the plan. Although the location is arbitrary, it is usually down the center or along one edge of the project. The baseline does not need to be straight. Slight curvature or angles can be used when the situation warrants. Lines perpendicular or radial to the baseline are drawn where the cross sections are desired and the sections are then plotted. Distortion of the topography as it appears on the cross-section plot increases when the section line is significantly skewed relative to the prevailing land slope. This, in turn, contributes to errors in the cut and fill values. On the other hand, the distance is not uniform between two section lines if they are not perpendicular to the baseline, and this also adds to the error in the ETO. Judgment and experience dictate the orientation of the section line relative to the baseline and prevailing land slope in order to obtain reasonably precise ETO quantities.

Section lines do not have to be at constant intervals. They are located where the cut and fill does not substantially change and at points where there is an abrupt change between cut and fill. For example, cross sections located before and after foundation walls will not account for the excavated volume. Additional cross sections should be included just inside of the foundation walls. Precision of the ETO depends on judicious selection of the cross-section locations and their orientation to the baseline.

Once the existing and proposed grades are plotted on the cross sections, thicknesses for pavement, concrete slabs, and subbase depths are then added to the drawings. The gross cut and fill quantities are adjusted according to these depths. The cut and fill areas of each section are determined by using a planimeter, geometry or grids. The latter way is tedious, has inherent errors (e.g. estimating partial segments of the squares), but can be effective for very rough esti-

mates. If the measured area from the cross-section drawing is in square inches, the conversion to actual square feet based on the horizontal and vertical scale is

$$SF_{act} = A \times H_{scale} \times V_{scale} \tag{24.6}$$

where  $SF_{act}$  = actual square feet,  $A$  = measured area in square inches,  $H_{scale}$  = horizontal scale in feet per inch, and  $V_{scale}$  = vertical scale, also in feet per inch.

$$V_{inc} = \frac{A_i + A_{i+1}}{2} \times L \tag{24.7}$$

The incremental volume of cut and fill between successive cross sections is equal to the averaged area of the cut or fill multiplied by the (average) length between the cross sections as given in Eq. (24.7), where  $V_{inc}$  is the incremental volume of material between two consecutive cross sections,  $A_i$  and  $A_{i+1}$  are the areas of cut or fill on the two consecutive cross sections, and  $L$  is the average horizontal distance between the sections. Summation of the incremental cut and fill volumes determines the total cut and fill.

The volume from the average end area (Eq. (24.7)) significantly overestimates the volume at the two end sections. As shown in Figure 24.32, the solid segment bounded by station 10 + 60 and station 11 + 00 is wedge-shaped. The volume for a wedge (pyramid) is

$$V_w = \frac{A}{3} \times L \tag{24.8}$$

where  $A$  is the area of the first cross section, in this case station 11 + 00. A comparison of the volumes for the

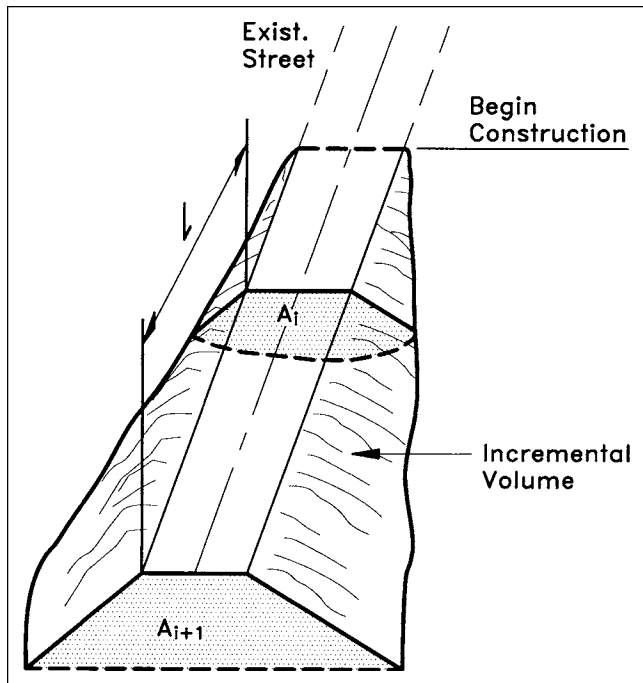


FIGURE 24.32 Wedge-shaped volume and prismatic volume.

wedge segment as computed by the average end area and the wedge equation shows that the average end area volume is 50% greater. The volume due to this error in this segment must be weighed against the total volume of earthwork. Since the error applies only to the two end segments, it is left to the judgment of the engineer which equation is used for the wedge-shaped segments.

**EXAMPLE 2:** Figure 24.33 shows three cross sections of a roadway where C and F designate the cut or fill area on the cross section. Table 24.3 shows the tabular organization of the data to determine the earthwork quantities using the cross-section method. (Assume stations 0 + 50 and 2 + 50 are the begin and end points of construction.)

The accuracy of this method is determined by the variation of cut or fill areas of subsequent cross sections. Although most cross sections are taken at constant intervals, intermediate cross sections may be necessary. If one cross section is in total fill and the following cross section is in total cut, an intermediate cross section is necessary for a higher degree of accuracy. The intermediate cross section is located where the fill section transitions to the cut section; this is the point where the cut and fill areas are nearly zero. The accuracy of this method can be increased if the interval length is reduced. The trade-off for this increased accuracy is the time to evaluate more cross sections. Another factor affecting accuracy is the cost

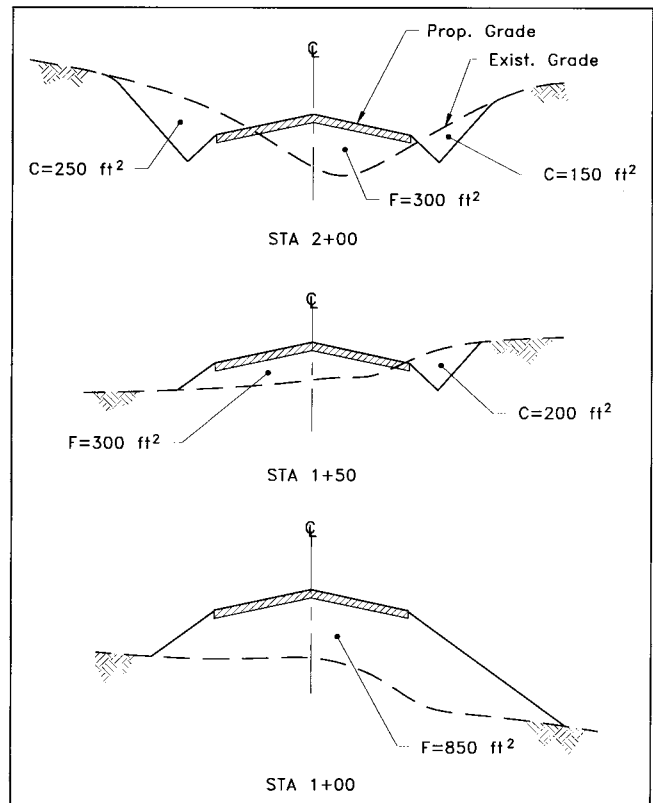


FIGURE 24.33 Earthwork example: cross-section method.

**TABLE 24.3 ETO by Cross-section Method**

(1) STATION	(2) LENGTH (FT)	(3) AREA (FT <sup>2</sup> )	(4) AVERAGE AREA (FT <sup>2</sup> )	(5) INCR. VOLUME (FT <sup>3</sup> )	(6) ACCUM. VOLUME (FT <sup>3</sup> )
FILL					
0 + 50		0			0
	50		425	21,250	
1 + 00		850			21,250
	50		575	28,750	
1 + 50		300			50,000
	50		300	15,000	
2 + 00		300			65,000
	50		150	7500	
2 + 50		0			72,500
CUT					
0 + 50		0			0
	50		0	0	
1 + 00		0			0
	50		100	5000	
1 + 50		200			5000
	50		600	15,000	

Column 1: STATION—the stationing along the street that identifies the location of the cross section.

Column 2: LENGTH—the length between the two cross sections. Notice this value is written on a line between the two cross sections of column 1.

Column 3: AREA—the total area of the cut or fill for the section. The first and last stations are the stations where the grading begins and ends. Here the cut and fill areas are zero.

Column 4: AVERAGE AREA—the average of the cut or fill area as determined by equation (24.7).

Column 5: INCR. VOLUME—the volume of cut or fill between the two cross sections. This is equal to column 2 × column 4.

Column 6: ACCUM. VOLUME—the accumulated volume of cut or fill. Equal to the preceding value of column 6 added to the current value of column 5. The last value in this column is the total cut or fill for the total number of cross sections.

for cut and fill. An extremely high cost associated with the cut or fill operations will necessitate that the computed earthwork quantity be considerably more accurate. Consideration for additional cross sections through a curve is necessary if the length parameter is longer than the arc length of the curve. In general, the average end area equation tends to overestimate volumes.

A variation of the average end area method considers the average the areas within a closed contour loop. The average area is multiplied by the difference in elevation between the contour lines to obtain a volume. This method is best suited for computing volumes of lakes and mounds.

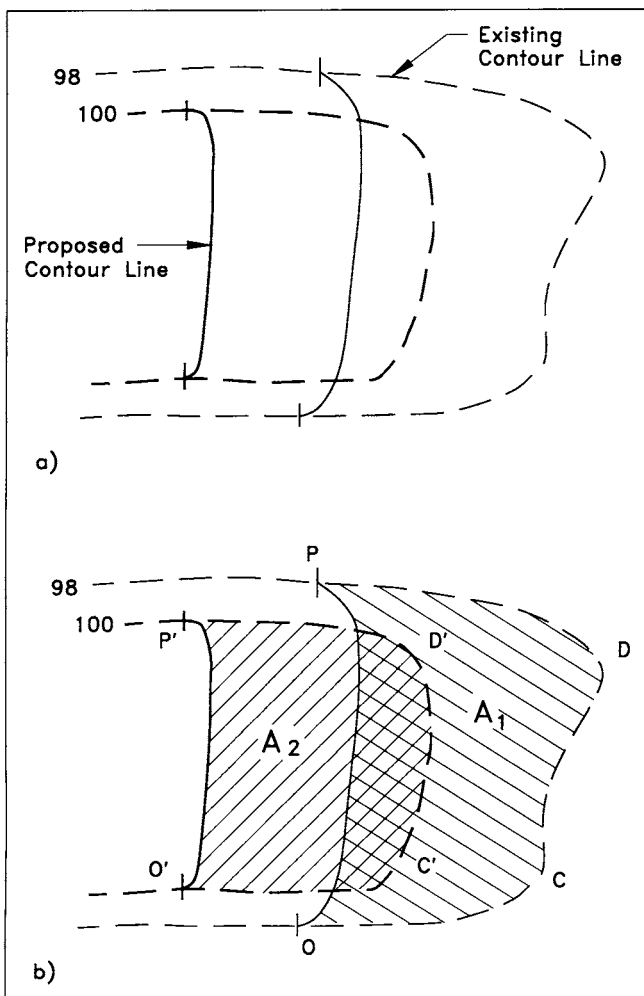
**EXAMPLE 3:** Table 24.4 organizes the data for calculating the volume of the depression in Figure 24.34. The values for the area (shown in column 2) represent the plane area contained within the contour line. Column 4, *ht*, is the difference between the elevations of the contour lines.

**The Contour Method**

The contour method requires a contour grading plan. The amount of cut or fill material is determined by averaging the change in areas due to grading on two successive horizontal planes. This area is contained within the loop of the existing contour line and the proposed contour line. This averaged area is multiplied by the distance between the two

**TABLE 24.4 Contour Method: ETO for Fill Area**

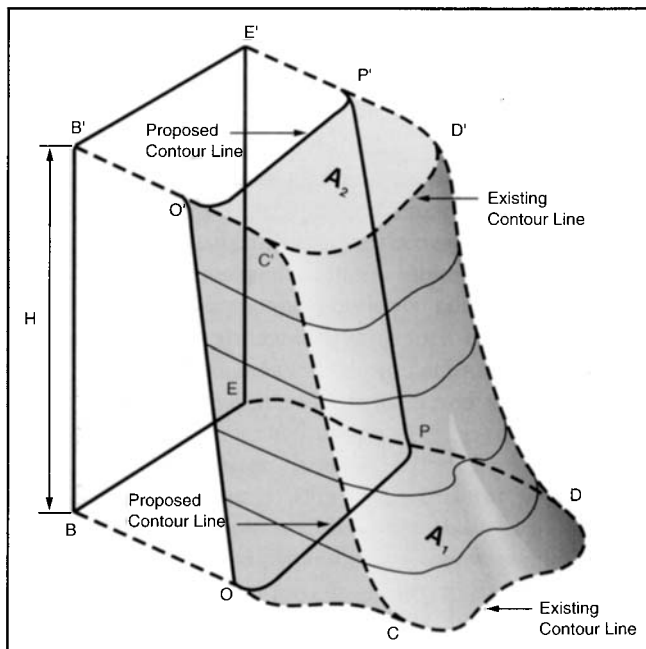
(1) ELEV. (FT)	(2) LOOP AREA (FT <sup>2</sup> )	(3) AVG. AREA (FT <sup>2</sup> )	(4) DIFF. IN ELEV. (FT <sup>2</sup> )	(5) INCR. VOL. (FT <sup>3</sup> )	(6) ACCUM. VOL. (FT <sup>3</sup> )
413	0				0
		2714	1	2714	
414	5428				2714
		7348	2	14,696	
416	9268				17,410
		5529	2	11,058	
418	1789				28,468
		895	1	1790	
419	0				30,258



**FIGURE 24.34** (a) Portion of a graded area; (b) change in surface areas.

plane areas to obtain a volume. To illustrate this point, Figure 24.35-a shows a portion of a contour grading plan where the graded area is in cut. Figure 24.35-b shows the change in plane areas resulting from the new grading. Each area is bounded by a loop comprised of the existing contour line and the proposed contour line.

In the three-dimensional figure, lines BCDE and B'C'D'E' are the existing contour lines representing the two planes. Lines OP and O'P' are the proposed contour lines



**FIGURE 24.35** Three-dimensional representation of the graded area.

on each plane. The change in surface areas from the existing to the proposed conditions are the loops OPDC and O'P'D'C', which enclose areas  $A_1$  and  $A_2$  respectively. The volume of material is the prism of material between the horizontal planes of  $A_1$  and  $A_2$  and the inclined surfaces of O'P'PO and C'D'DC. This volume,  $V$ , is given as

$$V = \left( \frac{A_1 + A_2}{2} \right) \times H \quad (24.9)$$

The procedure for performing an ETO using the contour method is:

1. Delineate on the grading plan the limits of the cut areas and fill areas. These limits are identified by a closed loop that connects the points where proposed contour lines connect to the existing contour lines, essentially where the cut and fill is zero. These loops are identified as the zero loops and the number of zero loops depends on the grading scheme. All grading within a zero loop is either all cut or all fill. All lines on the drawing representing zero loops should be drawn in the same color. On the plan view, these points are indicated where the proposed contour line ties into the existing contour line or where a cut area changes to a fill area. In Figure 24.36 points A and C are where the proposed contour lines tie to the existing contour lines. Point B is where the cut area changes to a fill area. The zero lines are indicated in the plan view of Figure 24.37.
2. For each contour line within a zero loop, there are other loops comprised of part of an existing contour and a proposed contour. Each of these loops outline the change in area between the existing and proposed conditions for that contour plane. On projects where extensive grading is required, the loops will overlap, as shown in Figure 24.38 and Figure 24.39. The basic procedure for delineating a loop is: (1) on the grading plan, find where the proposed contour line meets the existing contour line; (2) from this point using a colored pencil, trace the proposed contour line to the point where it joins the existing contour again; (3) then trace the existing contour line back to the original point; (4) do this for each contour within the zero loop; (5) identifying individual loops with lines of alternating colors helps when tracing the planimeter around the loop. If a loop cannot be completely closed—that is, the proposed contour does not meet the existing contour within the bounds of the zero line—the zero line is not drawn correctly.
3. The incremental volume is the averaged areas of the two loops multiplied by the difference in elevation between the two contour lines.

After all loops have been outlined, the area of each loop is determined for each cut and fill area. After all loop areas

have been determined and compiled into tabular format (see Tables 24.4 and Table 24.5), the table is completed by following through with the calculations of each column. If one table is used for each cut and fill area, a quick inspection of the tables will identify areas of the site where an adjustment can be made to balance the cut and fill.

**EXAMPLE 4:** Use the contour method to determine the amount of cut and fill for the grading plan of Figure 24.36.

The first step in the procedure is to determine the zero lines. For the grading plan example, some of these points along the zero line where the proposed contour lines tie into the existing contour lines are shown as points a, b, c, d, e and f in Figure 24.37. The line c-g-h-d separates the fill area from the cut area. It is analogous to the line in Figure 24.36 that runs through the center of the graded area to divide the cut and fill areas.

Figure 24.38 shows the change in graded areas for the contour lines for elevations 414 ft, 416 ft, and 418 ft within the zero line for the fill area. For example, beginning at point "J," a loop is made that follows the proposed contour line to point "K," then along the existing contour back to point "J." Similarly, the areas for the loops MQM and the NOPN are measured.

The loop areas for each contour line in the fill areas are measured and entered into a computation table (Col. 2) as shown in Table 24.5. The following calculations complete the table.

COL 3: The average area for the first two consecutive contour lines (e.g., for the 413-ft and 414-ft contour lines), is

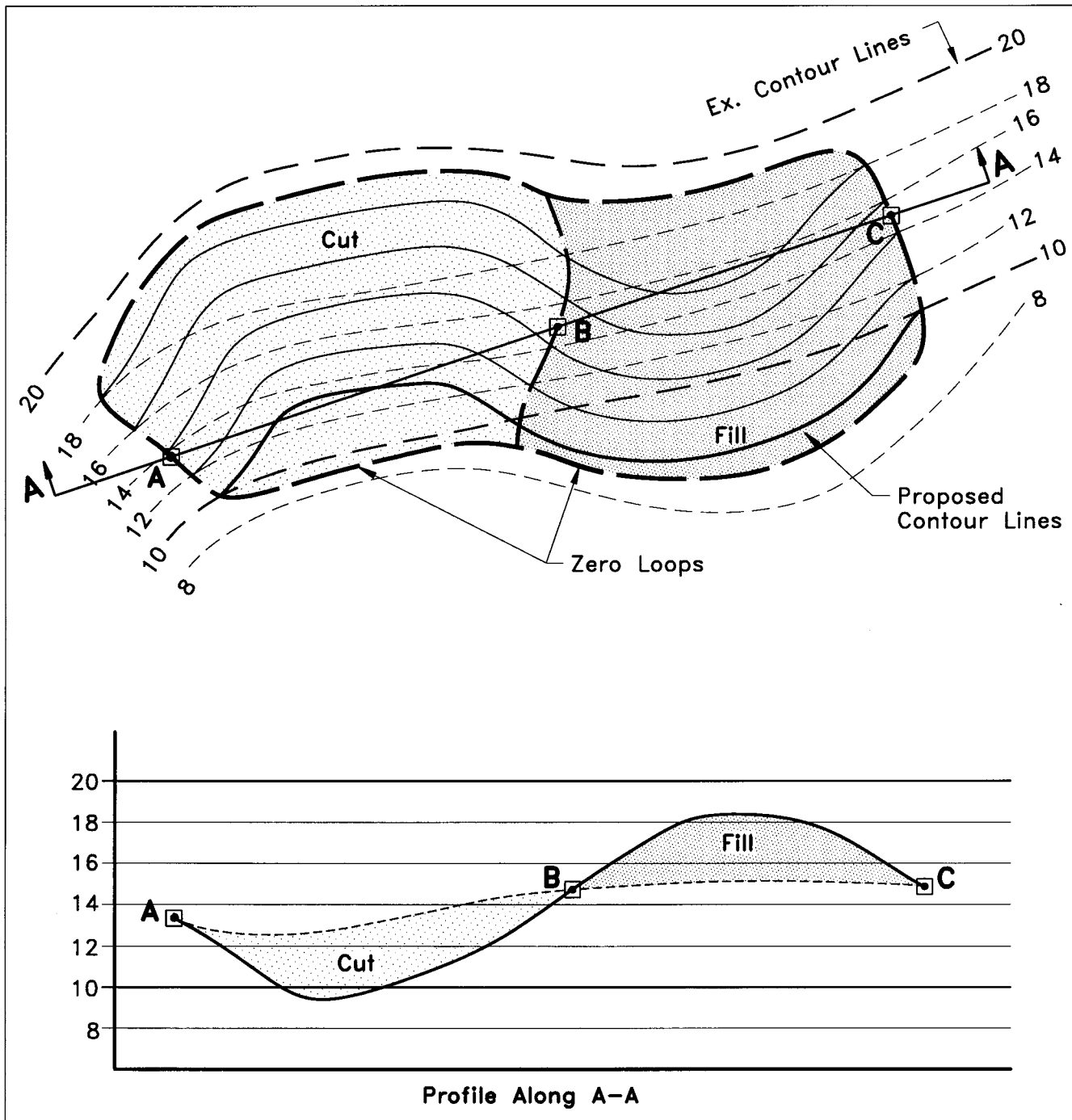
$$\frac{8823 + 11,453}{2} = 10,138 \text{ ft}^2 \quad (24.10)$$

COL 4: The incremental volume is the volume of material cut or filled corresponding to the two consecutive contour lines. It is the averaged area (Col. 3) multiplied by the elevation difference between the two consecutive contour lines.

COL 5: The accumulated volume is the preceding incremental volume added to the current incremental volume.

$$2 \times (416 - 414) \times 10,138 \text{ ft}^2 = 20,276 \text{ ft}^3 \quad (24.11)$$

Similar loop areas are obtained for the contour lines within the zero loop of the two cut areas as shown in Figure 24.40. One detail that must be pointed out involves the cut area around the building. A common error is to trace the building with the planimeter in the wrong direction. The path indicated as abcdef is the correct loop (see discussion on Contour Patterns for Buildings and Retaining Walls). The other loop areas around the building have not been shown for clarity.



**FIGURE 24.36** Schematic diagram showing zero lines.

The total volume of material of cut and fill is

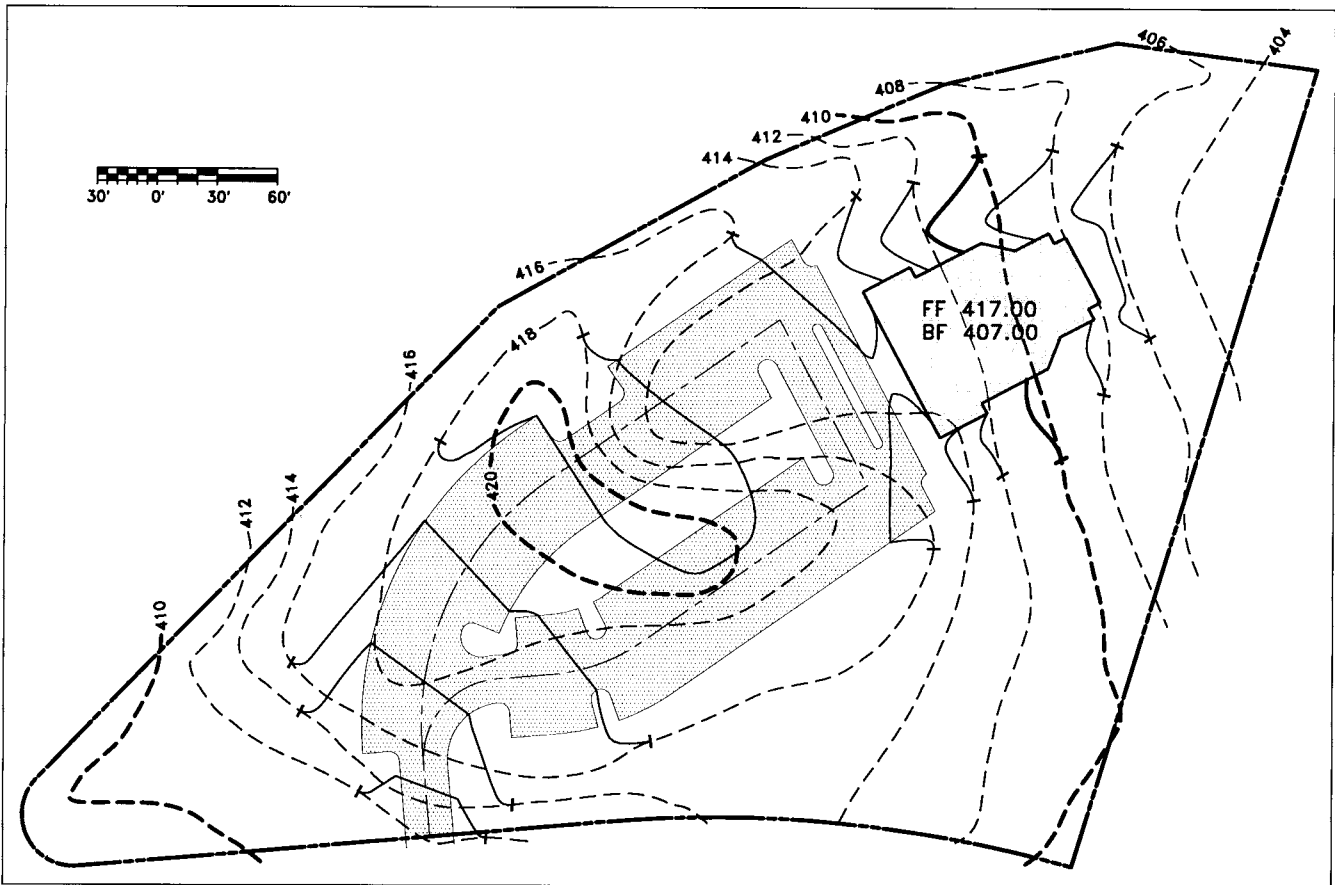
$$\begin{aligned} \text{Total Volume of cut} &= \frac{62,556 + 38,439 \text{ ft}^3}{27 \text{ ft}^3 \text{ per yd}^3} \quad (24.12) \\ &\approx 3740 \text{ yd}^3 \end{aligned}$$

$$\begin{aligned} \text{Total Volume of fill} &= \frac{35,662 \text{ ft}^3}{27 \text{ ft}^3 \text{ per yd}^3} \\ &\approx 1321 \text{ yd}^3 \end{aligned}$$

However, these volumes must be adjusted for topsoil, allowance for concrete and pavement thicknesses, and shrink and swell.

**Equal Planes Method**

Another method for estimating earthwork considers the volume based on averaging incremental depths of cut or fill. Each specific depth extends up or down from the proposed ground to the original ground line. The resulting configu-



**FIGURE 24.37** Grading plan.

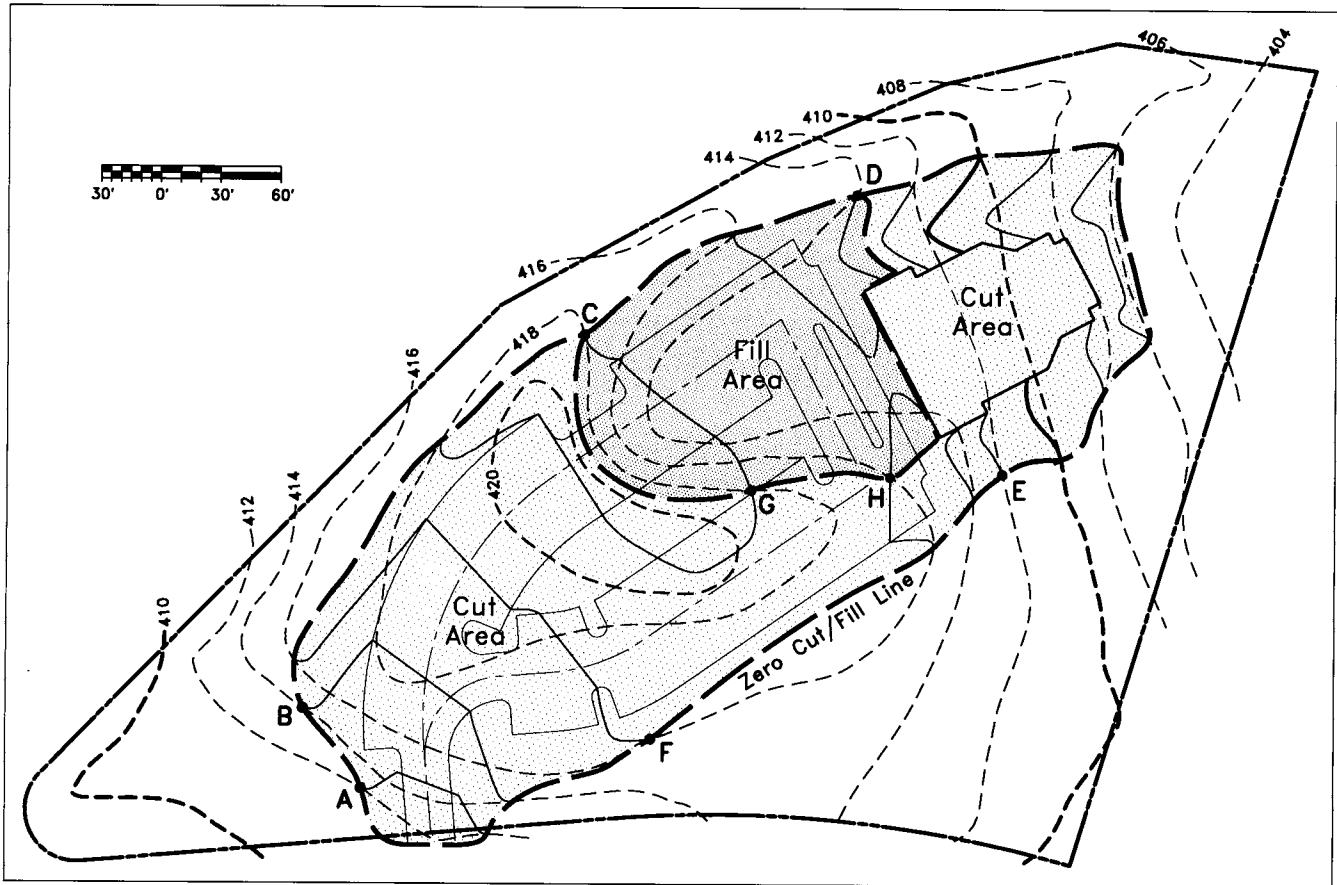
ration is a series of concentric nested rings of uniform depths and variable widths. The height of each ring represents an incremental depth of cut or fill. The width of each ring represents the difference in area between two successive cut or fill contour lines. Whereas the contour and cross-section methods average the cut and fill *areas* on successive planes, the equal planes method averages the depths of cut or fill of successive rings.

To illustrate the incremental volumes of the equal planes method, consider an 8-ft high mound built up from level ground. Imagine that the 8-ft elevation is represented by a ring of a specific radius,  $r_8$ , as shown in Figure 24.41. The outer edge of the ring is formed by the 8-ft contour line. The volume of earth in this initial “ring” is that of a right circular cylinder =  $8(\pi r_8^2)/4$ . Next, consider a ring formed by the 6-ft contour line with radius  $r_6$ , ( $r_6 > r_8$ ), concentric with the 8-ft-high cylinder. The incremental volume in this ring is  $6\pi(d_6^2 - d_8^2)/4$ . Note that the volume between the ground line and the horizontal planes of the successive rings as shown in the triangle PQS of Figure 24.41 is not included in the calculations. This error is reduced by computing the incremental volumes by averaging the depths of fill (or cut) on two successive rings as indicated by triangles TUV and T'U'V on the left side of Figure 24.41.

The height of a ring represents a layer that is a constant depth above (for fill) or below (for cut) the original ground line. The constant distance is measured along the vertical direction from the ground surface. Although the equal planes method is predicated on layers of uniform depth of cut or fill parallel to the ground surface as shown in the profile of the mound (Figure 24.43-a) the projection of these surfaces onto the plan view assumes these surfaces to be parallel to a horizontal plane as shown in Figure 24.42-b.

Figure 24.43 shows a profile view of the mound and the corresponding contours in plan view. The profile shows the underlying surfaces in increments of 2 ft. The first layer under the original ground represents a cut depth equal to 2 ft. In plan view, this layer appears as a loop where the difference in elevation between the existing grade and proposed grade is two ft. Likewise, successive layers of 4, 6, 8, . . . -foot cuts are identified by loops where the difference in elevation between the existing and proposed grades are 4, 6, 8, . . . etc. ft respectively. Each loop is identified on the plan view illustration. A loop represents the inner edge on one shape and the outer edge of the subsequent inner shape. Hence, the difference in area between successive loops is the incremental area used to determine the volume





**FIGURE 24.38** Grading plan showing zero cut/fill lines and cut and fill areas.

of one of the rings. Summation of all incremental volumes is the estimated earthwork volume.

The basic procedure for using the equal planes method on a grading plan is:

1. Delineate all loops representing equal cut or fill layers beginning with the zero loop. This loop identifies the plane where the grades tie out—essentially where the difference between proposed grade and existing grade elevations are zero. There could be more than one cut or fill area on a project. The zero loop isolates these individual cut and fill areas.
2. From each zero loop work inward. Decide the next level for which a plane will be identified. The easiest way is to keep the planes at the same increment as the contour interval. If the contour interval is 2 ft, the next plane would be at a depth/height of 2 ft. Inside the zero loop, locate the points where an existing contour intersects a proposed contour where the elevation difference between the two contours is 2 ft, as indicated by points A and A' in Figure 24.43. A loop that connects these points and all points where the proposed elevation is 2-ft higher/lower than the existing elevation outlines a plane that is 2 ft higher/lower than the zero plane. A loop will cross contours only where the existing contour and proposed contour intersect—unless there is a retaining wall or other structure.
3. Decide the next plane. Again, if the contour interval is 2 ft, the next plane is at a depth/height of 4 ft. Draw a loop that connects all points 4 ft higher/lower than the existing elevation (points B and B' in Figure 24.42). Continue this procedure inside all zero loops until all loops have been drawn for the site. The plan becomes more readable if two colors are used to draw the loops—one color for cut areas and another color for fill areas. Very light shading of alternate rings in the same color enhances the cut and fill areas. Large numbers written on the loops identify the depths/heights of cut/fill areas.
4. For each cut and fill area, a table keeps a record of the individual quantities and the accumulated earthwork, as shown in Table 24.6. The area of each loop is determined and entered into column 2. Column 3 is the difference between the current loop and the preceding loop. Column 4 is the average of the current depth and the preceding depth. Column 5 is the prod-

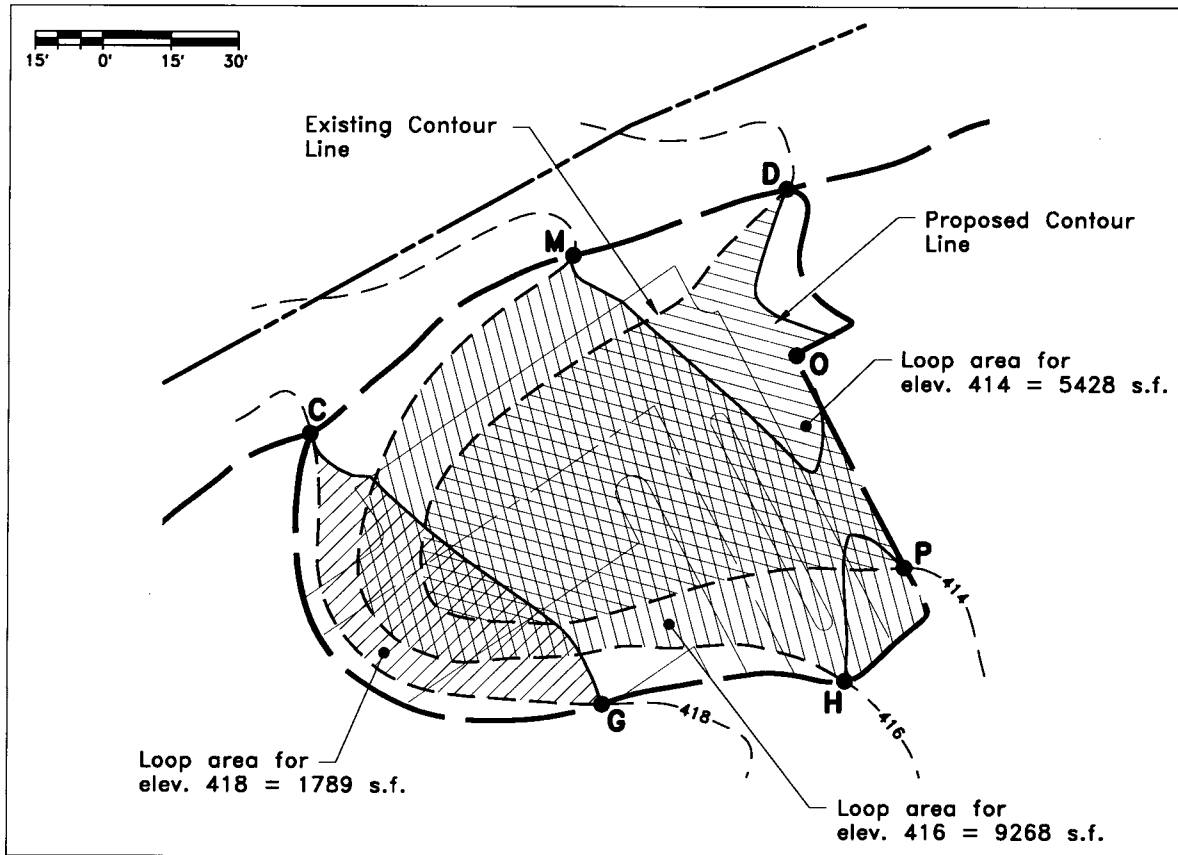


FIGURE 24.39 Contour method: loops for fill area.

uct of column 3 and column 4. Column 6 is the accumulated earthwork of column 5.

**EXAMPLE 5:** Using the same grading plan as the previous example, perform an ETO using the equal planes method. Figure 24.44 shows the grading plan with the zero lines, and the 2-ft, 4-ft and 6-ft depths of cut loops and depths of fill loops.

The area of each loop is determined and entered into column 2 of Table 24.6. Column 3 of this table is the difference in area between two consecutive loops. Column 4 is the averaged depth of cut or fill, and column 5 is the incremental volume = col. 3  $\times$  col. 4.

The main advantage to using the equal planes method is that the final plan view shows the cut and fill areas and how deep these areas are. Any adjustments to the site grades to adjust the ETO can be limited to specific areas.

### Grid Method (Borrow Pit Method)

The grid method for computing earthwork quantities averages the cut or fill depths over a unit area. The product of the averaged depths with the unit area is the net incre-

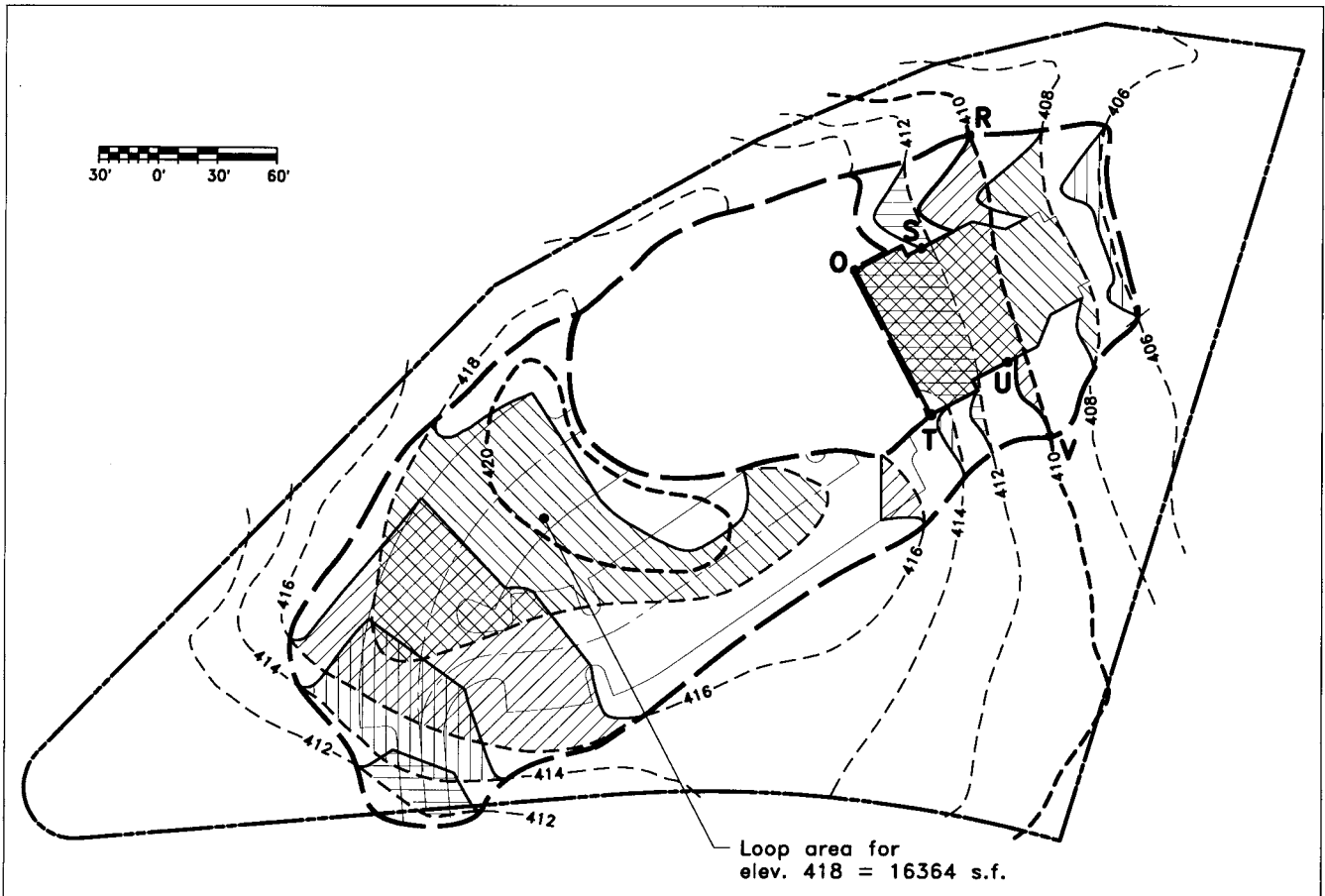
mental volume of cut or fill. A summation of all incremental fill volumes and cut volumes gives the total for the site.

Figure 24.45 shows a proposed ground surface and the existing ground surface for a unit area. (Note that the intersection of the two surfaces establishes the zero cut and fill line.) The depth of cut or fill is written at each corner of the unit area. For this particular unit area, the average cut/fill is zero. That is, the volume of material in the cut area is equal to the volume of material in the fill area.

However, as shown in Figure 24.46, the computations presume that the actual ground surface is a plane representing a linear ground surface. The areas that contribute to the error in the actual volume are shaded. The accuracy of the computed volume is a function of the size of the unit area and how close the representative planes for the existing and proposed ground approximate the true ground surface. As the undulations of the actual ground surface increase in number and deviate from the straight line approximation, the precision of the computed volume decreases. To compensate for the irregular topography, the unit area can be reduced to increase precision. The penalty for this is the increase in number of grids and the computing time to perform the method. In some instances, decreasing the unit area may not be the solution either. On extremely flat sites,

TABLE 24.5 Contour Method: ETO Quantities for Cut Area

(1) ELEV. (FT)	(2) LOOP AREA (FT <sup>2</sup> )	(3) AVG. AREA (FT <sup>2</sup> )	(4) DIFF. IN ELEV. (FT <sup>2</sup> )	(5) INCR. VOL. (FT <sup>3</sup> )	(6) ACCUM. VOL. (FT <sup>3</sup> )
<b>PARKING AREA</b>					
411	0				0
		680	1	680	
412	1360				680
		2829	2	5658	
414	4298				6338
		8468	2	16,936	
416	12,638				23,274
		14,501	2	29,002	
418	16,364				52,276
		8182	1	8182	
419	0				60,458
<b>BUILDING AREA</b>					
405.4	0				0
		324	0.6	194	
406	648				194
		4336	2	8672	
408	8024				8866
		7186	2	14,372	
410	6348				23,238
		5168	2	10,336	
412	3988				33,574
		2055	2	4110	
414	122				37,684
		61	0.4	24	
414.4	0				37,708



**FIGURE 24.40** Contour method: loops for cut area.

decreasing unit area size decreases the precision. For each unit area, an interpolation between contours is necessary to determine the elevations at each corner. Each interpolation calculation has an error associated with it. The propagation of this error contributes to the precision of the volume. In these extreme cases, the grid method may be abandoned in favor of an earthwork method that is more accommodating.

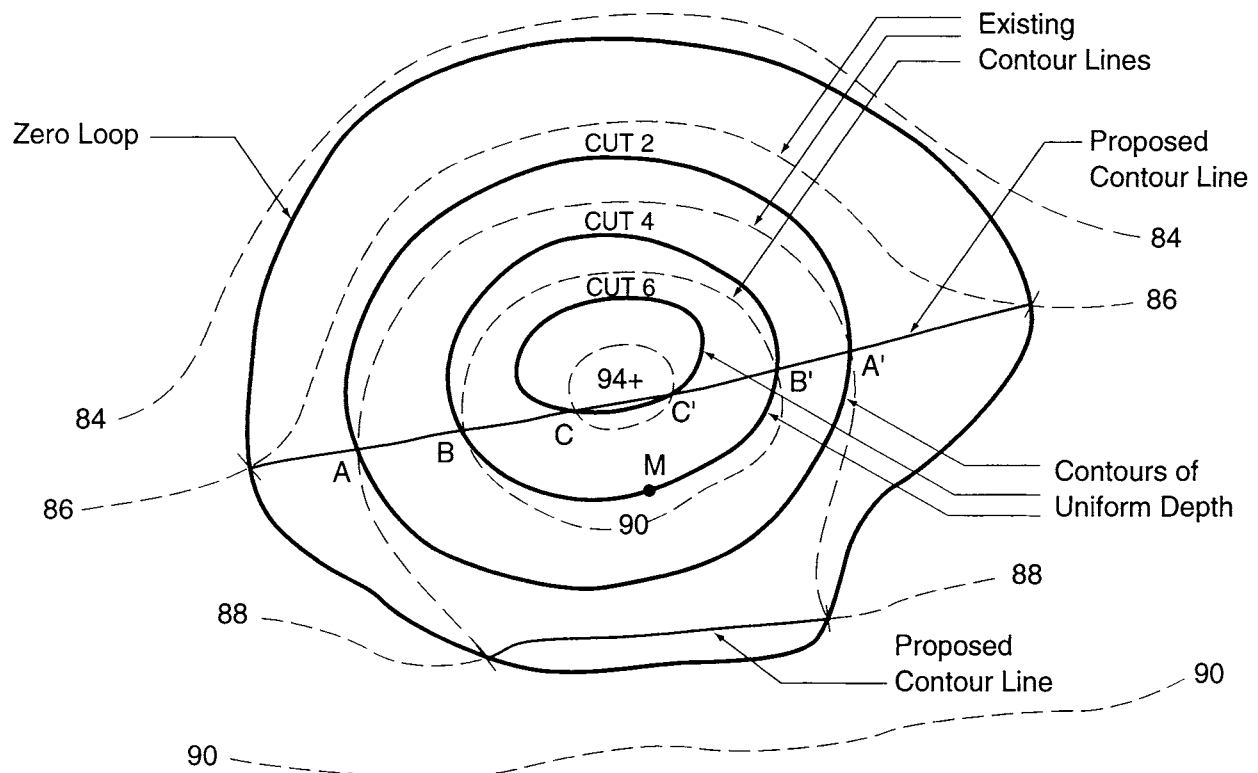
The mechanics of the method are:

1. Obtain the grading plan of the site and create a grid of uniform-sized squares over the graded area. The procedure is facilitated if the grid is created on transparent material and overlain on the grading plan. For reasonably precise earthwork quantities, the grid squares should be 1-in. squares for plans with scales of 1 in. = 100 ft and larger.
2. The grid is overlain on the grading plan. At each corner of a square, a value is entered to identify the amount of cut or fill at that point. A positive value indicates fill and a negative value indicates cut.
3. For each square, the cut/fill values are averaged and written in the center of the square. The incremen-

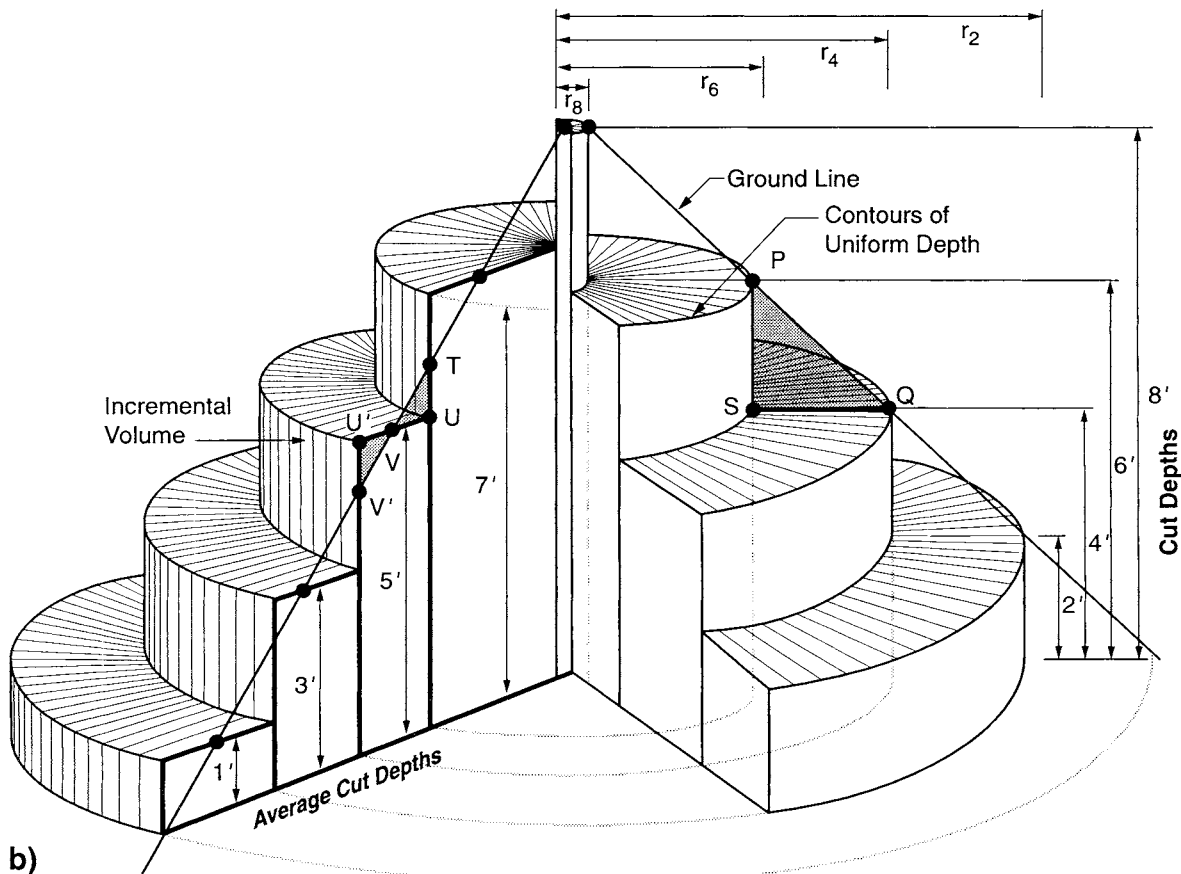
tal volume is the averaged cut/fill depths multiplied by the grid area. Only the corners of grid squares within the grading limits are used to obtain the average cut/fill values.

4. Any corner outside the grading limits is not included in the computational process. The incremental volume is the averaged cut/fill of these grid corners in the graded area multiplied by the fraction of the area of the grid within the grading limits.
5. Summing all averaged values within the grid squares is the net amount of excess/deficient dirt. Summing all positive and negative numbers separately is the amount of cut and fill respectively.

Figure 24.46 shows the grid method for the same grading plan with 1-in. grids. The cut or fill depths are written at the corners of each square (negative values indicate cut, positive values indicate fill). The value written in the center of the grid is the average cut or fill depth for that grid (i.e., the sum of the values at the corners divided by the number of summed values). The volumes of cut or fill for each grid are determined by the product of the averaged cut or fill

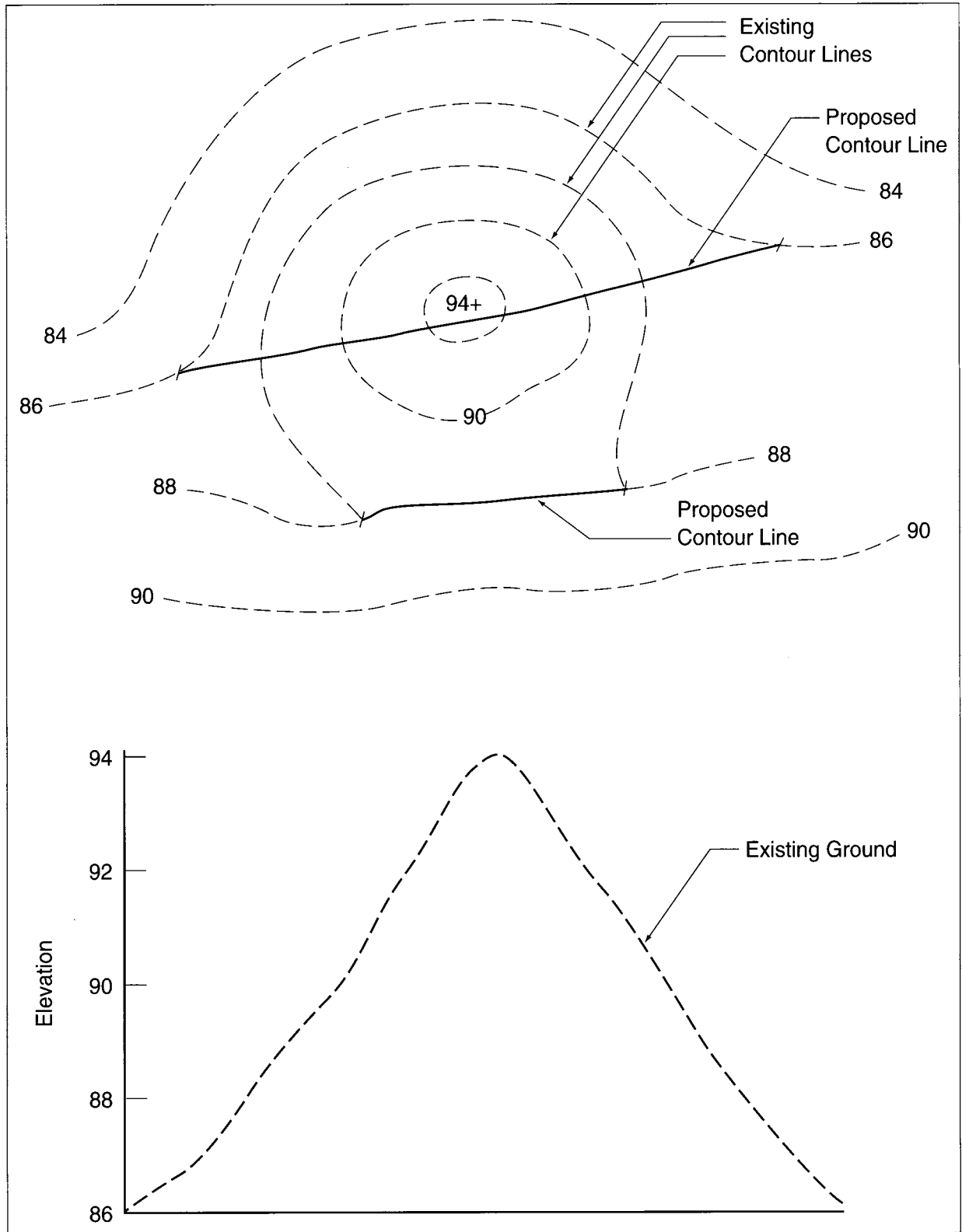


a)



b)

FIGURE 24.41 Contours of uniform depth for the 8-foot-high mound a) plan view b) schematic.



**FIGURE 24.42** Plan and profile view of a mound showing layers of equal depths.

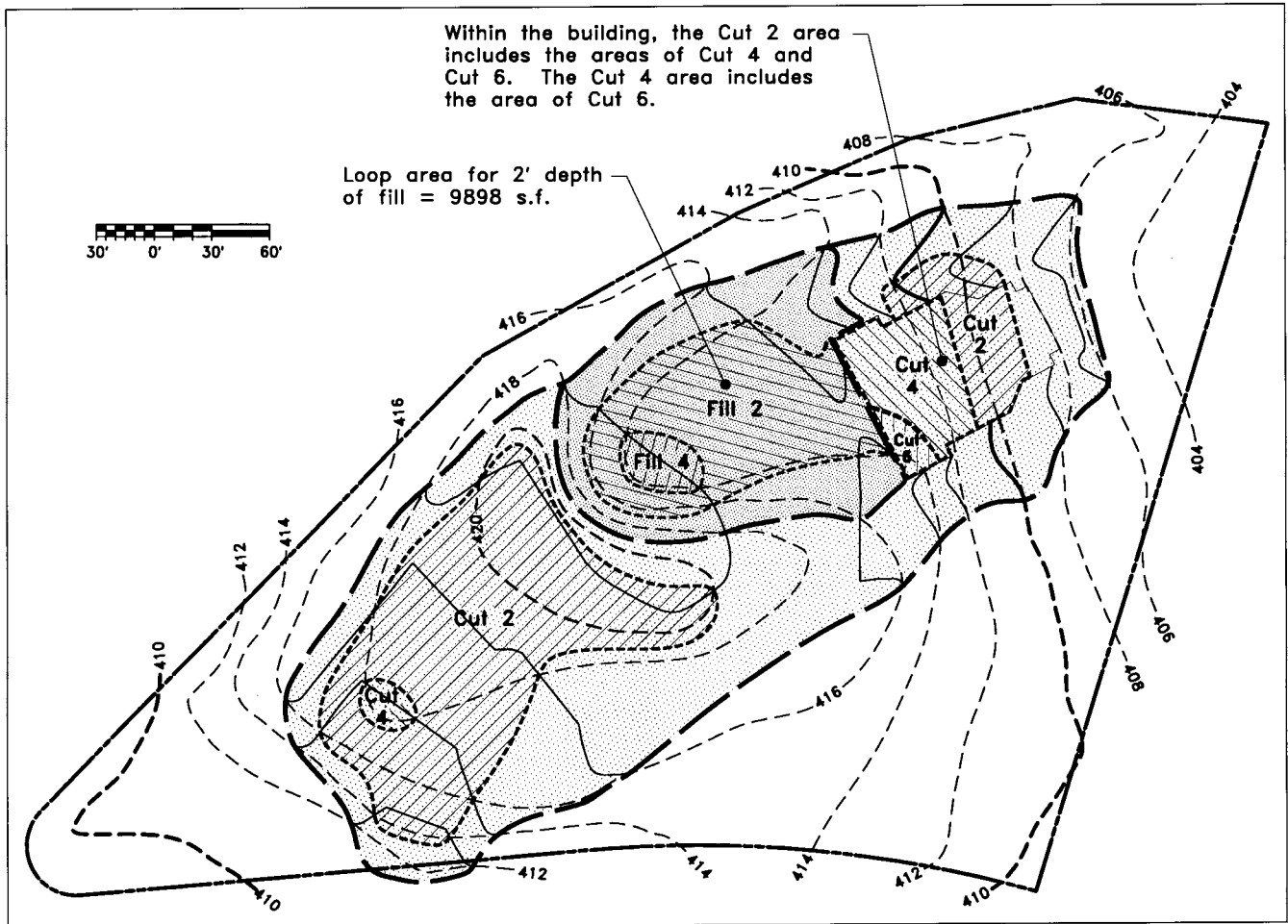


FIGURE 24.43 Cut and fill loops for the equal-planes method.

and the area of each grid. Some squares are not entirely within the grading area. The volume of cut or fill for these squares is found by multiplying the averaged cut or fill depth by the fraction of the area of the grid within the grading area. An “eyeball” estimate of the fractional area should suffice in most cases.

The grid on Figure 24.46 has four rows and eight columns. The incremental volume for each grid is given in Table 24.7.

Summing all of the fractions of the cut values and all of the fill values multiplied by the area of a grid gives the total volumes. The sum of the fractions of the cut and fill are 38.3 and 3.2 respectively. This corresponds to

$$\frac{38.3 \text{ ft} \times (60 \text{ ft} \times 60 \text{ ft})}{27 \text{ ft}^3 \text{ per yd}^3} = 5,100 \text{ yd}^3 \text{ of cut} \quad (24.13)$$

$$\frac{3.2 \text{ ft} \times (60 \text{ ft} \times 60 \text{ ft})}{27 \text{ ft}^3 \text{ per yd}^3} = 430 \text{ yd}^3 \text{ of fill}$$

The unusual low value for the fill volume (430 yd<sup>3</sup>) is

due to the large grid size relative to the small fill area of the plan.

### ADJUSTMENTS TO EARTHWORK QUANTITIES

The ETO methods previously explained calculate the gross quantity of cut and fill. That is, the quantities are computed based on the existing topography and the proposed finished grades. Computing final earthwork quantities includes adjustments to the gross values of cut and fill to account for soil characteristics, topsoil, sub-base allowance, foundations and other items that affect the amount of displaced soil. Since earthwork is a major expense for a project, the adjustments are necessary, considering that the adjustments may affect the computed gross quantity by 10% to 30%. Frequently the client requests the adjustment quantities separate from the gross quantities. These adjustments are shown as separate volumes in a tabulated format. This can be helpful when determining causes for excess or deficient volumes. After all adjustments are made, the results are the net volumes of cut and fill.

**TABLE 24.6 ETO Computations for the Equal-Planes Method**

(1) CUT DEPTH	(2) LOOP AREA (FT <sup>2</sup> )	(3) DIFF. IN AREA (FT <sup>2</sup> )	(4) AVG. DEPTH (FT)	(5) INCR. VOLUME (FT <sup>3</sup> )
0	57,580			
		35,360	1	35,360
2	22,220			
		18,222	3	54,666
4	3998			
		3618	5	18,090
6	380			
		380	6.5	2470
7	0			
				TOTAL = 110,586 ft <sup>3</sup> = 4,096 yd <sup>3</sup>
FILL DEPTH				
0	17,468			
		7570	1	7570
2	9898			
		8930	3	26,790
4	968			
		968	4.2	4066
4.4	0			
				TOTAL = 38,426 ft <sup>3</sup> = 1423 yd <sup>3</sup>

### Topsoil

Before any major grading operations begin, the site is stripped of the topsoil. The topsoil is stockpiled and used for planting and landscaping purposes when the project is nearly complete. In high-density residential projects and many commercial sites, very little area is available for landscaping. The resulting excess topsoil must be either hauled away or disposed on site.

Topsoil depths generally range from 6 to 18 in. Estimates for topsoil depths can be determined from the soils report and the soil boring logs of the site. If the soils report indicates a nearly uniform depth of topsoil, calculate the topsoil volume assuming a constant depth over the graded area. Some sites may have a large variation in topsoil depth

over the site. The engineer decides whether to use a uniform depth or to divide the site into areas of nearly uniform topsoil depth for calculations. The topsoil volume is the estimated depth multiplied by the graded area. In most of the cases the earthwork quantity will not be adversely affected if the topsoil depth is assumed to be uniform over the entire graded area. Another, more time-consuming method is to actually draw the topsoil depth on the cross-section drawings and calculate the topsoil quantity using the average end-area method. This procedure is done when cross sections of the site are available and the topsoil depth is highly variable.

The gross volume of topsoil to be stockpiled is equal to the topsoil stripped from both the cut and fill areas. The



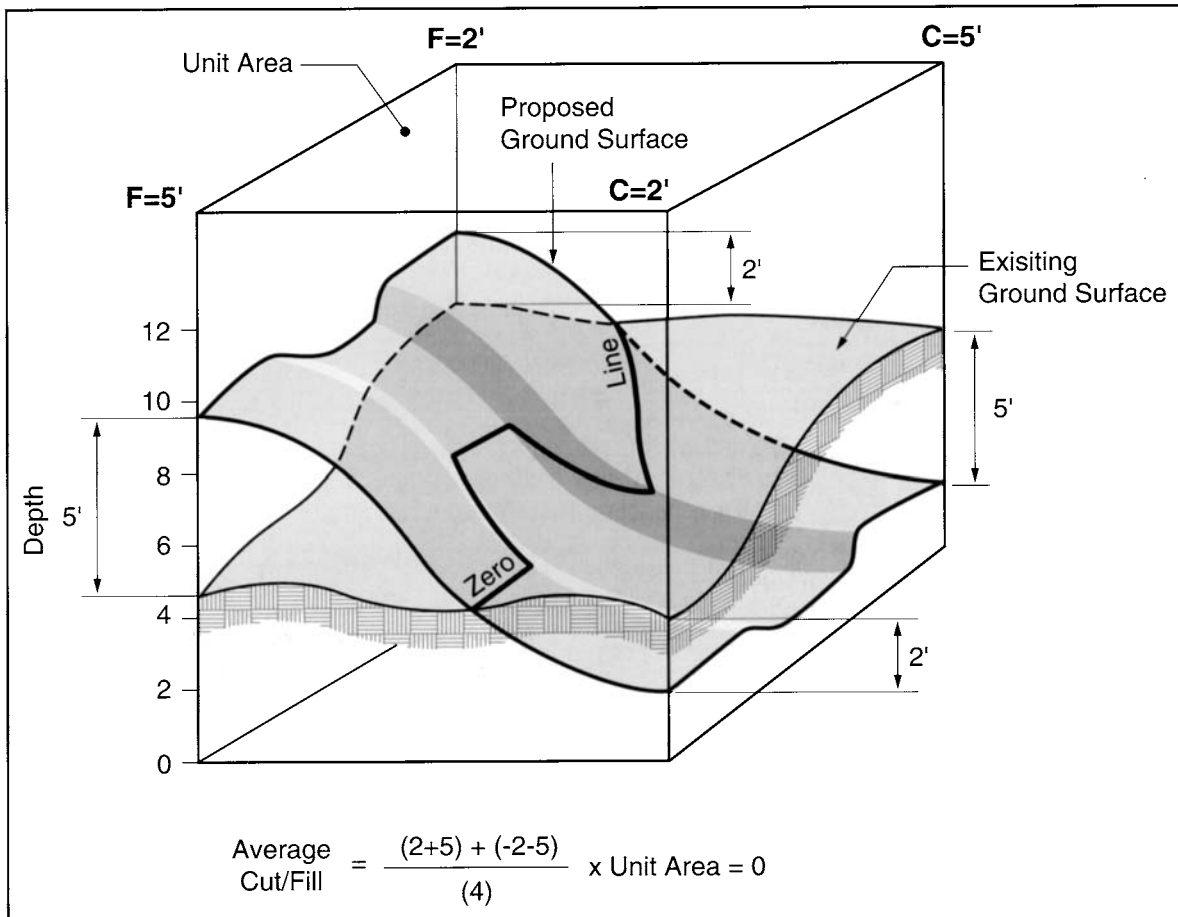


FIGURE 24.44 Ground surfaces for unit area of the grid method.

stripped volume of topsoil from the cut areas is *subtracted* from the gross cut volume.

Conversely, the volume of topsoil stripped in the fill areas is *added* to the gross fill volume. The sum of these two values is the volume of stockpiled topsoil. Topsoil reused in vegetated areas is replacement topsoil. The vegetated areas are determined for both the cut and fill areas of the site. Calculated replacement topsoil volumes in the vegetated areas of the cut areas are added to the gross cut quantity, and the replacement topsoil volumes in fill areas are subtracted from the gross fill quantity. The stockpiled topsoil volume less the replacement topsoil volume is the net topsoil remaining.

Referring back to the grading plan used in the ETO examples, the area of topsoil to be considered is that contained within the zero loops. The area of the zero loop for the cut area is 58,268 ft<sup>2</sup>. An assumed uniform topsoil depth of 6 in equates to approximately 1100 yd<sup>3</sup> of topsoil. Likewise, within the fill area loop of 18,711 ft<sup>2</sup>, the approximate topsoil volume is 350 yd<sup>3</sup>. The vegetated area in the cut area is 26,200 ft<sup>2</sup> and in the fill area is 8000 ft<sup>2</sup>.

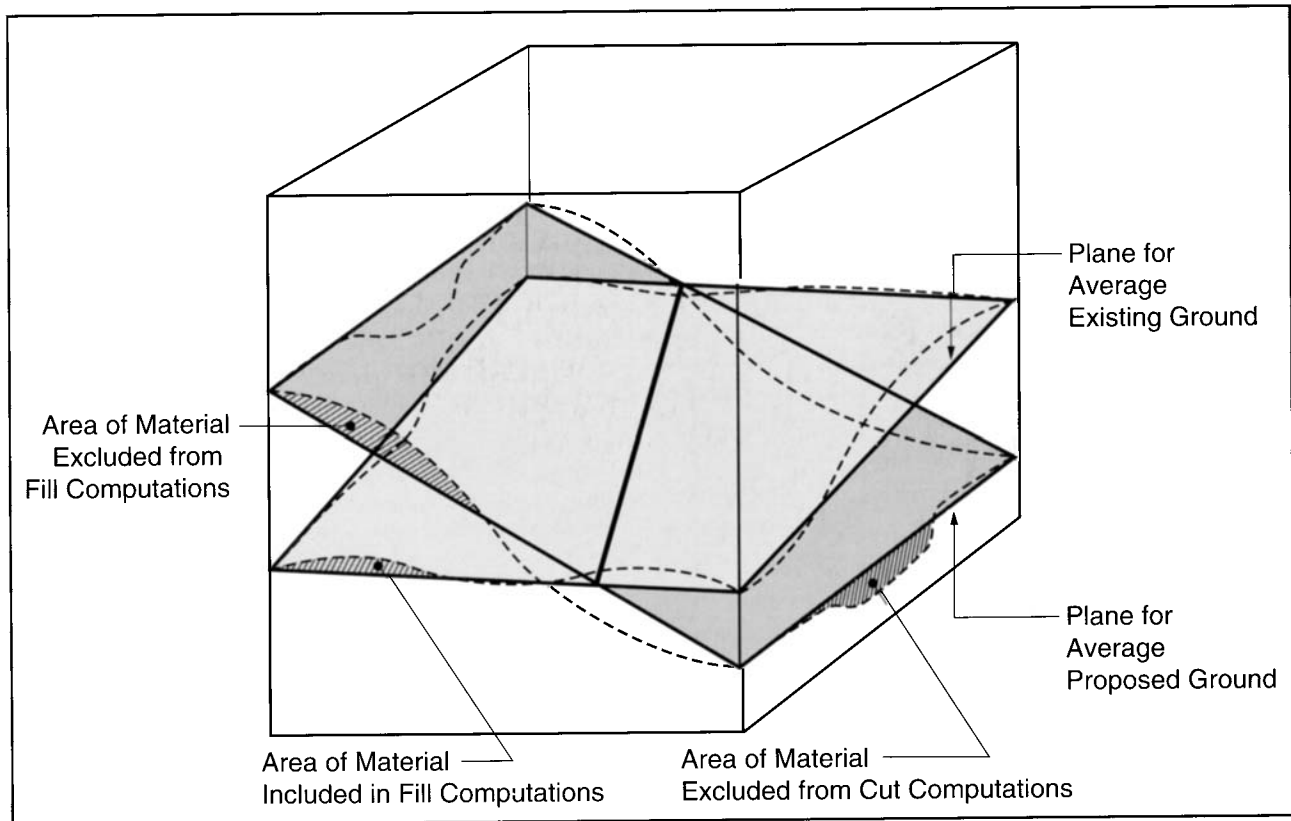
Using a replacement topsoil depth of 14 ± in. results in using all of the stockpiled topsoil tabulated in Table 24.8.

### Subbase and Concrete Pads

Roads, concrete slabs, and building pads consist of different layers of gravel, coarse aggregate, concrete and wearing surface. The total thickness of these layers depends on the bearing capacity of the soil and the type of structure and ranges from eight in to 2 ft in typical situations. In extreme cases the thickness may be double these depths. Concrete pads and the subbase for residential houses has a thickness of 8 to 12 in. Concrete pads and subbase for parking structures and high rise commercial buildings typically are 18 to 24 in. thick. The pavement structure for roads may be 12 to 18 in. thick.

To account for subbase and concrete thickness the earthwork quantity is based on the subgrade<sup>6</sup> elevation rather

<sup>6</sup> In cut areas, the subgrade is the original soil lying below the concrete pad, pavement and gravel layers. In fill areas, the subgrade is the controlled fill placed over the existing ground to bring the area to grade at the subgrade elevation.



**FIGURE 24.45** Average ground surface by linear approximation.

than the finished grade elevation. Therefore, in cut areas the base volume of cut must be increased by the total volume of the subbase, pavement, and concrete. In fill areas, the pavement and subbase volumes are subtracted from the base value of the fill volume.

Following through on the ETO example problem, assume the combined pavement and subbase thickness is 10 in. The area of pavement in the cut area and fill area is determined (by planimeter) to be 25,000 ft<sup>2</sup> and 10,000 ft<sup>2</sup> respectively. The building pad has combined concrete and coarse aggregate layer thickness of 12 in. and a footprint area of 7000 ft<sup>2</sup>. The adjusted earthwork volumes for these items are shown in Table 24.9.

### Undercut

Another adjustment to the ETO must account for the inadequate bearing capacity of the underlying soil or other bad soil conditions (e.g., high shrink/swell characteristics) or underlying rock layers. Although a thorough subsurface investigation performed during early planning stages usually alerts the design team to these undesirable conditions, small pockets of poor soil conditions and rock layers may not become evident until excavation exposes them during construction. Nonetheless, the removal (undercut) and replacement of this unsuitable material must be accounted

for in the earthwork quantities. If such conditions cover wide areas and extend to great depths, the removal and replacement with acceptable material becomes costly.

The undercut quantity is added to the cut volume as well as the fill volume.

### Utilities

Accounting for soil volume for relatively narrow shallow trenches is not typically done. For example, the volume of a trench 6 ft deep and 3 ft wide is equal to 0.67 yd<sup>3</sup> per linear foot. The volume of a 15-in. conduit with 6 in. of bedding material is 0.10 yd<sup>3</sup> per linear foot. If a 15% shrinkage factor is assumed, then the additional amount of soil required for backfill is compensated for by the volume of conduit and bedding.

However, when the sizes of the trenches become larger or deeper than the shallow example described above, the amount of trench backfill should be accounted for. The backfill volume is the volume of the trench less the volume of conduit and bedding plus allowance for shrinkage.

### Shrinkage

Soil volume increases when it is displaced from its natural state due to the increase in the amount of voids. Relative to soil in a natural state, soil used as compacted fill has a

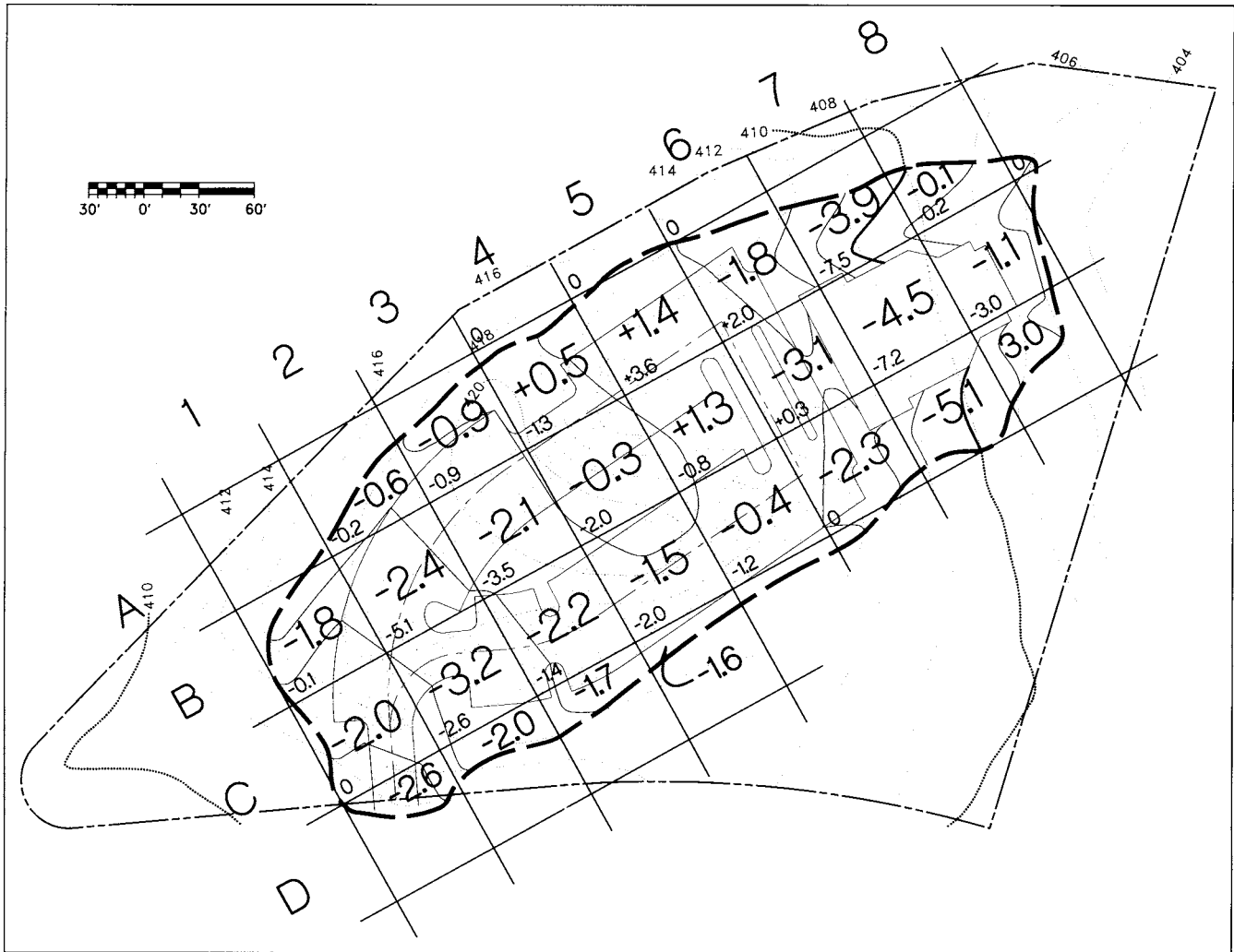


FIGURE 24.46 Grid method for ETO.

higher in-place density in most cases. If the same dirt excavated from a 1-ft<sup>3</sup> hole is placed back into the hole without any compaction efforts, there will be excess dirt. Later a depression appears in the hole area due to natural settlement. If the dirt is placed back into the hole at a density higher than its natural state, there will not be enough dirt to fill the hole. With the added compaction, the amount of voids are less than the amount of voids of the naturally consolidated soil. Hence, the reason for the deficiency in excavated soil volume. This apparent decrease in volume of excavated soil is referred to as shrinkage. The ratio of the remaining volume of the excavated hole after the original dirt has been replaced (with compaction) to the total volume of the hole is the shrinkage factor. The amount of shrinkage can be measured relative to the volume of excavated material or relative to the volume of required fill.

To illustrate this last statement, consider a soil with a density of 90 lbs per ft<sup>3</sup> with 21.2% water content in its natural state. Laboratory test results show that this same

soil has a maximum density of 110 lbs per ft<sup>3</sup> at optimum moisture content (OMC).

(a) If a site requires 10,000 yd<sup>3</sup> of fill at maximum density, how much borrow material should be excavated?

To compute soil volumes in going from borrow to fill or fill to borrow, recognize that density is inversely proportional to volume,  $D \propto (1/V)$ . The relative ratios of the borrow and fill densities and volumes can be expressed as

$$\frac{D_b}{D_f} = \frac{V_f}{V_b} \tag{24.14}$$

where the *b* and *f* subscripts refer to borrow or fill. Therefore, the required borrow volume is

$$\frac{90 \text{ pcf}}{110 \text{ pcf}} = \frac{10,000 \text{ yd}^3}{V_b} \text{ or } V_b \approx 12,200 \text{ yd}^3 \tag{24.15}$$

(b) How much volume will 10,000 yd<sup>3</sup> of borrow material occupy if placed at maximum density?

**TABLE 24.7 Incremental Cuts and Fills for ETO Grid Method Example**

GRID (ROW × COLUMN)	AVG. CUT OR FILL DEPTH	FRACTION OF GRID	FRACTION OF CUT OR FILL	GRID (ROW × COLUMN)	AVG. CUT OR FILL DEPTH	FRACTION OF GRID	FRACTION OF CUT OR FILL
(A,1)	0	0	0	(A,5)	+1.4	1	+1.4
(B,1)	-1.8	0.8	-1.4	(B,5)	+1.3	1	+1.3
(C,1)	-2.0	1	-2.0	(C,5)	-0.4	1	-0.4
(D,1)	-2.6	0.5	-1.3	(A,6)	-1.8	0.8	-1.4
(A,2)	-0.6	0.4	-0.2	(B,6)	-3.1	1	-3.1
(B,2)	-2.4	1	-2.4	(C,6)	-2.3	0.9	-2.1
(C,2)	-3.2	1	-3.2	(A,7)	-3.9	0.6	-2.3
(D,2)	-2.0	0.4	-0.8	(B,7)	-4.5	1	-4.5
(A,3)	-0.9	0.7	-0.6	(C,7)	-5.1	0.7	-3.6
(B,3)	-2.1	1	-2.1	(A,8)	-0.1	0.3	-0.3
(C,3)	-2.2	1	-2.2	(B,8)	-1.1	0.6	-0.7
(D,3)	-1.7	0.4	-0.7	(C,8)	-3.0	0.3	-0.9
(A,4)	+0.5	0.9	+0.5				
(B,4)	-0.3	1	-0.3				Σ Cut = 38.3 ft
(C,4)	-1.5	1	-1.5				Σ Fill = 3.2 ft
(D,4)	-1.6	0.2	-0.3				

**TABLE 24.8 ETO Values Adjusted for Topsoil Depth**

ADJUSTMENT	CUT VOL. (YD <sup>3</sup> )	FILL VOL. (YD <sup>3</sup> )
Gross	4096	1423
Strip Topsoil (6")	-1100	+ 350
Replacement Topsoil (14")	+1130	- 345
ADJUSTED VOLUME =	4126	1428

**TABLE 24.9 ETO Values Adjusted for Subbase, Pavement and Concrete Thickness**

ADJUSTMENT	CUT VOL. (YD <sup>3</sup> )	FILL VOL. (YD <sup>3</sup> )
Gross	4096	1423
Strip Topsoil (6") Replacement	-1100	+ 350
Topsoil (14")	+1130	- 345
Building Pad (12")	+ 260	- 0
Pavement (10")	+ 770	- 310
ADJUSTED VOLUME =	5156	1118

TABLE 24.10 Tabulated ETO Values

ADJUSTMENT	CUT VOL. (YD <sup>3</sup> )	FILL VOL. (YD <sup>3</sup> )
Gross	4096	1423
Strip Topsoil (6") Replacement	1100	+ 350
Topsoil (14")	+1130	- 345
Building Pad (12")	+ 260	- 0
Pavement (10")	+ 770	- 310
Undercut	+ 0	+ 0
Large Conduits & Structures	+ 0	—
SUBTOTAL =	5156	1118
Shrinkage (20%)		+ 224
NET FILL VOL. =		1342
NET CUT VOL. =	3814	

$$\frac{90 \text{ pcf}}{110 \text{ pcf}} = \frac{V_f}{10,000 \text{ yd}^3} \text{ or } V_f \approx 8200 \text{ yd}^3 \quad (24.16)$$

(c) What is the relative change in volume in parts a and b?

Part a:

$$\frac{12,200 \text{ yd}^3 - 10,000 \text{ yd}^3}{10,000 \text{ yd}^3} = 22\% \quad (24.17)$$

Part b:

$$\frac{12,200 \text{ yd}^3 - 10,000 \text{ yd}^3}{12,200 \text{ yd}^3} = 18\%$$

Part (c) shows that relative to the required fill volume, 22% more borrow material is required, and relative to a given borrow volume, the material reduces only 18% in volume when placed at maximum density. Notice that the borrow volume reduces by the same amount in both cases (i.e.  $(12,200 - 10,000)/12,200 \approx 18\%$  and  $(10,000 - 8200)/10,000 \approx 18\%$ ). The point here is to recognize that careful attention is warranted when specifying volumes for fill and excavation projects. Payment for fill dirt is typically based on the amount required to fill the hole (as shown in part a above) and not the volume of the hole itself. However, in most cases, the cost of doing the work is based on the amount of soil excavated. Since the density of excavated soil is less than its natural-state density, allowances must be made for computing haul quantities.

Typical shrinkage factors may range from 10% to 30% depending on the type of soil, the amount of compaction

and the amount of losses expected in hauling. In the previous example, the natural soil density is known. In most cases, the natural soil density is unknown and the engineer must use judgment in estimating the shrinkage to make an adjustment. This adjustment is typically made by increasing the amount of fill that is reusable. This accounts for the increased amount of soil required as a result of compaction.

Referring back to the ETO example, the assumed shrinkage factor is 20%. The measured fill volume is 1095 yd<sup>3</sup>. This will require  $1.2(1095) = 1314 \text{ yd}^3$ . Assuming the cut volume is adequate for use as compacted fill, the resulting excess is  $4560 - 1314 = 3246 \text{ yd}^3$ . This can be hauled away to be used as compacted fill elsewhere. The remaining  $3246 \text{ yd}^3$  of excavated material will convert to  $0.8(3246) \approx 2600 \text{ yd}^3$  of fill material.

Table 24.10 shows the final tabulation to account for earthwork adjustments.

### SUMMARY

Grading and siting buildings for a project requires the site designer to consider many issues when developing a grading plan that is cost-effective to the builder and appealing to the consumer. Issues such as saving trees, optimizing grading, streetscape and roof lines, house type, drainage, zoning ordinances and the rest are all integrated into the design. Although each item may be addressed separately, the important concern is that they must be analyzed collectively after the design is complete. Many times the designer will adjust a site to address a single issue. In doing so, several other issues undoubtedly will be affected—usually adversely. The designer must collectively reanalyze

the site to be sure that the small, single adjustment does not cause a conflict with any other item.

## REFERENCES

- Brown, Thomas L., *Site Engineering for Developers and Builders*, National Association of Home Builders, Washington, D.C., 1988.
- Department of Housing and Urban Development, Final Fair Housing Accessibility Guidelines, *Federal Register*, Vol. 56, No. 44; 3/6/91.
- Harris, C., and N. Dines, *Times Saver Standards for Landscape Architecture*, McGraw-Hill, New York, 1988.
- Harrison, Henry S., *Houses: The Illustrated Guide To Construction Design and Systems*, Realtors National Marketing Institute, Chicago, 1973.
- Landphair, Harlow C., and Fred Klatt, Jr., *Landscape Architecture Construction*, Elsevier, New York, 1979.
- Nelischer, Maurice, ed., *Handbook of Landscape Architectural Construction*, Vol. I, 2e., Landscape Architecture Foundation, 1985.
- Robinette, Gary O., and Charles McClennon, *Landscape Planning for Energy Conservation*, Van Nostrand Reinhold, New York, 1983.
- Strom, Steven, and Kurt Nathan, *Site Engineering For Landscape Architects*, Van Nostrand Reinhold, New York, 1985.
- Untermann, Richard K., *Principles and Practices of Grading, Drainage and Road Alignment*, Reston Publishing, Reston, Va., 1978.
- U.S. Architectural & Transportation Barriers Compliance Board, Accessibility Guidelines for Buildings and Facilities, *Federal Register*, Vol. 56, No. 144; 7/26/91.



## CHAPTER 25

# WASTEWATER COLLECTION

*Ernest Jennelle, PhD., P.E.*

### OVERVIEW

Proper management of wastewater is one of the most important factors for insuring the general health of a community and its surface water quality. Much has been written about the early sewer systems, such as the underground drains installed in ancient Rome, but these early sewer systems were generally storm drains. In most urban areas, it was unlawful to discharge sanitary sewage into these drains prior to the mid 1800s. Thus, the history of sanitary sewers only covers the period from the mid-1880s to the present.

Major construction of sewer systems in many communities occurred during the 1930s period of economic depression as a part of the Federal Government Works Progress Administration (WPA) program. Many smaller communities obtained their first central sewer system and sewage treatment facilities as a part of this program. This was the first significant period of sanitary sewer construction in this country. This period was also noted for the significant increase in the popularity of indoor plumbing.

World War II was a period of both urban and industrial growth throughout the United States. At the end of the war, many segments of surface streams were excessively polluted. Two of the reasons for the degradation of so many segments of streams were the substantial increase in the discharge of untreated industrial wastes and pollution from untreated overflow from combined sewers. Many older urban areas installed storm drains prior to the time of widespread adoption of indoor plumbing. When indoor plumbing became fashionable, the building sewers were connected into the existing storm drains. These "combined" sewer systems generally drained directly, without treatment of the flow, to the nearest watercourse. Also, the wastewater

collected in many sanitary sewer systems received little or no treatment prior to its discharge into surface waters at that time.

The U.S. Public Health Service was the federal agency responsible for water quality related matters prior to establishment of the Federal Water Quality Administration in 1968. This agency later became the Environmental Protection Agency. It should also be noted that at this time federal agencies had little control over intrastate matters related to the environment. The U.S. Public Health Service, private organizations and universities in the United States, Great Britain and Europe were conducting research and studies related to stream water quality and water borne diseases as early as the last half of the nineteenth century. One of the most significant contributions to water quality knowledge in the first half of this century is the work of two U.S. Public Health Service officers, which was published in 1925, on the analysis of the oxygen balance in streams. This classical work of Streeter and Phelps remains the basis for modeling stream quality today.

It was only after World War II that the individual states began to establish agencies and enforce regulations controlling surface water pollution. The Water Quality Act of 1972 (P.L. 92-500) was the first legislation giving the Federal Government authority to set requirements and enforce standards on a national basis. This legislation, together with the federal funding for water quality programs and facilities, has changed the quality of sewer system design and construction.

A primary limitation to the effectiveness of any water pollution control program prior to the 1970s was the poor quality of sewage collection systems. Most community sewer systems were in a state of disrepair because of the



lack of suitable materials for use in construction, poor construction practices and poor maintenance. From 1950 to 1970, while significant progress was made in the construction of sewage treatment facilities, most sewer systems, pumping stations and treatment facilities were still subject to flooding, overflow and bypassing of flow particularly during wet periods.

Segments of some sewer systems, primarily in older larger communities, are still combined sewers. Thus, during wet periods, overflow and bypassing may occur with these systems. Federal and state programs are attempting to correct these conditions, as funds become available.

P.L. 92-500 provided for establishing regulations and timetables for eliminating pollution on a uniform national basis. To accomplish these new goals, it became necessary to design proper reliability into new sewer systems and to rehabilitate existing sewers. This meant using suitable construction materials and methods. The engineers of today have suitable materials and technology to design and construct sewer systems that will eliminate problems with both infiltration and exfiltration. New sewers, when designed and constructed to today's technological standards, should provide reliable service for many years.

The design engineer now has several choices of suitable pipe material for use in sewer construction that will provide long service and near 100% exclusion of both infiltration and exfiltration. Proven design procedures are available to the engineer for sizing and routing of sewers to ensure that the completed project will properly serve the contributing area. The design engineer must be thoroughly familiar with federal, state and local laws and regulations before proceeding with a design.

This chapter presents material that, when properly applied, will result in a reliable and effective sewer system design.

### Glossary of Terms

This glossary includes common terms used in regard to sewer systems. Many other terms are defined in the text as they are used.

**Sewer.** A conduit for conveying wastewater. Sewers may be designated as sanitary, industrial or storm depending on the type of flow being conveyed. Sanitary sewers convey sanitary sewage that originates in a community. All the wastewater flow from a community including the flow from residential areas, from commercial areas and from normal community industry such as laundries, restaurants, and service stations is designated as domestic wastewater. Sanitary sewers are generally closed circular conduits. They are designed such that the hydraulic gradient is below the inside crown of the conduit (see section on the hydraulics of sewers).

**Sewage (Wastewater).** The liquid conveyed by a sewer. The water carriage system used to convey waste from the community is called sewage. The term "wastewater" is now the more familiar term for this flow. Sewage may be des-

ignated as domestic, industrial or storm to indicate the source. Domestic sewage is also called sanitary sewage or wastewater.

**House (building) Spur or Lateral.** The conduit between the building foundation and the sewer owned and maintained by the local utility. The uppermost segments of a sewer system may also be designated as a lateral (where no other public sewer discharges into the segment). It is common practice for the local utilities to maintain the building lateral segment between the public sewer and the property line.

**Submain, Main and Trunk Sewers.** These terms are used subjectively in defining the type of sewer. Trunk sewers are generally the main conduits receiving the flow from a large area and would include the flow from several submain and main sewers.

**Interceptor Sewers.** A conduit that intercepts other sewers, such as the sewer routed along a stream and intercepting sewers in the collecting system.

**Outfall Sewer.** The conduit between the treatment facility and the receiving body of water. The trunk sewer between the collection sewers and the treatment facility is also often designated as an outfall sewer.

**Combined Sewers.** A combined sewer refers to the older systems where the initial drainage was storm drainage. Before the mid-1900s it was not uncommon for sanitary sewers to be connected to existing storm sewers. Thus, the sewers conveyed combined sanitary and storm wastewater and were designated as combined sewers. These combined sewers generally discharged directly to a nearby body of water without treatment. Since all wastewater is now treated prior to discharge, combined sewers are no longer constructed. Separate sewers are being constructed for collecting sanitary sewage in localities where combined sewers exist, as funds become available for the design and construction.

**Manhole.** An opening to a sewer for inspection and maintenance. Manholes are generally placed at all sewer intersections, at all changes in alignment and grade of a sewer system and at least every 400 ft. The manhole structure usually consists of a vertical 4-ft in diameter barrel section, a top cone section reducing the circular opening to two ft, and a cast iron frame and cover having a two-ft opening.

**Infiltration and Inflow.** Infiltration is ground water that enters the sewer system. An infiltration problem results from broken and cracked sewers, from poor pipe material and poor joints and from poorly constructed manholes. Additionally, concrete and clay pipes are semi-pervious. So-called "barrel weeping" is a principal source of infiltration in sewer systems employing these materials. Local design standards generally limit infiltration to less than 100 gallons per day per inch-mile of sewer (length of sewer line in miles multiplied by the diameter in inches). However, the engineer now has several excellent sewer pipe materials available that should provide a completely waterproof line when properly installed.

Inflow is surface water that enters the sewer system. The principal sources of inflow include inundation of manholes and unauthorized connections. Unauthorized connections can include building downspouts, area drains and other connections of storm sewer to the sewer system. Ideally, all storm or surface water should be excluded from sanitary sewers. Sewage collection and treatment capacity is a valuable resource to a community, and the local utility should no longer permit this capacity to be consumed by infiltration and inflow.

**Exfiltration.** The seepage of sewage from the sewer system into the surrounding soil. Exfiltration may result from the same physical conditions as listed above for infiltration. Infiltration may occur when the water table is above the sewer elevation, and exfiltration may occur when the water table is below the elevation of the sewer.

**Wastewater Treatment.** Any controlled process for removing the pollutional constituents from wastewater and the proper management of the residuals removed. The type of process employed depends on the characteristic of the pollutants to be removed and the treatment level required.

**Sewerage.** Sewage works, including the facilities for the collection and conveying of sewage.

### Parties Involved in the Planning and Design of Sewer Systems

The design of any sewer system always involves several parties. Input from all affected parties should be organized early in the design. The design engineer must be familiar with the local, state and federal laws and regulations as they apply to the proposed design. The local government and often a state agency will have to approve the design and issue a permit for construction. Also, it is important to determine if the downstream segments of the sewer system and the treatment facilities that will receive the flow have adequate capacity. If not, a treatment plant and disposal of the treated effluent must be a part of the planning phase. The general responsibilities of the principal parties are discussed below.

Sewer system planning and design will normally be a part of the overall project plans. The overall project plans should show all infrastructure, and the construction contract should provide for coordination of all utilities to be constructed.

**Owner.** The owner is the party that initiates the project. The owner may be a local government or utility or it may be a landowner or developer. Regulations vary, with some utilities taking the responsibility for the design and construction of all sewers constructed within their jurisdiction, while the landowner or developer pays the cost of the extensions. Other utilities permit the landowner or developer to design and construct the sewers, after which they are dedicated to the utility. In both instances, the local government or utility may have design standards with which the engineer must comply.

The owner should arrange proper financing for the project early in the planning stage. In many instances, the private owner will commission an attorney to provide legal services related to zoning and other jurisdictional approvals prior to commissioning the engineering design. When the local government or utility is the owner, funds will have been budgeted for the phase of the work to be completed prior to commissioning the design.

**Legal Counsel.** As noted above, legal counsel is generally needed by the private owner early in the planning stage of a project. Legal counsel provides direction to the project with respect to property rights, zoning requirements, etc. In most urban areas, attorneys can be found who specialize in legal requirements and procedures for land development.

**Engineer.** The engineering firm offering services in sewer design will generally be staffed to provide complete professional services, including both surveying and engineering. Thus, the term “engineer” as used herein refers to an organization that offers consulting services capable of handling the project from the beginning through the completion of construction and acceptance by the owner.

**Regulatory Agencies.** The engineer must be knowledgeable in the laws and regulations related to the project. It is common for sewerage projects to be under both state and local regulations. One or both levels of government may review the plans and specifications and issue a permit for construction. Generally the plan reviewing agencies will inspect the completed project before it is placed in operation. Most reviewing agencies also have minimum design standards that must be followed. These standards represent good minimum design practice and also insure uniformity throughout the sewer system, which is important to the maintenance staff. It is critical that the engineer be well informed on all local design standards and procedures.

**Contractor.** The engineer prepares the contract documents, plans and specifications. The forms for the contract documents and the general specifications generally come from standards adopted by the local utility or from standard sources, such as the National Society of Professional Engineers and the American Society of Civil Engineers. The engineer prepares the technical sections of the specifications specifically for the project. The engineer generally assists the owner in advertising the project for construction, in evaluation of bids, in selecting the contractor and in the award of a contract for construction. The engineer generally represents the owner in relations with the contractor. The contractor is responsible for constructing the project in accordance with the plans and specifications.

## DESIGN OF SEWER SYSTEMS

### Characteristics of Sewage

Sewage is a water transport means of conveying wastes from the source. Domestic sewage conveys wastes from dwellings and the normal business, commercial and industrial activities associated with any community, such as restaurants,

laundries and service stations. Sewage also contains the dissolved inert chemicals from the water supply as well as dissolved and suspended organic matter from the sources. The primary source of the material in sewage is the bathroom, kitchen and laundry. Sewage also contains very high concentrations of bacteria and viruses. Sewage should always be considered as being a source of enteric pathogens.

Since sewage behaves hydraulically as water, the design engineer is only concerned with the composition of sewage when it contains substances that may be damaging to the sewer system or maintenance personnel. Principal substances of concern include those that are corrosive, present potential fire or explosive hazards, emit toxic fumes or interfere with downstream wastewater treatment. Normal domestic sewage does not contain hazardous constituents other than pathogens. However, poor design or construction can create hazards. The accumulation of solids can result in the creation of odors as well as corrosive and toxic gases from the decomposition of the solids.

Domestic sewage is more than 99.9% water and generally between 0.05% to 0.075% dissolved and suspended solids. The broad classification "strength" is used to indicate the level of organic matter in sewage. A weak sewage generally results from high infiltration to the sewer system. A strong sewage indicates the presence of industrial wastes. There is little difference in sewage strength that is related to living standards.

The two most important parameters for defining the strength of sewage are the biochemical oxygen demand (BOD) and suspended solids concentration. BOD is a measure of the amount of organic matter present, expressed in terms of the oxygen required to biologically oxidize the material to a stable form. A five-day period is generally used for the test. Thus, the 5-day BOD is an expression for the strength of sewage. The use of a broad test of this type is necessary because sewage is a mixture of many organic compounds, making it nearly impossible to conduct a complete chemical analysis. Furthermore, many of the compounds present are partly refractory to biological oxidation, making a chemical analysis meaningless. The BOD test measures the strength in a manner that most nearly represents treatment accomplishment and potential impact of the flow on the receiving body of water.

Suspended solids concentration is also an important characteristic of domestic sewage. The sewer system must be designed to transport the solids. Suspended solids also represent a load to the treatment plant. Solids not removed in the treatment process may accumulate on the bottom of the receiving stream, creating a pollution problem. The suspended solids in domestic sewage are generally more than 80% organic matter. These solids originate from fecal material, laundry and kitchen wastes. Sewage will contain some organic material from laundry and kitchen wastes. Inorganic matter also originates from washing vegetables and from laundry wastes. Inorganic suspended material is known as grit or fixed solids.

Table 25.1 presents information on the range of strength-related parameters for domestic sewage that may be of interest to the sewer design engineer. Textbooks and other sources present more detailed analyses of sewage, as would be of interest in the design of treatment works.

### Quantity of Sewage

All wastewater management facilities should be designed to serve the needs of the contributing area for some time into the future. The design engineer for a wastewater treatment facility may use several methods for determining future design populations. These methods will always include population projections for the design period. In sewer design, the engineer needs to know the area to be sewered and the developed density. The developed density will depend on the planned or zoned activity. The area to be served can be established from available maps or from field surveys. Most areas subject to being sewered should be included in a comprehensive plan or have been zoned for a specific land use. The use of this information permits the design engineer to establish sewage flow for an area at complete development. This information can then be used to establish sewage flow for each segment of the sewer to be designed. Sewers are normally designed to sewer the tributary land area at complete development. Specific conditions, such as political boundaries or land use planning, may also dictate the design area. It is important to design a sewer with sufficient capacity to serve the area for at least 25 years. Regulations of the local utility may control the area to be included in the design and the design requirements.

Most states and many local utilities have established minimum design standards for sewers. The engineer must be familiar with the requirements for the area where the sewer will be installed. Table 25.2 and Table 25.3 present data that may be used in the absence of any local requirement. Table 25.2 presents the requirements of Fairfax County, VA. Table 25.3 is taken from the sewage regulations of the Virginia Department of Health. No separate allowance is needed for infiltration if the design and construction keeps this source of flow below 100 gpd/in-mile of pipe.

### Variation in Sewage Flow

Sewage flow from commercial, business and industrial establishments are generated during the operating hours. Very little flow is generated from many of these sources between late evening and early morning, as well as on weekends and holidays.

Sewage flow from residential areas is associated with activities in the homes, with peak flows occurring generally from 7 to 10 A.M. and in the evening between 6 and 10 P.M. Peak flow hours will vary somewhat for different sections of the country, as they depend on the living habits of the community. Figure 25.1 shows a typical daily variation in sewage flow for a home. The sewage flow from a home will be very close to the rate of water use. However, sewage flow in a sewer system will differ from the rate of water use

**TABLE 25.1 Strength of Sewage in mg/L**

	CONSTITUENT		
	WEAK	NORMAL	STRONG
Total Solids*	400	700	1000
Suspended Solids	100	200	350
5-Day BOD	100	200	400

\*Total solids includes the dissolved inorganic material from the water supply

in two distinct ways. The peaks in sewage flow are less pronounced because the time of concentration to a measuring point will be different for different segments of the system. For example, flow from a section of the community that is 0.25 miles away will reach the measuring point after the flow from 0.10 mile away has passed. The peak sewage flow at a treatment plant will occur some time after the peak demand on the water supply because of the flow time in the sewer. The water supply is a pressure system and a demand is placed on the source at the instant water is drawn at a faucet. Thus, peaks in flow in a sewer system occur later than what is shown in Figure 25.1 and will be less pronounced. The actual conditions will depend on the size and configuration of the sewer system. Note from Figure 25.1 that water use drops to near zero from about midnight until about 6 A.M. Flow in a sewer system during these hours will be primarily infiltration, except for some industrial and business areas.

The design engineer must design sewers to accommodate peak flows. Table 25.2 and Table 25.3 provide information on daily rates of sewage flow generation. A peak factor is used to account for the differences in the average daily flow and the instantaneous flow. Nominal infiltration and inflow is accounted for by the daily flow quantities shown in Table 25.2 and Table 25.3. The table values are considered to be high for properly constructed sewers. No additional allowance needs to be made for infiltration and inflow. These average daily flows shown in Table 25.2 and Table 25.3 must be adjusted for peak flow rates for use in designing a sewer. The periods of minimum flow are generally not of a concern in sewer design.

Some local utilities have developed or adopted from other sources regulations for establishing peak flow. Most, if not all, states also have minimum standards. Most equations for calculating a peak factor are based on population. Two such equations are given below.

**TABLE 25.2 Recommended Average Design Flows from the Requirements of Fairfax County, Virginia**

TYPE OF DEVELOPMENT		DESIGN FLOW (gpd)
Residential:	General	100/person
	Single-family	370/residence
Townhouse		300/unit
Apartment unit		300/unit
Commercial:	General	2000/acre
	Motel	130/unit
	Office	30/employee 0.20/net square feet
Industrial:	General	10,000/acre
	Warehouse*	600/acre
School Site:	General	16/student

\*Varies with type of industry

**TABLE 25.3 Recommended Design Flows as Required by the Regulations of Virginia**

SOURCE OF FLOW	DESIGN UNITS	Flow (gpd)
Dwellings	Per person	100
Schools with showers and cafeteria	Per person	16
Schools without showers and cafeteria	Per person	10
Boarding schools	Per person	75
Motels (rooms only)	Per room	130
Trailer courts	Per trailer	300
Restaurants	Per seat	50
Interstate or through restaurant	Per seat	180
Factories	Per person	15–35
Hospitals	Per bed	300
Nursing homes	Per bed	200
Doctors' offices	Per 1000 square feet	500
Laundromats	Per machine	500
Community colleges	Per person	15
Swimming pools	Per swimmer	10
Theaters	Per seat	5
Picnic/park areas	Per person	5
Camps with flush toilets	Per camp site	100

The above flows are considered to be adequate to include limited infiltration.

State and local utility regulations are generally considered as being minimum design conditions.

\*Waterworks Regulations, Va. Dept. of Health (1993).

$$\text{Peak Factor} = \frac{5}{p^{0.2}} \quad (25.1)$$

$$\text{Peak Factor} = 1 + \frac{14}{4 + p^{0.5}} \quad (25.2)$$

where  $p$  is the contributing population in thousands. The peak factor should be limited to a maximum value of 5 and a minimum value of 2.5.

A more useful approach in sewer design is the use of curves to establish the peak factor. This type of curve is shown in Figure 25.2 and Figure 25.3. The average daily flow that has been calculated by using the tributary area and appropriate densities is then multiplied by the peak factor to establish the design flow for use in sewer design.

### Hydraulics of Sewers

Sewage is considered to have the same hydraulic characteristics as water. It is an incompressible fluid having viscous properties similar to water. Both gravity sewers and force mains are generally designed as circular pipes flowing full. Design practice also assumes that the flow is steady and uniform for each segment of pipe. In instances where building connections are adding flow to a segment of line, the flow at the downstream end of the segment is used for the design of the entire segment. A manhole should be installed where sewers other than building laterals connect to the line being designed. Design segments always run from manhole to manhole. Uniform flow applies when the cross-sectional area and slope of the segment are constant

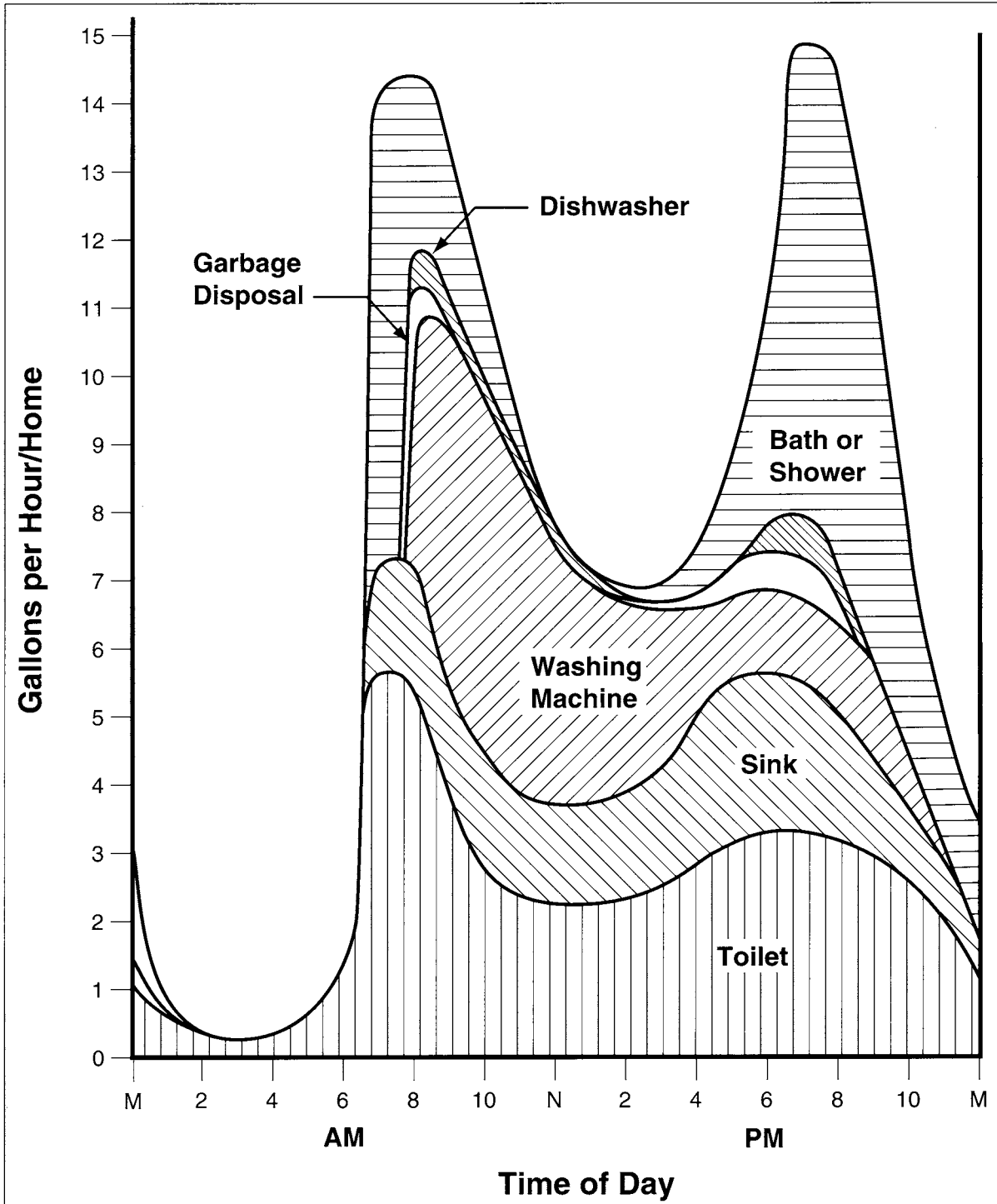


FIGURE 25.1 Hourly variation of sewage flow.

throughout the length. Manholes are located at all changes in slope and pipe diameter.

The flow rate is equal to the cross-sectional area of flow times the velocity at the section. This is expressed by the continuity equation,

$$Q = AV \tag{25.3}$$

where  $Q$  is the quantity of flow passing the reference point

in cubic ft per second (cfs),  $A$  is the cross-sectional area of flow in square ft (sq ft or ft<sup>2</sup>) and  $V$  is the velocity of flow at the section in ft per second (fps).

If the flow is steady and uniform as generally assumed, the continuity equation is applicable and Eq. (25.4) applies.

$$Q = A_1V_1 = A_2V_2 = A_3V_3 \cdots A_nV_n \tag{25.4}$$

According to the principle of conservation of mass, mass

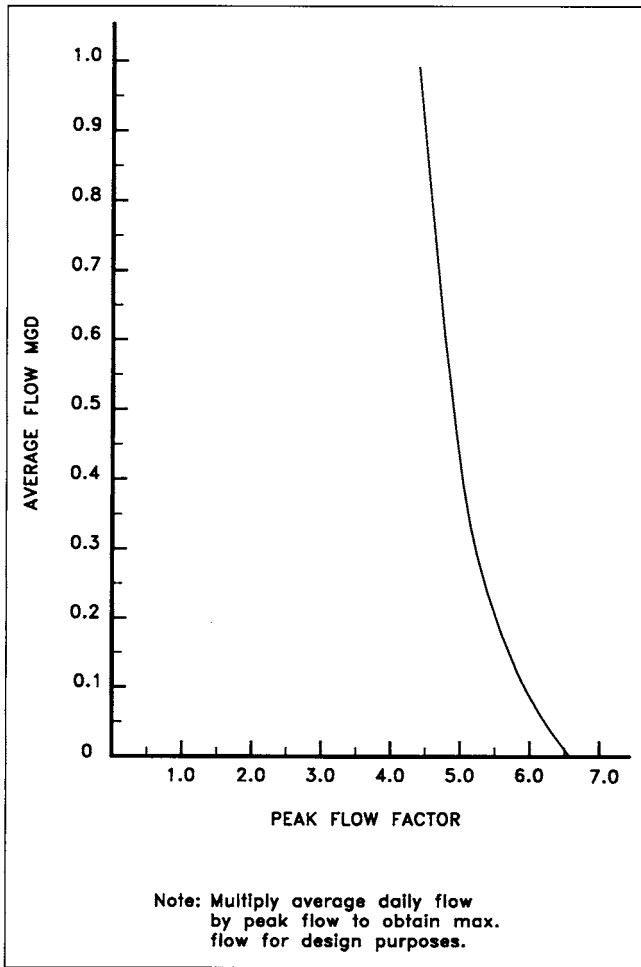


FIGURE 25.2 Peak-flow curve 0–1.0 MGD.

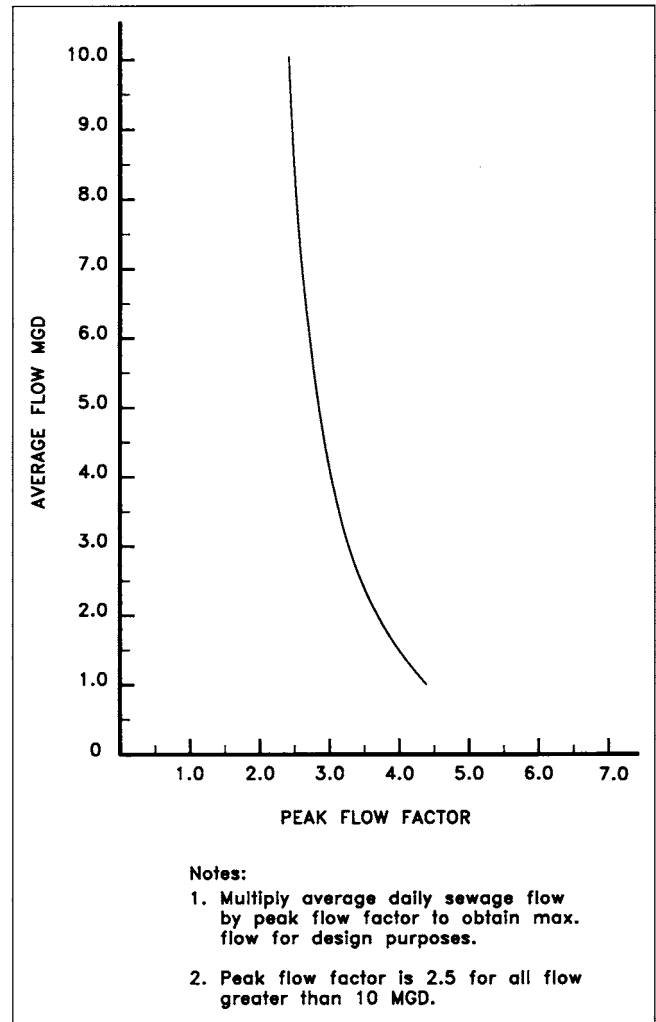


FIGURE 25.3 Peak-flow curve 1.0 and greater MGD.

can be neither created nor destroyed. Since  $Q$  remains constant along a segment of sewer having a constant slope and diameter, the velocity remains constant.

The energy of a particle of mass is the sum of the position of the particle plus the momentum of the particle. In a homogenous fluid, all particles in the fluid at a reference section have the same position energy (a particle at any depth has the same energy as any particle at the surface). The position energy for open channel flow is the water surface. The position energy for a closed conduit under pressure, such as a sewage force main, is the surface to which the liquid would rise if a vertical open tube were inserted in the top of the conduit. The pressure is indicated by the height of the water column above the center of the conduit and is known as pressure head. This is shown in Figure 25.4.

The momentum of a fluid particle is called velocity head. When a fluid moves from a stationary state to a moving state, some position energy is converted to momentum energy. This velocity head is defined as

$$H = \text{Velocity Head} = \frac{V^2}{2g} \quad (25.5)$$

where velocity head,  $H$ , is in feet = position energy equivalent of the momentum,  $V$  is the velocity of flow in ft/sec, and  $g$  is the acceleration of gravity in ft/sec<sup>2</sup>. A line that connects the values of position energy at various points along a segment of sewer is called the “Hydraulic Grade Line” (HGL). The line connecting the total energy values, which are the sum of the momentum (velocity head) plus position energy, is called the “Energy Grade Line” (EGL). Under steady, uniform flow conditions, the two lines are parallel and separated by a distance equal to the velocity head of the flow. This is shown in Figure 25.4(a) and (b). The slope of the two lines is the rate of energy loss or energy gradient, and the drop in the lines over a length  $L$  is the energy loss, also known as head loss or friction loss. Under gravity flow conditions, the HGL and EGL are parallel to the sewer invert (lowest line of flow). Sewers are, therefore, designed and constructed on the basis of invert elevation.

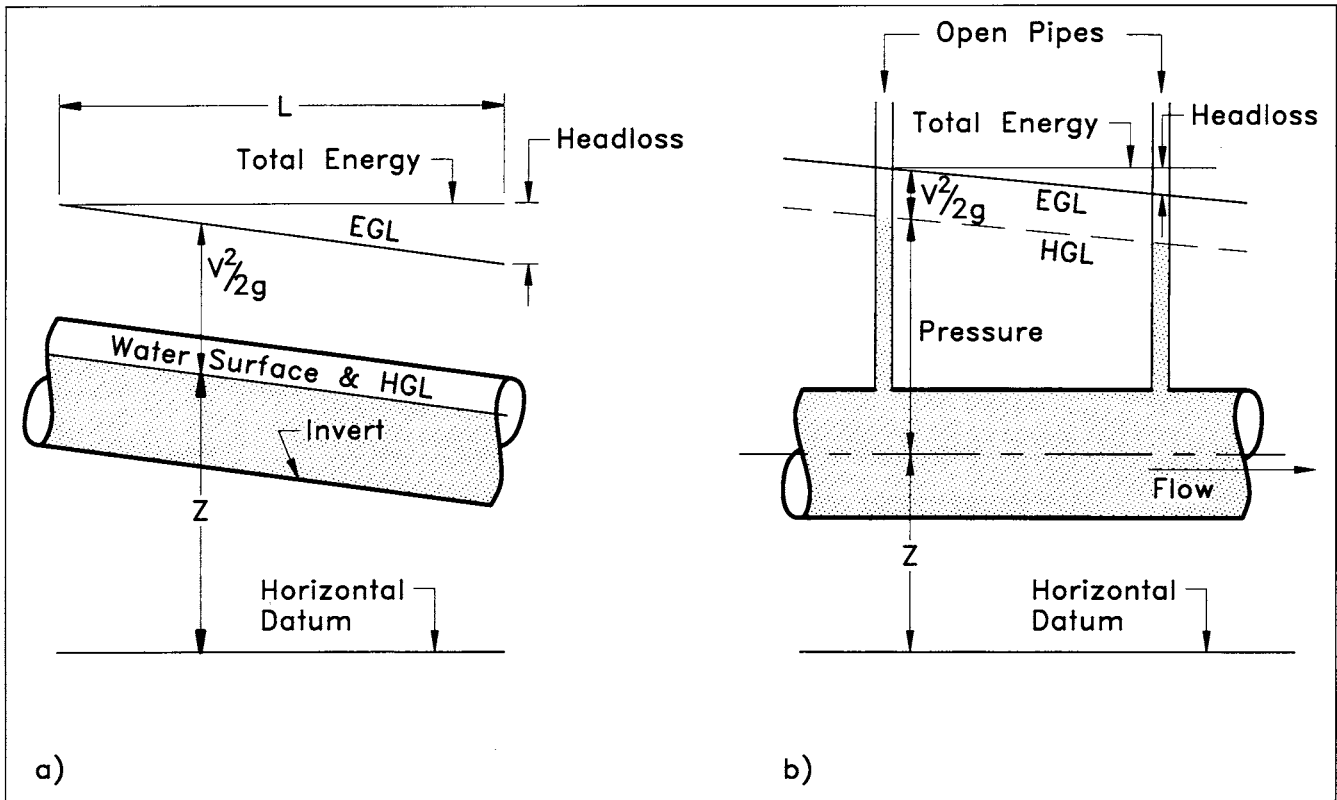


FIGURE 25.4 HGL and EGL for gravity flow (a); HGL and EGL for pressure flow (b).

A pump is a means of imparting energy to a flow. This added energy is generally expressed in feet and is the height that the energy grade line is raised by the pump. This is shown in Figure 25.5.

Flow of fluid in a conduit may be defined as being laminar or turbulent. Laminar flow is flow of molecular layers of fluid where one layer is moving with respect to the ad-

jacent layer. There is zero movement at the conduit wall under laminar flow conditions. The resistance to movement is the viscosity of the fluid. Viscosity is due to cohesion and interaction between adjacent fluid molecules. The resistance to movement is an irrecoverable energy loss that is dissipated as heat. For laminar flow, the rate of energy loss is a function of the fluid and the velocity of the fluid and not related to type of pipe material. This resistance to movement is represented by the hydraulic gradient.

The type of flow present is defined by the Reynolds Number  $N_R$

$$N_R = \frac{DV\rho}{\mu} \tag{25.6}$$

where  $D$  = diameter of conduit in ft,  $V$  = velocity of flow in ft/sec,  $\rho$  = fluid mass density in lbs-sec<sup>2</sup>/ft<sup>4</sup>, and  $\mu$  = shear between fluid layers in lbs-sec/ft<sup>2</sup>.

For laminar flow, the friction factor is related directly to the Reynolds number. The upper limit of the Reynolds number for laminar flow of water is 4000. In order to give you a frame of reference, water, flowing at a velocity of 2 fps and having a temperature of 68°, in an 8-in. pipe has a Reynolds number of 50,000. This indicates that the flow is well within a turbulent flow regime, as is always the case for water and sewage flow in conduits. In sewers, the fluid particles move in a heterogeneous manner, causing a com-

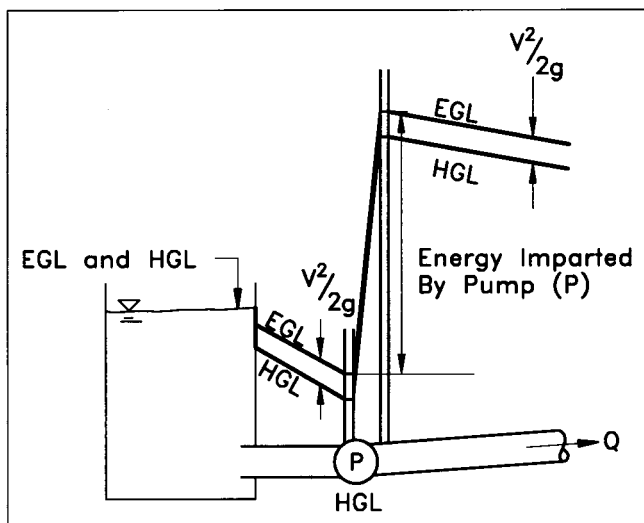


FIGURE 25.5 Change in HGL and EGL by a pump.



plete mixing of the fluid. This movement or turbulence is primarily created at the conduit wall by the roughness of the surface and by shear forces between the wall and the moving fluid. The conduit wall roughness and the shear forces generate turbulence vortices that move out from the wall, where they are subsequently dissipated by viscous attrition with other fluid particles. This generation and dissipation of turbulence consumes energy, which is converted to heat. The energy dissipated within the flow is lost as heat. This loss is known as head loss or friction loss. As was noted for laminar flow, the slope of the HGL and EGL shows the rate of energy loss along a length of conduit. The drop in the HGL over a length of conduit is the amount of friction loss.

The roughness of the conduit interior wall is of significant importance in any hydraulic design. Pipe manufacturers strive to produce a pipe having a very smooth wall surface, as this is an important item in marketing the product. The smoother the pipe wall, the less turbulence generated and, hence, the less energy loss in the flow. Most pipe marketed today is classified as smooth pipe.

The hydraulic design of a sewer system requires a procedure for determining the rate of energy loss. A number of equations have been developed for this purpose. Most of the equations were developed empirically from field or laboratory data, but some have been developed by dimensional analysis. Since the head loss depends on the level of turbulent vortices generated at the conduit wall, all calculations of head loss must relate to the type, height and spacing of roughness protrusions on the pipe surface. These conduit characteristics depend on the type of pipe material selected. Pipe manufacturing is sufficiently developed to where the engineer can expect all pipe of a given material to be of uniform quality. Furthermore, since most if not all pipe presently available can be considered as being smooth wall, the engineer needs to consider other characteristics when selecting the pipe material to be included in the construction. These include resistance to deterioration, ease of handling and laying, cost and availability. The characteristics of new pipe should not be used in design because all pipe material will deteriorate with age and use. There will be erosion of the surface, growth of slimes on the surface and accumulation of solids with use and aging. Sewers should be designed to function properly at the end of the design life.

There are several equations available for determining friction loss in sewer conduits. The "Manning Equation" is by far the most widely used. This empirical equation is accepted by all reviewing agencies. It is generally the only equation used by pipe suppliers in marketing material. Velocity is defined in open channel flow by the Manning Equation as

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \quad (25.7)$$

where  $V$  = the velocity of flow in ft/sec,  $n$  = the Manning

coefficient,  $R$  = hydraulic radius = area/wetted perimeter,  $S$  = the slope of the HGL (hydraulic gradient) in ft/ft =  $H_L/L$ , and  $H_L$  = head loss = the drop in the HGL in feet per length of sewer  $L$ .

The Manning Coefficient  $n$  must be selected by the engineer. Selection is based on the type of pipe being employed in the design, giving consideration to condition of the pipe at the end of the design period.

Since  $Q = AV$ ,

$$Q = A \left( \frac{1.486}{n} R^{2/3} S^{1/2} \right) \quad (25.8)$$

In turbulent flow, the conduit wall roughness is the principal condition that determines the intensity of turbulent vortices generated and, hence, the energy loss associated therewith. The Manning coefficient or "friction factor" is found in catalogue material supplied by pipe manufacturers, as well as in texts on hydraulics and related subjects. A value of 0.013 is generally used in sewer design. The Manning equation may be arranged to calculate the head loss for a length of sewer  $L$  as follows:

$$H_L = \left[ \frac{VnL^{1/2}}{1.486 R^{2/3}} \right]^2 \quad (25.9)$$

Information available from pipe suppliers may indicate that a Manning  $n$  value as low as 0.009 is appropriate for their product. This would indicate that for a required velocity and capacity the required slope of the HGL would be reduced 48% from that for an  $n$  value of 0.013.

$$\left[ \frac{.009}{.013} \right]^2 \times 100 = 48\% \quad (25.10)$$

Or, for a constant hydraulic gradient (HGL slope), the velocity and capacity would be increased 44.4% ( $0.013/0.009 = 1.444$ ). A Manning coefficient of 0.013 is conservative for smooth wall pipe available for sewer construction. This value of  $n$  is recommended and generally used because its use allows for erosion of the pipe wall, slime build up and other minor obstructions to flow. These forms of pipe deterioration, resulting in reduction of capacity, occur over time. A sewer pipe material should give reliable service for at least 50 years. Other useful relationships for the Manning equation for a given diameter of conduit include:

$$Q1/Q2 = n2/n1 \quad (25.11)$$

where  $n$  is a constant.

$$\frac{V_1}{V_2} = \left[ \frac{H_1}{H_2} \right]^2 \quad (25.12)$$

Since all pipe material used in sewer construction is essentially smooth wall, there will be little difference in the pipe roughness and, hence, in the value of  $n$  to be used in a design for different pipe material. The selection of the type

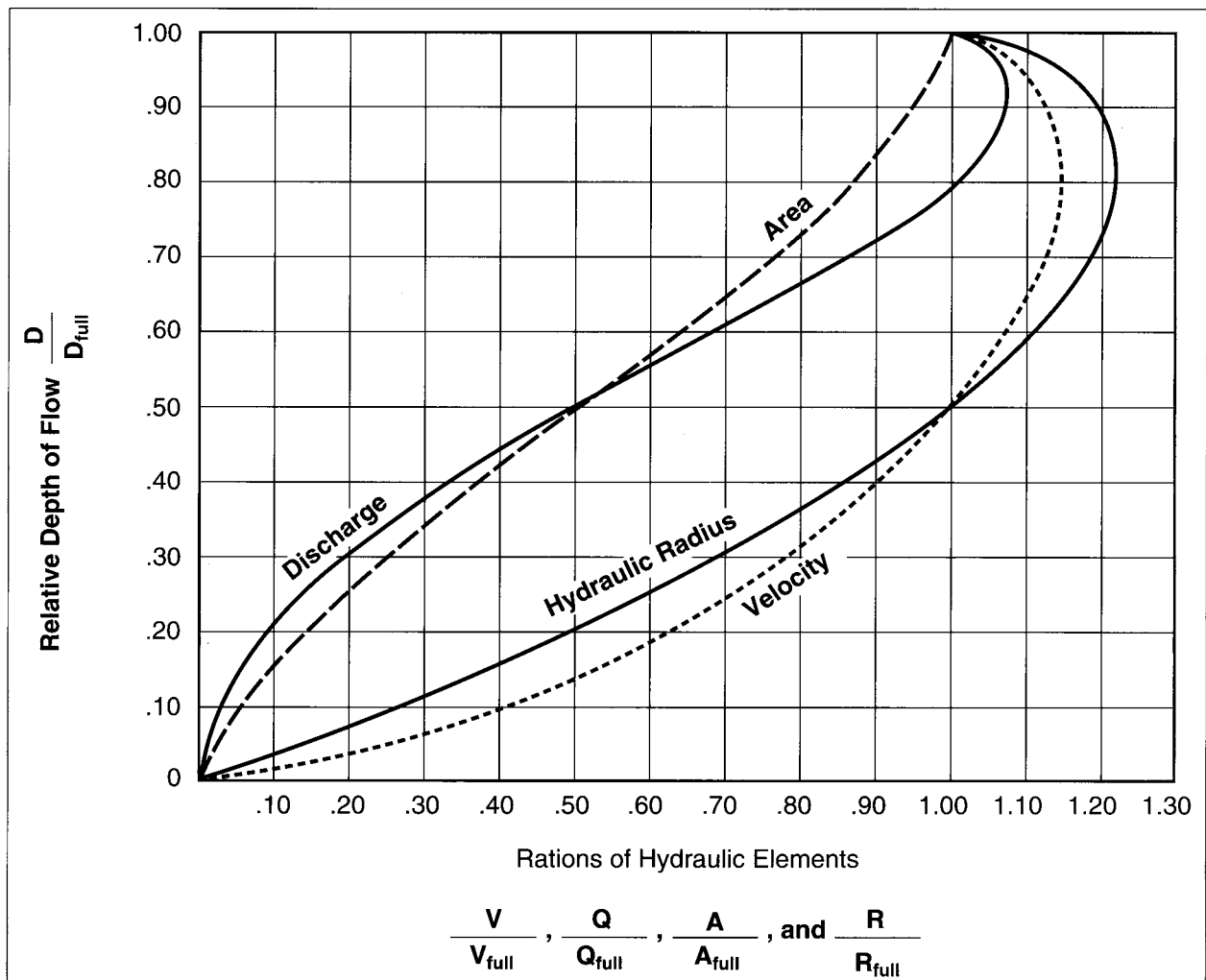
of pipe to be used in a design and included in the contract specifications as being acceptable for use in construction should be based on the long-term serviceability of the pipe material. Serviceability factors should include expected useful life, resistance to problems with infiltration, ease of installation, resistance to corrosion and erosion, and maintenance requirements. First cost alone should not be the determining factor.

Gravity sewers are designed to flow full at the design peak flow. Figure 25.6 presents the hydraulic properties of circular sewers for all depths of flow. As shown by the Manning equation, one of the factors influencing the velocity or capacity is the hydraulic radius  $R$  (area/wetted perimeter). The area of flow and the corresponding wetted perimeter do not vary uniformly with depth of flow. As shown by Figure 25.6, the hydraulic radius is greatest at 0.8 of full flow depth. The velocity will also be the greatest at 0.8 of full depth. While the cross-section area of flow is maximum at full depth, flow  $Q = AV$  and the maximum product of  $A$  and  $V$  is at 0.94 depth as shown by Figure 25.6. The capacity at the 0.8 depth is the same as for the full depth.

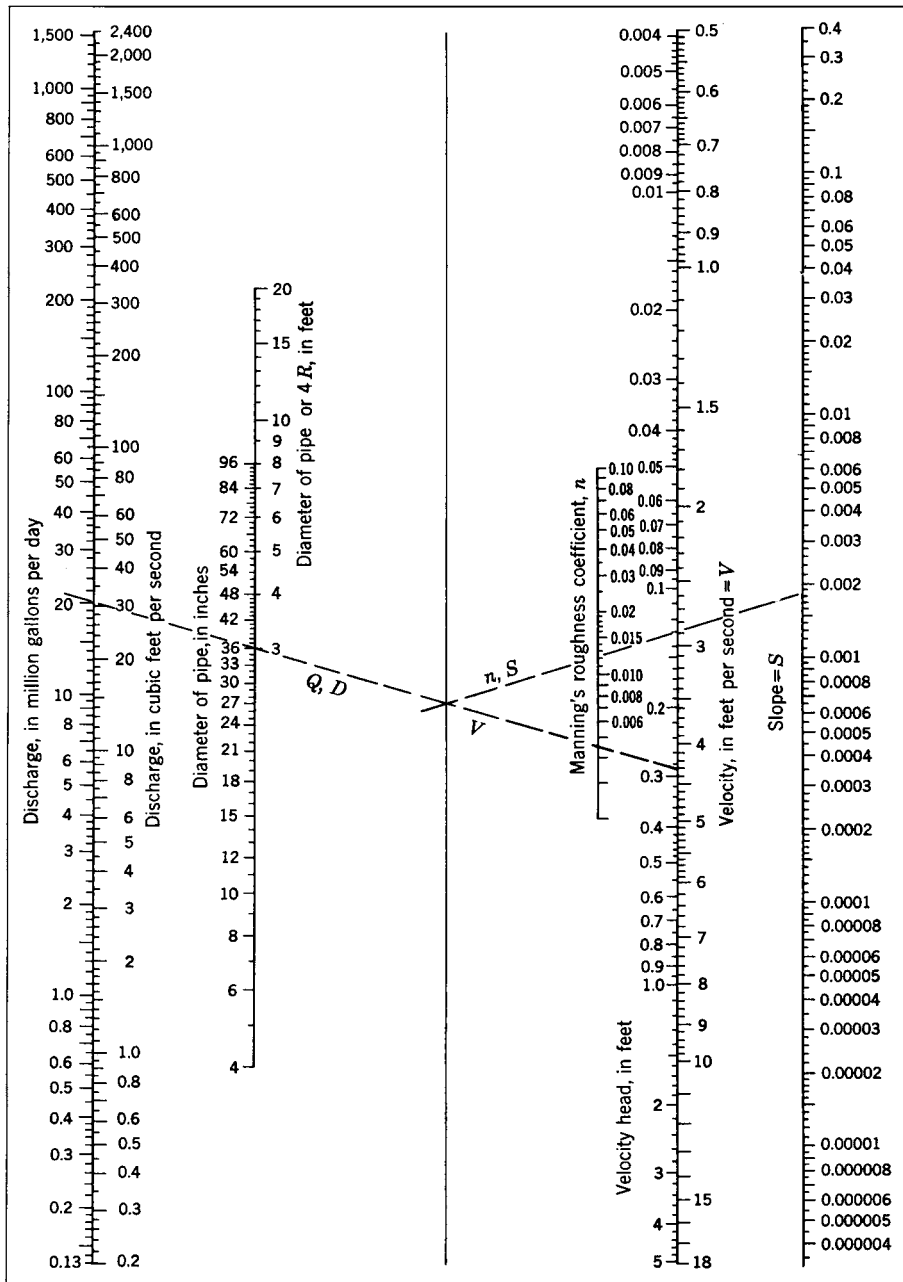
Therefore, a sewer under gravity flow will never flow full because the flow will by natural law flow at the lower energy (energy of flow = surface elevation plus momentum). Any sewer flowing at greater than 0.8 depth is being surcharged by a downstream obstruction. Since the velocity of flow at the 0.8 depth is about 14% greater than at full flow, the hydraulic gradient required to achieve the equivalent full depth capacity is reduced. However, it is not common to design for the 0.8 depth conditions.

Sewers will flow at less than 50% of full capacity most of the time. At 50% capacity, the depth of flow and velocity is also 50% of full flow conditions. Sewers should be designed to provide a minimum velocity of 2.0 to 2.25 fps at full flow. A full flow velocity of 2.25 fps will ensure a scouring velocity of 1.0 fps at about 11% of full flow (see Figure 25.6). A scouring velocity of 1.0 fps is considered adequate to prevent deposition of solids.

Various forms of nomographs and slide rules are available for solving the Manning equation. These devices are normally used in sewer design to facilitate solving the equation. A typical nomograph is shown in Figure 25.7. The



**FIGURE 25.6** Nomograph of selected hydraulic elements for circular pipe flowing partially full.



**FIGURE 25.7** Nomograph for solving the Manning equation. From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York.

nomograph includes lines for five hydraulic elements and a pivot line. The continuity equation ( $Q = AV$ ) may be solved by knowing two of three terms and finding the third from the nomograph without using the pivot line. The right side of the pivot line includes  $n$ ,  $V$ , and  $S$ . Knowing two of the three will establish the pivot point, from which knowing  $Q$  or  $D$  permits finding the other. Likewise,  $Q$  and  $D$  can be used to establish the pivot point, and knowing either  $n$  or  $S$  will permit solving for the other.

**EXAMPLE PROBLEM** (a) What is the slope and velocity of an 8-in. pipe carrying 1 cfs? Using

Figure 25.7, knowing  $Q$  and  $D$ , find  $V = 2.9$  fps. Knowing  $D$ ,  $Q$  gives pivot point, with  $n = 0.013$ ,  $S = 0.0019$ .

(b) What is the depth and velocity for the pipe in part (a) when the flow is 0.4 cfs? Knowing  $Q/Q_{full} = 0.4/1.0 = 0.4$  and using Figure 11.6, depth of flow = 43% of 8 in. = 3.44 in., velocity = 95% of  $V_{full} = 2.75$  fps.

There are several types of design aids in addition to the nomographs shown as Figure 25.7 and Figure 25.29 for use in solving both the Manning and the Hazen-Williams

**TABLE 25.4 Common Conversion Factors**

1 cfs	= 449 gpm
1 cu ft	= 7.48 gallons
1 MGD	= 1.55 cfs = 695 gpm
1 gal	= 231 cu in = 8.34 lbs
1 cu ft	= 62.4 lbs
Pressure	= 62.4/144 = .433 psi per foot of water

equations. These include special nomographs and various forms of slide rules. The conventional type nomograph as shown in Figure 25.7 and the slide rule types are generally preferred by design engineers. Some pipe suppliers may provide these design aids. Some common conversion relationships for water are shown in Table 25.4. Some common design relationships from using the Manning equation are shown in Table 25.5.

The data in Table 25.5 show that an 8-in. sewer installed to provide a minimum velocity of 2.0 fps has a capacity of 314 gpm (0.45 mgd). This flow would be subject to a peak factor of 5 (see Figure 25.2), giving an average design flow of 63 gpm or 90,700 gpd. At a per dwelling unit daily flow of 370 gallons (see Table 25.2), this 8-in. sewer will serve 243 housing units. Likewise, a 10-in. sewer will serve 389 single-family units (note that the peak flow factor for the 10-in. line capacity of 489 gpm (0.7 mgd) is 4.8. A 12-in. sewer will serve 633 single-family units. Velocities greater than 2.0 fps and smaller  $n$  values will provide still greater capacity. Many developments are sewerred by 8-in. sewers

and line sizes greater than 12 in. are seldom needed except for main or trunk sewers. An 8-in. diameter sewer is considered the minimum size line in public sewer design.

### Pipe Materials and Loads on Buried Conduits

The engineer now has several excellent types of pipe material available for use in sewer construction. It is important that the proper selection be made for the specific installation conditions. Pipe materials are classified as being either flexible or rigid. Pipe manufactured from materials such as concrete and vitrified clay are classified as rigid wall pipe. A deflection of the wall of a rigid wall pipe will result in pipe failure. The most common type of flexible pipe is polyvinyl chloride (PVC), which is widely used in both sewer and water line construction. A limited wall deflection of a flexible wall pipe does not reduce the serviceability of the conduit. Ductile iron is also a flexible pipe material.

Rigid pipe is classified on the basis of bursting strength, and for pressure pipe the internal pressure rating is also included. The bursting strength is established by standard tests and is provided by the manufacturer in pounds of external vertical force per foot of pipe. It is listed as the three-edge bearing strength that reflects the type of standard test procedure. Pipe is designed to withstand both the internal pressure and the external loading simultaneously. The manufactured wall thickness is adjusted to provide the required strength. The engineer should have library catalog data available for the types of pipe material being considered for incorporation into a design. The strength of rigid pressure pipe will be shown by pipe class, such as Classes 150, 200, etc.

Gravity flow sewer pipe is designed to withstand external pressure loadings. Rigid wall pipe is manufactured in several wall strengths as may be required for design conditions. For example, pipe classes are designated as: 1500, 2400,

**TABLE 25.5 Hydraulic Elements Determined by Using Figure 25.7 and Equations 25.11 and 25.12**

DIAMETER (in.)	$n$	$V$ (fps)	$S$ (ft/ft)	$R^{2/3}$	$Q$ (gpm)
8	0.013	2.0	0.0033	0.303	314
8	0.013	2.25	0.0042	0.303	354
8	0.010	2.0	0.0020	0.303	314
8	0.012	2.0	0.0028	0.303	314
10	0.013	2.0	0.0025	0.351	489
10	0.013	2.25	0.0031	0.351	551
12	0.013	2.0	0.0019	0.397	705
12	0.013	2.25	0.0025	0.397	793

3300, 4000 and 5000. The class strengths represent the design crushing strength in pounds per linear foot of pipe.

Flexible wall pipe is classified by the ratio of the external diameter to the wall thickness. This is expressed as “dimension ratio” (DR) or “standard dimension ratio” (SDR). Since limited deflection of the pipe can be tolerated, the strength is expressed in terms of the force required to cause a defined deflection. Pressure flexible pipe is designated by pressure class. A PVC pressure pipe manufactured by Johns-Manville under the name “Blue Brute” is available in the three classes shown in Table 25.6.

As shown in Table 25.6, a DR-25 pipe has a design pressure rating of 100 psi, while the sustained pressure strength is 350 psi. This difference provides for surges in pressure, known as water hammer, and a safety factor against failure. For example, a design pressure condition of 100 psi plus a surge maximum of 100 psi above the design pressure provides a safety factor against failure of 1.75 ( $350/200 = 1.75$ ).

### Buried Conduits

The theory for loads on buried conduits states that the load on a buried conduit is equal to the weight of the prism of earth directly over it, plus or minus the frictional shearing forces transferred to that prism by adjacent prisms of earth. The magnitude and direction of these frictional forces is a function of the relative settlement between the prisms. The equation for calculating the load on a rigid buried conduit as developed by Marston at the Iowa Engineering Experiment Station in 1913 states:

$$W_c = C_D w B^2 \quad (25.13)$$

where  $W_c$  is the load acting on the conduit after settlement has taken place, lbs/ft,  $C_D$  is a dimensionless coefficient related to the diameter of the conduit, the width of the trench, the characteristics of the fill material, and the pipe bedding conditions ( $C_D$  is obtained from tables or charts),  $w$  is the unit weight of the fill material, lbs/ft<sup>3</sup> (normal earth material may be taken as 130 #/ft<sup>3</sup>), and  $B$  is the width of the trench in feet (note that load increases as the square of  $B$ ).

The engineering design can mitigate the load on the conduit by keeping the trench as narrow as practical, as the load varies as the square of the width, and by requiring good bedding and backfill conditions. Figure 25.8 shows the types of forces affecting the load on the conduit.

Note from Figure 25.8 that the load on the conduit is related to the weight of the soil in the area ABDC. The upward shearing forces  $F$  reduce the load on the conduit. If the fill below the top of the conduit is not well compacted, the load on the conduit is increased as the material settles. It is sometimes necessary to slope the ditch walls for construction safety and convenience. Figure 25.8b and c show two construction means of widening the trench without increasing the earth load on the conduit. It is important to keep the trench as narrow as practical from the top of the pipe to the bottom of the trench because the load on the conduit increases as the square of the width  $B$ .  $B$  is taken as the width of the trench at the top of the conduit.

There are four classes of bedding as shown in Figure 25.9. The engineer is required to select a bedding type based on field conditions. Classes B or C are suitable except where unusual loading conditions exist. Class D bedding should not be used for sewer line construction. This bedding class is only used for small diameter pipe.

Class A—Concrete Cradle—bedding in which the lower part of the exterior of the conduit is set in a plain or reinforced concrete foundation of suitable thickness extending upward on each side for not less than 25% of the height of the conduit.

Class B—First Class—bedding where the conduit is set on fine granular materials in an earth foundation shaped to conform to the lower part of the conduit exterior for a width of at least 60% of its exterior diameter. The trench is then backfilled with granular materials, hand-placed and tamped in 6-in. layers to fill completely all spaces under and adjacent to the conduit for a distance of at least 1 ft above the top of the conduit.

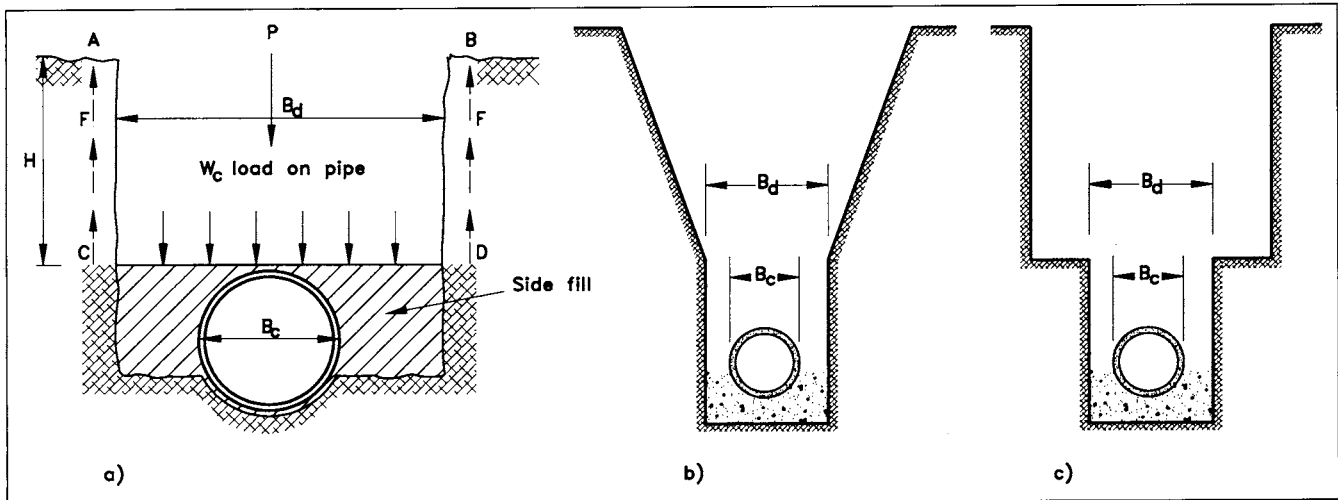
Class C—Ordinary—bedding in which the conduit is placed with ordinary care in an earth foundation shaped to fit the lower part of the conduit to the spring line. The ditch is then backfilled with granular materials, shovel-

**TABLE 25.6 Class and Pressure Characteristics of Pressure PVC Pipe**

PRESSURE	CLASS	DR	BURSTING PRESSURE	SUSTAINED
psi	100	25	535 psi	350
psi	150	18	755 psi	500
psi	200	14	985 psi	650

The class of pipe is the limit of internal pressure to be used in design.

DR is the ratio of the external diameter to the pipe wall thickness. For example, an 8-in. DR 25 pipe has an external diameter of 9.050 in. and a wall thickness of 0.362 in. Dimension ratios are controlled by manufacturing standards.



**FIGURE 25.8** Typical trench conditions for a rigid conduit. *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York.

placed and tamped to fill all spaces to a height of at least  $\frac{1}{2}$  ft above the top of the conduit.

Class D—Impermissible—bedding in which little or no care is exercised to shape the foundation to fit the pipe.

The engineer also should specify the embedment material to be used with the installation of the conduit. This information should be included in the specifications and also shown on the plans. Table 25.7 provides information on the standard material classification system. The soil types are standard under the United Soil Classification System (ASTM D2487). The materials are grouped into categories that are considered to be most suitable for embedment use.

### Flexible Conduits

Flexible sewer pipe, such as PVC, is designated by the “standard dimension ratio” as noted for flexible pressure pipe. The most often used sewer flexible pipe is SDR-35. Heavier wall thickness pipe, such as DR-21, is available for use where required to meet laying conditions. Data for the more common sizes of SDR-35 pipe is given in Table 25.8.

A flexible pipe having a DR greater than 35 should not normally be considered for sewer construction. The smaller wall thickness associated with the greater DR values may not provide long-term reliability.

A flexible pipe yields under an external load by deflecting. This deflection results in a shortening of the vertical diameter because of the load on the pipe from the fill material on top of the pipe. Excessive deflection results in failure through openings at the joints and a reduction of internal area for flow, even though the pipe has not collapsed. In conservative design, the allowable deflection is limited to 5%, although deflections up to 7.5% are used in design.

Figure 25.10 gives an illustration of the embedment zones and the forces acting on a buried flexible pipe. As shown by Figure 25.10a, the vertical load on the pipe is

distributed evenly over the width of the pipe. The vertical reaction acting on the bottom of the pipe is distributed evenly over the width of the pipe subtended by the bedding angle. The parabolically distributed side forces act through the middle 100° of the pipe. The maximum pressure, found at the pipe’s springline, is passive resistance of the backfill around the pipe. Using suitable backfill material and proper compaction is very important in establishing the field strength of a flexible conduit, as the vertical diameter cannot shorten—that is, “deflect”—without a corresponding lengthening on the horizontal diameter. The embedment zones, as shown by Figure 25.10b, illustrate the proper bedding conditions and the common nomenclature associated with pipe installation.

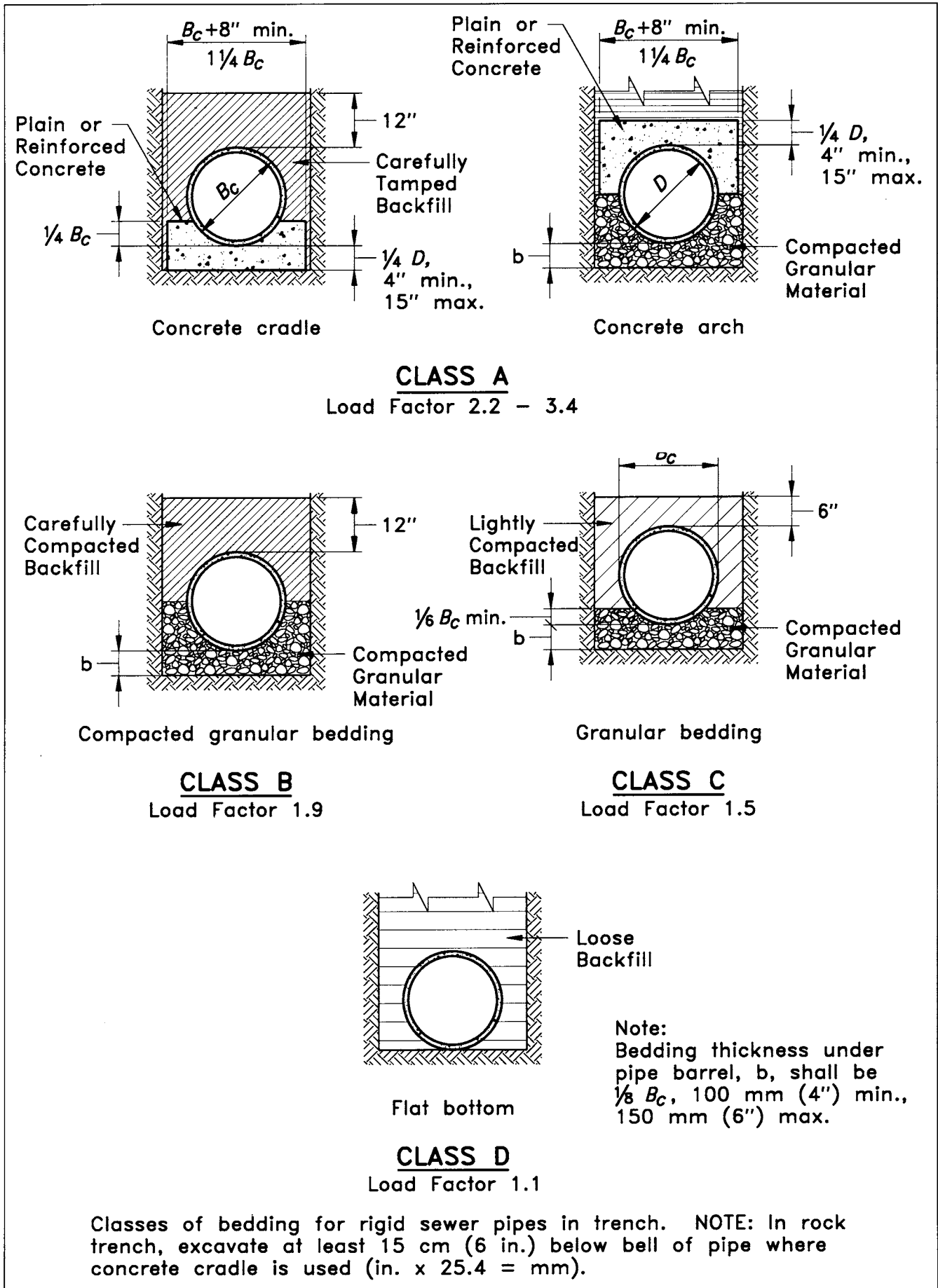
### Zone Terminology

**Foundation.** A foundation is required when the trench bottom is unstable. Any foundation that will provide continuous support without causing loss of grade or flexural deformation is suitable.

**Bedding.** The bedding directly underneath the pipe is required to bring the trench to grade. The bedding should be shaped to fit the conduit, with a depression formed to receive the bell of the pipe so that the pipe rests on the pipe barrel for its entire length. The labor to shape manually the bottom of the trench is expensive, so it is common practice to over-excavate a couple of inches and bring the trench bedding to grade with granular material, such as crushed stone. The bedding should be firm to support the pipe.

**Haunching.** The haunching area is most important in providing good pipe support. The contractor must ensure that the haunch area under the pipe is completely filled and that the area is compacted to the specified density. Most of the pipe support comes from this area.

**Initial Backfill.** The initial backfill begins at the springline and continues to 6 in. or 12 in. above the top of the conduit. When the conduit is properly installed to this level,



**FIGURE 25.9** Bedding methods for trench conduits. *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9, ASCE 1969.* Reprinted with permission from the American Society of Civil Engineers, New York, New York.

**T A B L E 2 5 . 7   E m b e d m e n t   M a t e r i a l s   C l a s s i f i c a t i o n   S y s t e m**

CLASS	TYPE	SOIL GROUP SYMBOL D 2487	DESCRIPTION	PERCENTAGE PASSING SIEVE SIZES			ATTERBERG LIMITS		COEFFICIENTS	
				1 1/2 in. (40 mm)	No. 4 (4.75 mm)	No. 200 (0.075 mm)	LL	PL	UNIFORMITY C <sub>u</sub>	CURVATURE C <sub>c</sub>
IA	Manufactured aggregates: open-graded, clean	None	Angular, crushed stone or rock, crushed gravel, broken coral, crushed slag, cinders or shells; large void content, contain little or no fines	100% ≤ 10%	≤ 10%	< 5%		Nonplastic		
IB	Manufactured, processed aggregates; dense-graded, clean	None	Angular, crushed stone (or other Class 1A materials) and stone/sand mixtures with gradations selected to minimize migration of adjacent soils; contain little or no fines (see X1.8)	100% ≤ 50%	≤ 50%	< 5%		Nonplastic		
II	Coarse-grained soils, clean	GW	Well-graded gravels and gravel-sand mixtures; little or no fines	100%	< 50% of "Coarse Fraction"	< 5%		Nonplastic	> 4	1 to 3
		GP	Poorly-graded gravels and gravel-sand mixtures; little or no fines						< 4	< 1 or > 3
		SW	Well-graded sands and gravelly sands; little or no fines		> 50% of "Coarse Fraction"				> 6	1 to 3
		SP	Poorly-graded sands and gravelly sands; little or no fines						< 6	< 1 or > 3
	Coarse-grained soils, borderline clean to w/fines	e.g., GW-GC, SP-SM	Sands and gravels which are borderline between clean and with fines	100%	Varies	5% to 12%		Nonplastic		Same as for GW, GP, SW and SP



TABLE 2.5.7 Embedment Materials Classification System (Continued)

CLASS	TYPE	SOIL GROUP SYMBOL D 2487	DESCRIPTION	PERCENTAGE PASSING SIEVE SIZES			ATTERBERG LIMITS		COEFFICIENTS	
				1½ in. (40 mm)	No. 4 (4.75 mm)	No. 200 (0.075 mm)	LL	PL	UNIFORMITY $C_u$	CURVATURE $C_c$
III	Coarse-grained soils with fines	GM	Silty gravels, gravel-sand-silt mixtures	100%	<50% of "Coarse Fraction"	12% to 50%	<4 or <"A" Line			
		GC	Clayey gravels, gravel-sand-clay mixtures				<7 and >"A" Line			
		SM	Silty sands, sand-silt mixtures		>50% of "Coarse Fraction"		>4 or <"A" Line			
		SC	Clayey sands, sand-clay mixtures				>7 and >"A" Line			
IVA <sup>4</sup>	Fine-grained soils (inorganic)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, silts with slight plasticity	100%	100%	>50%	<50% or <"A" Line			
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays				>7 and >"A" Line			
IVB	Fine-grained soils (inorganic)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	100%	100%	>50%	<"A" Line			
		CH	Inorganic clays of high plasticity, fat clays				>"A" Line			
V	Organic soils	OL	Organic silts and organic silty clays of low plasticity	100%	100%	>50%	<4 or <"A" Line			
		OH	Organic clays of medium to high plasticity, organic silts			>50%	<"A" Line			
	Highly organic	PT	Peat and other high organic soils							

Reprinted with permission from American Society for Testing and Materials, *ASTM Standards in Building Codes* Vol. 3.

<sup>4</sup>Includes Test Method D 2487 borderline classifications and dual symbols depending on plasticity index and liquid limits.

NOTE—"Coarse Fraction" as used in this table is defined as material retained on a No. 200 sieve.

**TABLE 25.8 Characteristics of the Most Commonly Used Sizes of SDR-35 Pipe**

NOMINAL SIZE LAYING LENGTH	EXTERNAL DIAMETER	WALL THICKNESS	WEIGHT (lb/ft)	LAYING LENGTH
8 inches	8.4 inches	0.24 inches	4.42	20
10	10.5	0.300	6.93	20
12	12.5	0.36	9.91	20
15	15.3	0.43	14.90	12.5

Reprinted with permission from American Society for Testing and Materials. *ASTM Standards in Building Codes* Vol. 4.

problems related to pipe bedding should not develop. If proper compaction is not provided above this point, there may be problems with surface settling for some time.

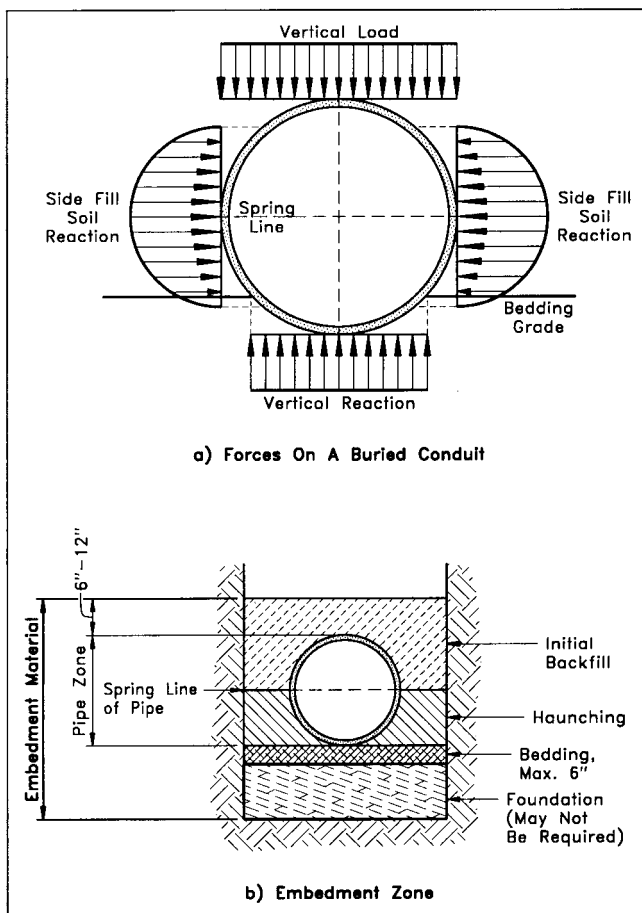
The equation for determining the load on flexible conduits installed under a ditch condition, as developed by Marston, is

$$W_C = C_D w B_D B_C \quad (25.14)$$

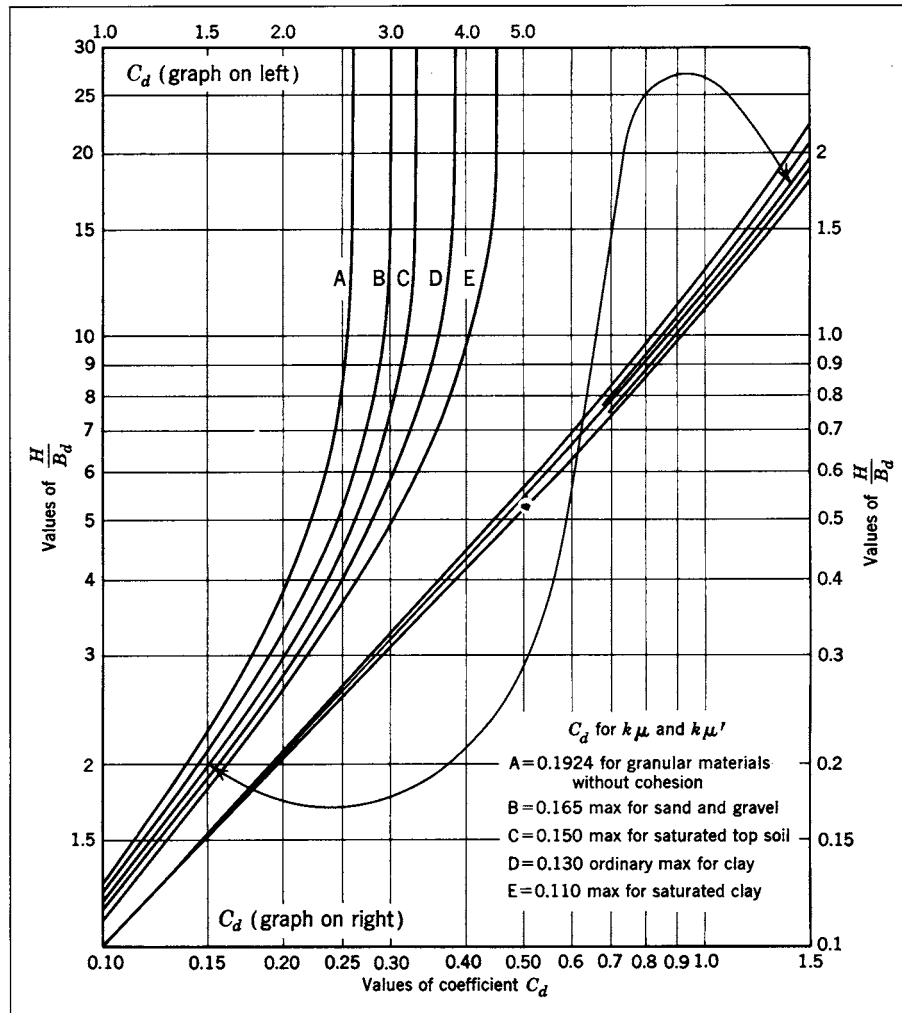
where  $W_C$  = external earth load on the conduit (lbs/ft),  $C_D$  = load coefficient for conduits installed in a trench condition (suitable charts are available for determining  $C_D$ , as shown by Figure 25.11),  $w$  = unit weight of backfill material in (lbs/ft<sup>3</sup>),  $B_D$  = width of the trench at the top of the conduit in feet,  $B_C$  = external horizontal diameter of the conduit in feet.

The ratio of the external load for rigid versus flexible conduits is given by the ratio of Eqs. (25.13) to (25.14) and is equal to  $W_D/W_C = B_D/B_C$ . This ratio shows that the load on a flexible conduit is significantly less than that for a rigid conduit installed under the same conditions ( $C_D = \text{constant}$ ). This is because the load on a buried conduit is modified by the response of the conduit. A minor deflection of the flexible conduit can materially reduce the load if the conduit has been properly installed. "Properly installed" means that the conduit has been placed in a relatively narrow trench, properly bedded, and that the backfill has been properly compacted to above the top of the conduit. Where the trench is sufficiently narrow, a part of the backfill load is transferred to the sides of the trench by shear action. Where the backfill is not properly placed and compacted to the top of the conduit, the consolidation of the fill material along side the conduit results in an additional load being placed on the conduit as the fill settles. The specifications and field inspection must ensure that the trench width is limited, the pipe is properly bedded and the backfill is properly placed and compacted to 6 in. above the top of the conduit. This will mitigate problems with failing conduits and infiltration.

While it is desirable to keep the trench width as narrow as possible, it is necessary to provide space for the workmen to install the pipe. Also, room must be allowed for a trench box. Normally a 42-in. trench is suitable for sewer diameters up to 16-in. Above 16 in., the trench width should be the pipe diameter plus 24 in. (allows 12 in. on each side of the pipe for the trench box and the workmen). Figure 25.11 provides a graphical means for determining  $C_D$  for use in Eq. (25.13) and (25.14).  $H$  is the height of the fill



**FIGURE 25.10** Forces on a buried conduit (a); Embedment zones (b). Reprinted with permission from College of Engineering, Utah State University, Logan, Utah.



**FIGURE 25.11** Values of  $C_D$  for trench conditions. (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice, No. 9*, ASCE 1969. Reprinted with permission of ASLE.)

from the top of the conduit to the surface. Other parameters are as defined above.

An embankment condition, as opposed to a trench condition, exists when a conduit is installed in a trench that is so wide that the sides of the trench have no effect on the load to the conduit. A transition width is where there is no additional load imposed on the conduit as the ditch becomes wider. This transition width depends on the depth and diameter of the conduit. Conduits installed in trenches wider than the transition widths are said to be installed in a "projection condition." Conduits should not be installed in this manner. Even in fill, the fill should be placed and a trench excavated for the conduit. However, where it may be found necessary to install a conduit under a projection condition, Marston has provided the following equation for determining the resulting load to the conduit.

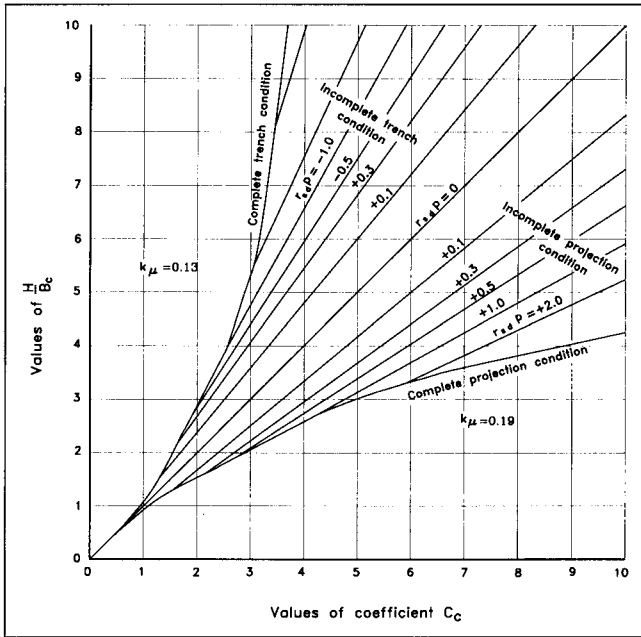
$$W_c = C_c w B_c^2 \quad (25.15)$$

where  $C_c$  is the coefficient for a projection condition (replaces  $C_D$  as shown for trench condition) and  $w$  and  $B_c$  are defined above for Eqs. (25.13) and (25.14).

A suitable chart has been developed for use in determining  $C_c$  as shown by Figure 25.12.

### Superimposed Loads

Superimposed loads and live loads may come from vehicle traffic where the conduit is located in the street or from storing of material on the surface, such as in a lumber yard. Figure 25.13 shows the general effect of the combined trench earth load plus the superimposed load. It is important to note that for pipe cover greater than 8 ft, the superimposed load is generally of little significance. In general, sewers located in a roadway pavement area should have at least 6 ft of cover over the top of the pipe. Sewers not located in roadways should be installed below the freeze depth or a minimum of 3 ft. Figure 25.13 is for a H20 loading, which is what is generally used in practice (rear axle carries 16 tons). The impact of the superimposed load must also be considered, but impact is not considered significant where the conduit has at least three ft of cover. The superimposed load is added to the dead load (trench back-fill load). Tables and equations for calculating superimposed



**FIGURE 25.12** Values of  $C_c$  for positive projection conduits. (From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice, No. 9*, ASLE 1969. Reprinted with permission of ASLE.)

loads are found in reference books and pipe supplier catalogs.

While it is important for the engineer to have an understanding of the principles related to loads on buried con-

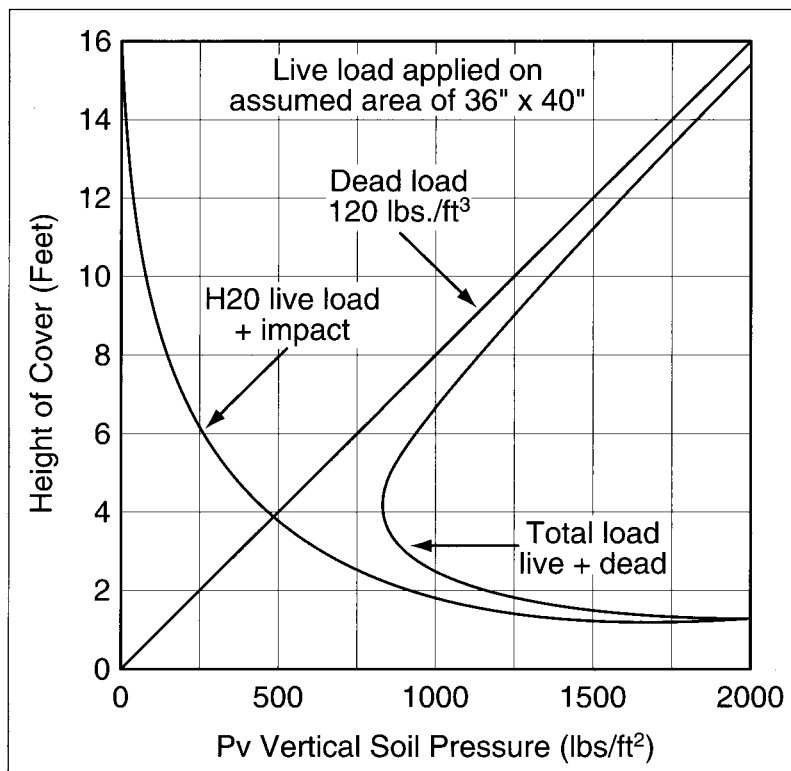
duits, it is not always practical to attempt to calculate the load by Eq. (25.13). The end result from calculating loads will be no more reliable than the assumptions as to field conditions. Soil conditions vary from location to location, and even an experienced soil engineer would find it necessary to conduct extensive field studies to make the input parameters more reliable than good engineering judgment and data available from tables.

Pipe is manufactured to standards with safety factors included for the broad classes of embedment conditions as presented above. Pipe suppliers provide tables or curves for use in design, such as those shown in Table 25.9. This table shows the deflection of SDR pipe under the indicated installed condition. As shown by the data, pipe deflections can be held to less than 5% by providing proper embedment.

Data similar to that provided in Table 25.9 for SDR 35 pipe is available from all suppliers of both sewer and pressure pipe. Standard design “Go-No Go” devices as well as an “electronic deflectometer” are available for measuring the deflection of sewer pipe after installation. The engineer should include a requirement in the specifications for this measurement to be made after the backfill load has stabilized.

**Air Testing**

The contract specifications should require that a leakage test be performed to verify the water tightness of the sewer. An air test is generally required for this purpose, although an



**FIGURE 25.13** Total pipe load—H2O live load plus dead load.

**TABLE 25.9 Maximum Long-term Deflections\* in % of SDR 35 Pipe**

ASTM EMBEDMENT MATERIAL CLASSIFICATION	DENSITY PROCTOR AASHTO T-99	HEIGHT OF COVER (FEET)														
		3	5	8	10	12	14	16	18	20	22	24	26	28	30	
Manufactured granular angular	CLASS I	0.2	0.3	0.4	0.5	0.6	0.7	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	
Clean sand and gravel	CLASS II	90%	0.2	0.3	0.5	0.7	0.8	0.9	1.1	1.2	1.3	1.4	1.6	1.7	1.8	2.0
		80%	0.9	1.4	2.3	3.2	3.6	4.1	5.0	5.5	6.0	6.4	7.3	7.7	8.2	9.1
Sand and gravel with fines	CLASS III	90%	0.2	0.4	0.6	0.8	0.9	1.1	1.2	1.4	1.6	1.7	1.9	2.1	2.2	2.3
		85%	0.7	0.9	1.7	2.2	2.6	3.0	3.5	3.9	4.3	4.8	5.2	5.6	6.0	6.5
		75%	1.1	1.8	2.9	3.8	4.5	5.5	6.8	8.5	9.9	11.3	12.7	14.1	15.5	16.8
		65%	1.3	2.4	3.6	4.7	5.5	6.8	8.5	9.6	11.4	13.0	14.5	16.0	17.3	18.0
Silt and clay	CLASS IV	85%	0.65	0.9	1.7	2.2	2.6	3.0	3.5	3.9	4.3	4.8	5.2	5.6	6.0	6.5
		75%	1.3	2.3	3.3	4.3	5.0	6.5	7.8	9.5	10.6	12.2	13.5	15.0	16.3	17.0
		65%	1.3	2.4	3.6	4.7	5.5	8.0	10.5	12.5	15.0	17.6	20.0	22.0	24.0	26.0
Organic soils and peat	CLASS V	THIS SOIL CLASS NOT RECOMMENDED														

\* Deflection is the percent reduction of the vertical diameter.  
Reprinted with permission from College of Engineering, Utah State University.

older water test may be used. The air test is easier to conduct and gives good results. The section of line to be tested is plugged and pumped up to a pressure of 4 psi plus 0.433 psi for each foot of ground water over the conduit (pressure should not exceed 9 psi). The pressure is allowed to stabilize to the sewer temperature. Pressure drop is then timed from 3.5 psi to 3.0 psi (note that 0.433 psi times the feet of ground water over the pipe must be added to these numbers). Acceptable time limits for the 0.5 psi pressure drop are shown in Table 25.10.

### Pipe Joints

Manufactured rubber compression joints should be used in sewer construction unless there are unusual conditions. The joint should be free of debris and the rubber gasket lubricated prior to installing the pipe.

### Manholes

Manholes are a required appurtenance to any sewer system. They provide access to the sewer for inspection and maintenance. Manholes should be placed at every change in grade or horizontal alignment, at every change in sewer size, at every sewer intersection and at least every 400 ft. For larger sewers, which are readily accessible to workmen, they may be spaced at greater intervals.

Only precast reinforced concrete manholes should be used except where special field conditions make it necessary to field-construct the manhole with brick. Figure 25.14 shows two typical manholes. The standard manhole includes a base that is generally cast as a part of the first barrel section. The base shown is an extended base, which

spreads the manhole weight over a larger area. Additional barrel sections are used as required to bring the manhole to where the eccentric cone is added. A manhole frame and cover is placed at the surface. The manhole supplier should adjust the height of the sections to comply with installation requirements. The precast supplier tailors each manhole for the specific site. Rubber boots are available that can be cast into the manhole at the locations where the sewers will pass through the manhole wall. A stainless steel clamp is used to secure the sewer to the boot making a watertight connection. The inside of the connection is grouted along with the placement of the bench (only shrink proof grout is used). Corrosion-resistant steps are placed inside the manhole on a 12-in. or 16-in. vertical spacing. The exterior of the manhole is waterproofed using bituminous mastic on the surface and in the joints.

Figure 25.15 shows two types of drop manholes. A drop manhole is used to reduce slopes of incoming sewers or to permit connecting a sewer entering the manhole at a higher elevation than the main sewer. Generally, if the difference in elevation is less than 5 ft and ground water or obstacles are not a problem, the slope of the higher sewer is increased to lower the sewer to where the drop manhole will not be needed.

The bench is constructed to provide a smooth section through the manhole so as to reduce energy losses, to prevent the accumulation of solids in the manhole and to provide a place for the maintenance person to stand when working in the sewer. The bench should extend to at least the springline (see Figure 25.10) of the sewer. For sewers with diameters larger than 10 in., the bench should extend

TABLE 25.10 Air Test Table

	PIPE DIAMETER												
	4"	6"	8"	10"	12"	15"	18"	21"	24"	27"	30"	33"	36"
25	2.9	4.2	5.7	7.1	8.5	10.6	12.7	14.8	17.0	19.2	21.2	23.3	25.5
50												23.3	25.5
75										19.2	21.2	24.3	28.9
100								14.8	17.0	21.6	26.8	32.2	38.5
125							12.7	16.3	21.2	27.0	33.3	40.1	48.2
150						10.6	14.3	19.6	25.5	32.6	40.1	48.3	57.6
175						11.6	16.7	22.8	29.7	37.9	46.7	56.2	67.3
200					8.5	13.3	19.1	26.1	34.0	43.3	53.5	64.4	77.0
225				7.1	9.5	15.0	21.5	29.4	38.2	48.7	60.1	72.3	86.7
250				7.4	10.6	16.7	24.0	32.6	42.5	54.0	66.9	80.5	96.1
275				8.1	11.7	18.3	26.3	35.9	46.7	59.6	73.5	88.4	105.8
300			5.7	8.9	12.7	20.0	28.7	39.1	51.0	65.0	80.3	96.6	115.5
350			6.6	10.4	14.9	23.4	33.4	45.7	59.5	75.7	93.7	112.7	134.6
400		4.2	7.6	11.9	17.0	26.7	38.2	52.2	68.0	86.6	107.1	128.8	154.0
450		4.8	8.5	13.4	19.1	30.0	43.0	58.7	76.5	97.4	120.5	144.9	173.1
500	2.8	5.3	9.5	14.9	21.2	33.3	47.8	65.3	85.0	108.3	133.9	161.0	192.5

When testing 4-in. house laterals with sewer main, add 2.8 minutes to test time.

to  $\frac{2}{3}$  of the sewer diameter. The specifications should require that the base section be supplied with an extended base, that a boot be cast in the section for all sewer connections and that the channels and bench be cast as a part of the section. All sewers entering a manhole should be provided a smooth channel into the main channel. When there is a change in sewer diameter at a manhole, the 0.8 depths of the sewers should be placed at the same elevation. This will prevent any upstream surcharging at full flow. The change in flow direction at a manhole should not exceed 90 degrees. Where a greater change in direction is required, use two manholes with a segment of sewer between them.

In the past it has been common practice to allow for a 0.1-ft drop in sewer invert through a manhole where there is no change in pipe diameter. This is no longer necessary; in fact, the change in slope and, hence, velocity is more likely to cause problems. A well-constructed channel will not require this drop. Many manhole suppliers now provide the base section with the channel and bench cast in place when the manhole was formed. The channel invert must be smooth, having the same shape as the sewer. In all instances, the penetrations (openings in manhole wall for sewers) should be cast in the manhole or core drilled where

an additional penetration is needed. A penetration made with a jackhammer should never be permitted. A short section of sewer is used at the manhole to provide a joint in the sewer not more than 3 ft from the manhole. This allows some flexibility for any difference in settlement between the manhole and the sewer.

An air or water test is used to check the manhole for water tightness, and this requirement should also be included in the specifications.

The manhole frame and cover should be of gray cast iron. A typical frame and cover is shown in Figure 25.16. A standard frame and cover as well as a watertight/locking frame and cover are also shown in the figure. The engineer's library should include a catalog on standard manhole castings. The weight of the frame and cover must be selected to carry the expected loading. For example, a traffic bearing frame and cover is required when the manhole is located in a street, whereas a lighter weight one will be suitable for off-street locations. Locking frame and covers are available for use where required. Waterproof frame and covers should be used at locations where the area is subject to flooding. Ventilation to the sewer is provided through the manhole cover. When waterproof frame and covers are

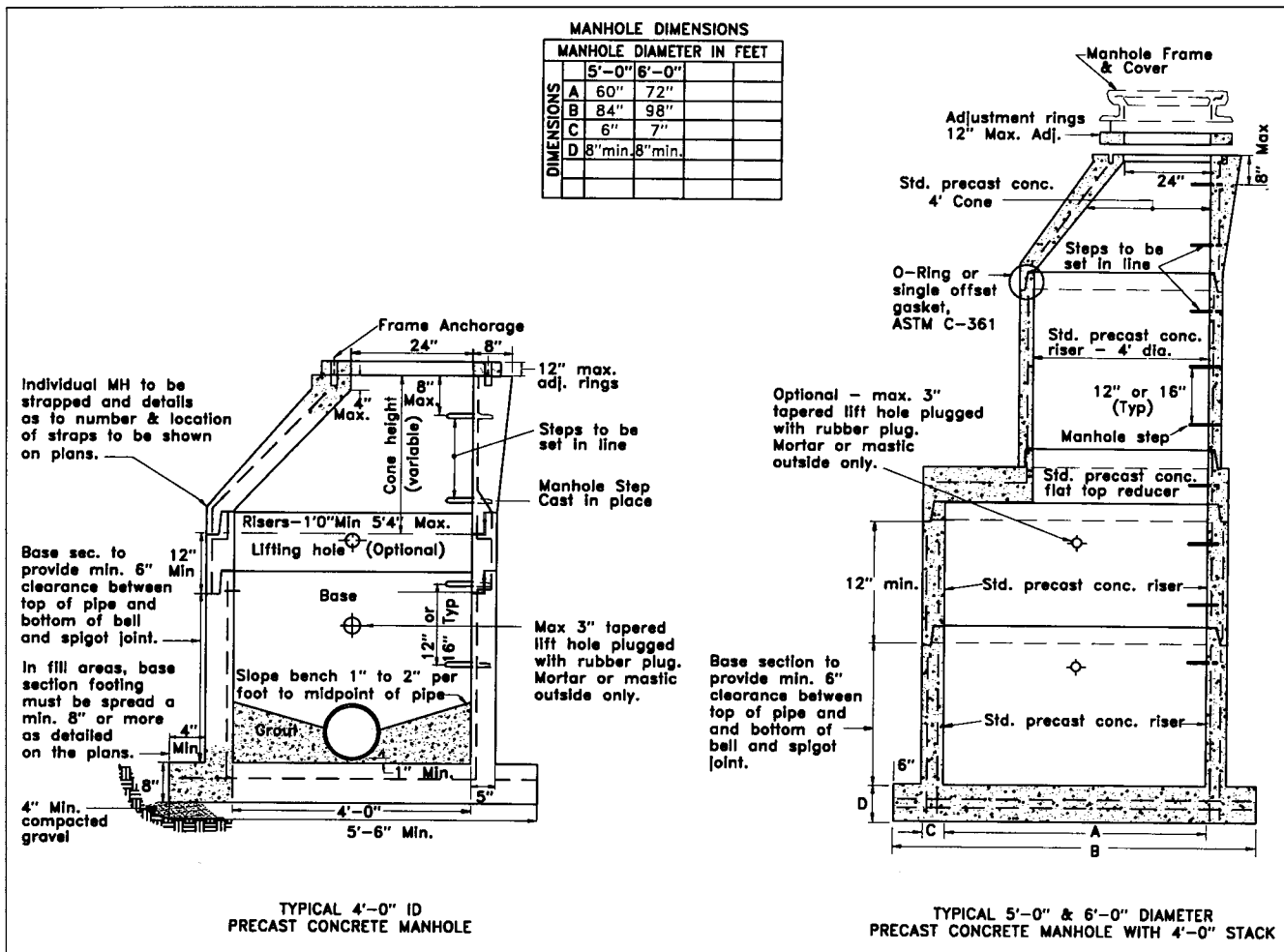


FIGURE 25.14 Typical precast manhole design.

used, alternate ventilation should be provided at least every 1000 ft of sewer.

Many utilities have a specific design on the manhole covers, such as the name of the utility. These utilities stock the covers and sell them to the contractor.

**Building Spurs**

When manholes are placed in the street, the building spur (often referred to as building lateral) should be installed from the sanitary sewer to a minimum of 1 ft inside the property line. A separate spur should be connected for each lot or building site. Where sidewalks are to be constructed, the building spur should be constructed to 5 ft beyond the back of the sidewalk. It is important that each building spur be shown on the sewer plan with the station of the connection being confirmed as a part of the as-built drawings. SDR-35 or heavier pipe should be used for building spurs. The spur should enter the main sewer through a manufactured wye or tee. An approved saddle may be used when connecting to existing sewers. Some localities require that the connecting wye or ell be of ductile iron because mechanical rodding equipment will bore through a PVC con-

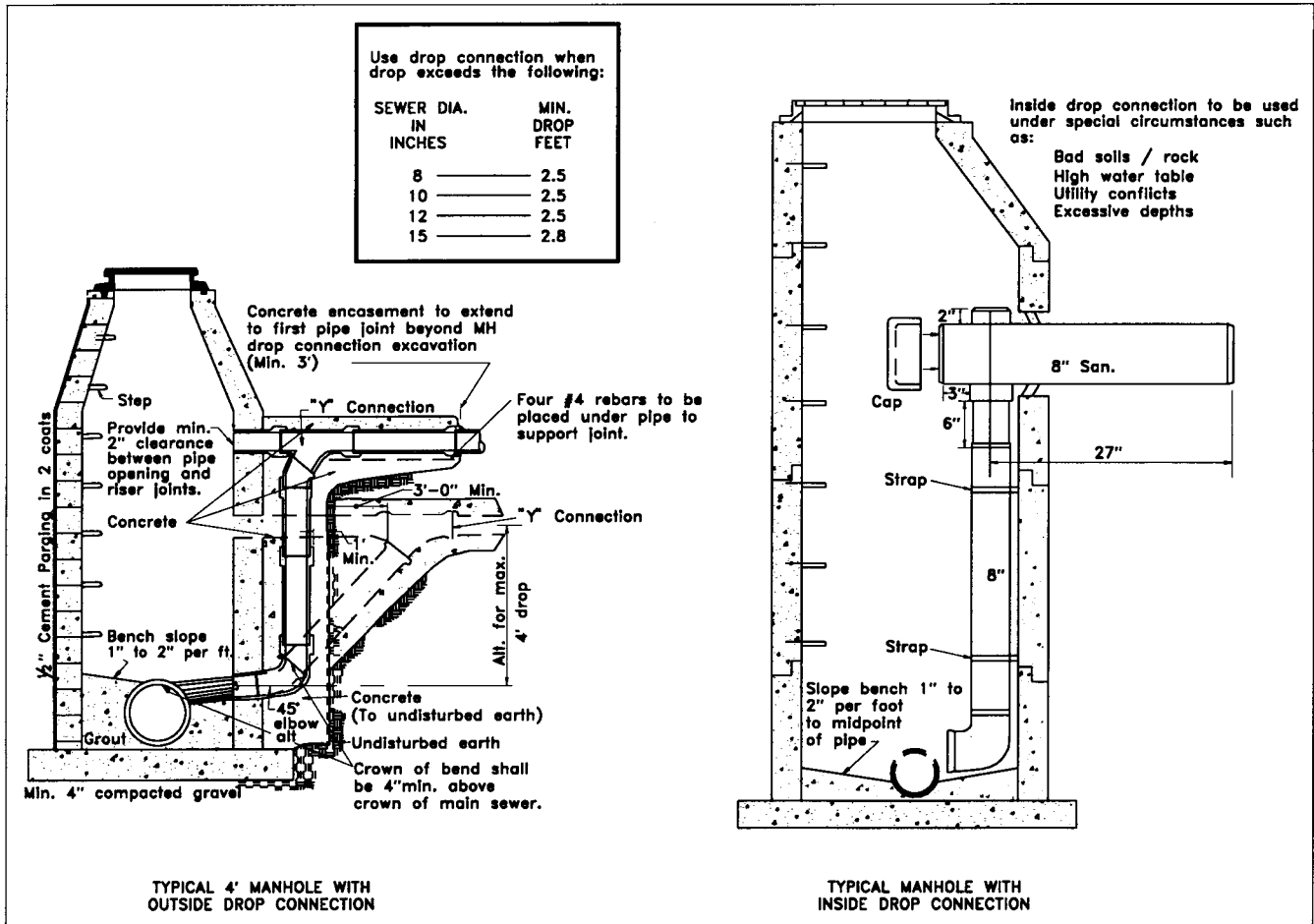
nection if care is not used. Building spurs should be laid to a grade of at least 0.005 slope, with a minimum slope of 0.01 being provided where possible.

A wye may be installed and the extended line capped at the surface at the lot line for access in the future as needed. Where the main sewer is excessively deep, the spur should be brought to a reasonable depth prior to reaching the lot line, but the spur must be kept deep enough to serve the building. Figure 25.17 shows typical connections for building spurs (Service Connections).

**DESIGN EXAMPLE**

**Preliminary Investigation**

This design example has been structured to include the principal considerations that the engineer may encounter with a sewer design project. A hypothetical example, such as presented here, is considered more appropriate for this purpose than an actual design because a hypothetical case can be structured to incorporate the circumstances needed to cover many aspects of sewer design. The example assumes that a Mr. John Jenn, the owner of 40 acres of land



**FIGURE 25.15** Typical drop manhole design.

fronting on West End Road, shown in Figure 25.18, has commissioned an engineering firm to design the infrastructure and obtain the required approvals so that construction permits for a single-family development on the entire tract can be obtained. This example covers the course of action the engineer may take to complete the sewer design and obtain the required approvals. Note that the engineering firm is responsible for providing the owner with complete plans and specifications that have been approved by all reviewing agencies, both local and state. When this has been properly accomplished, the contractor should not have any problems in obtaining a permit for the construction.

The first stage of the project is to conduct a preliminary investigation and prepare a report on the findings. The preliminary investigation should include:

1. Securing topographic mapping of the drainage shed that includes the 40-acre tract. In most instances, the local government will have area-wide mapping. Other sources of maps include: U.S. Geological Survey (7.7-in. Quadrangle Maps); Highway or Transportation Department; tax maps (will not show topographic information); local utilities, including water, gas, electric;

and photogrammetry or mapping firms providing services in the area.

2. Prepare a drawing showing the location of the 40-acre tract, the drainage shed and environs.
3. Meet with the staff of the sewer utility (may be Department of Public Works or a separate authority) to advise them of the project and to obtain information on the availability of public sewer to serve the area. The engineer should always advise the appropriate department of the local government about the project before starting design. This early meeting is also necessary to obtain information from the public utility on how the utility plans to sewer the area, as shown by the Master Utility Plan for the area where the 40-acre tract is located.

It was learned at this meeting that the parcel of land is in an area approved to receive public sewer and that the nearest public sewer is approximately 16,000 ft to the west. The engineer was advised that the utility's long-range plans provide for bringing sewer to the area but that no funds have been budgeted for the project. While capacity is avail-



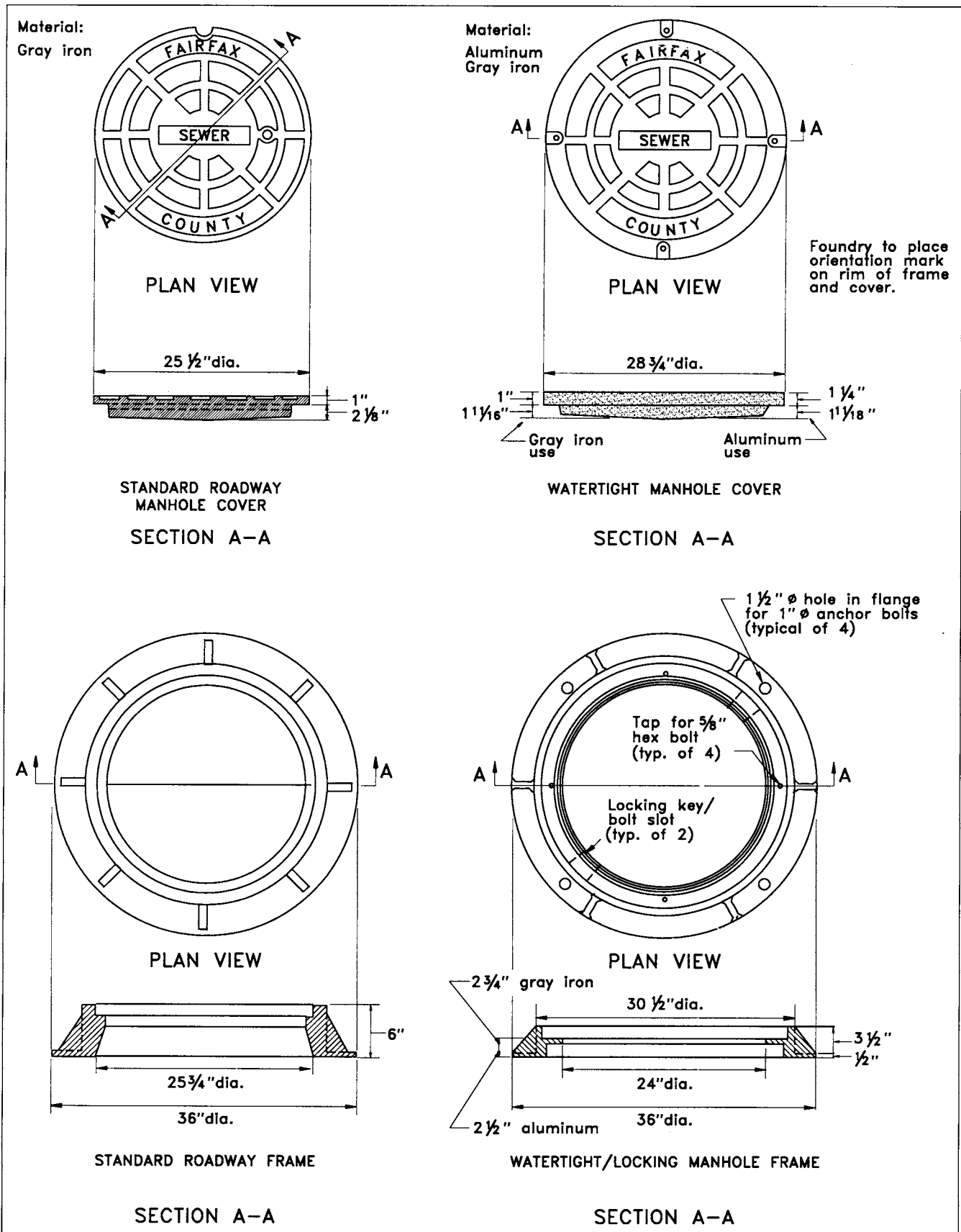
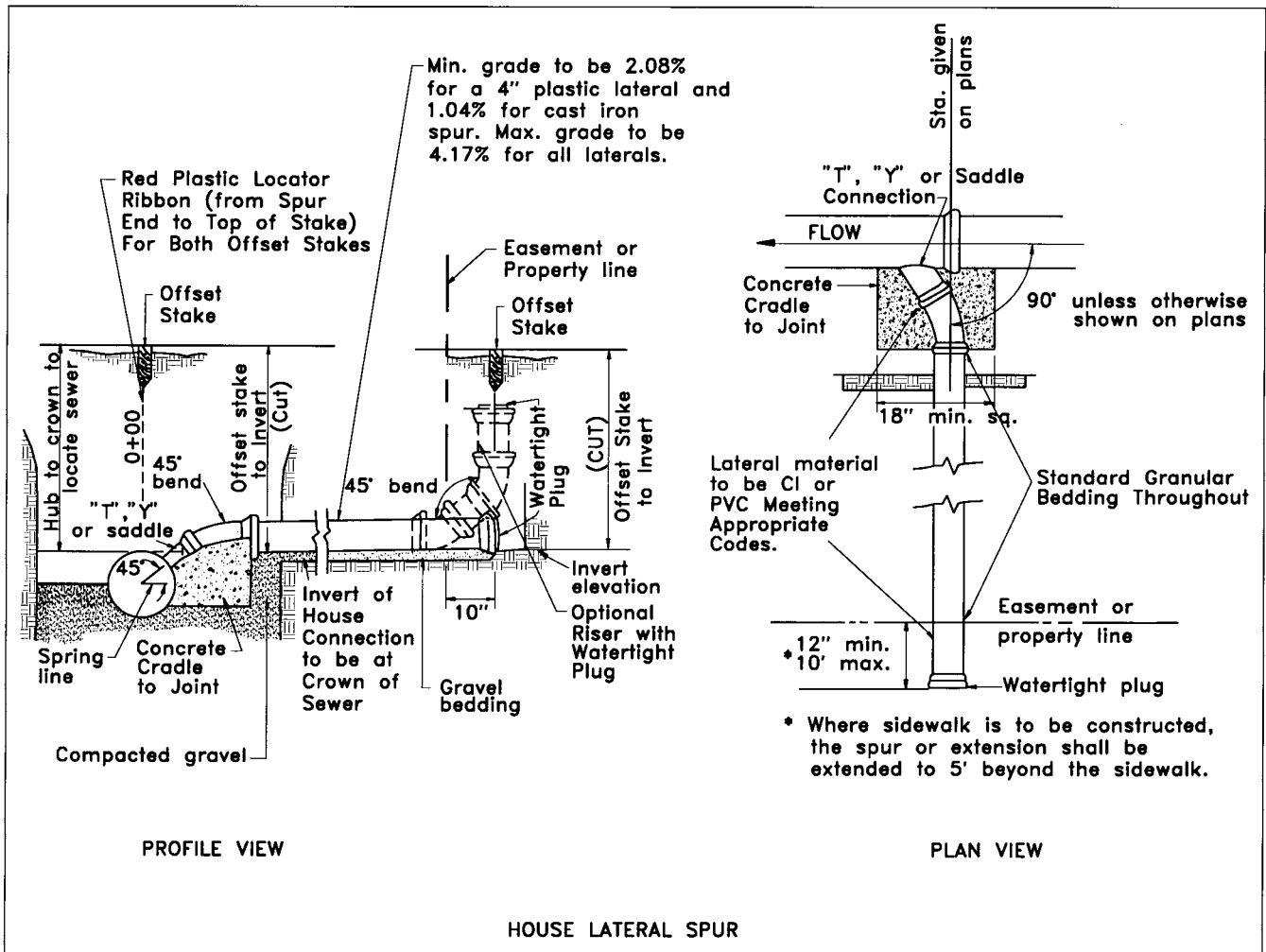


FIGURE 25.16 Typical manhole frame and covers. (Fairfax County Public Facilities Manual, 1993).



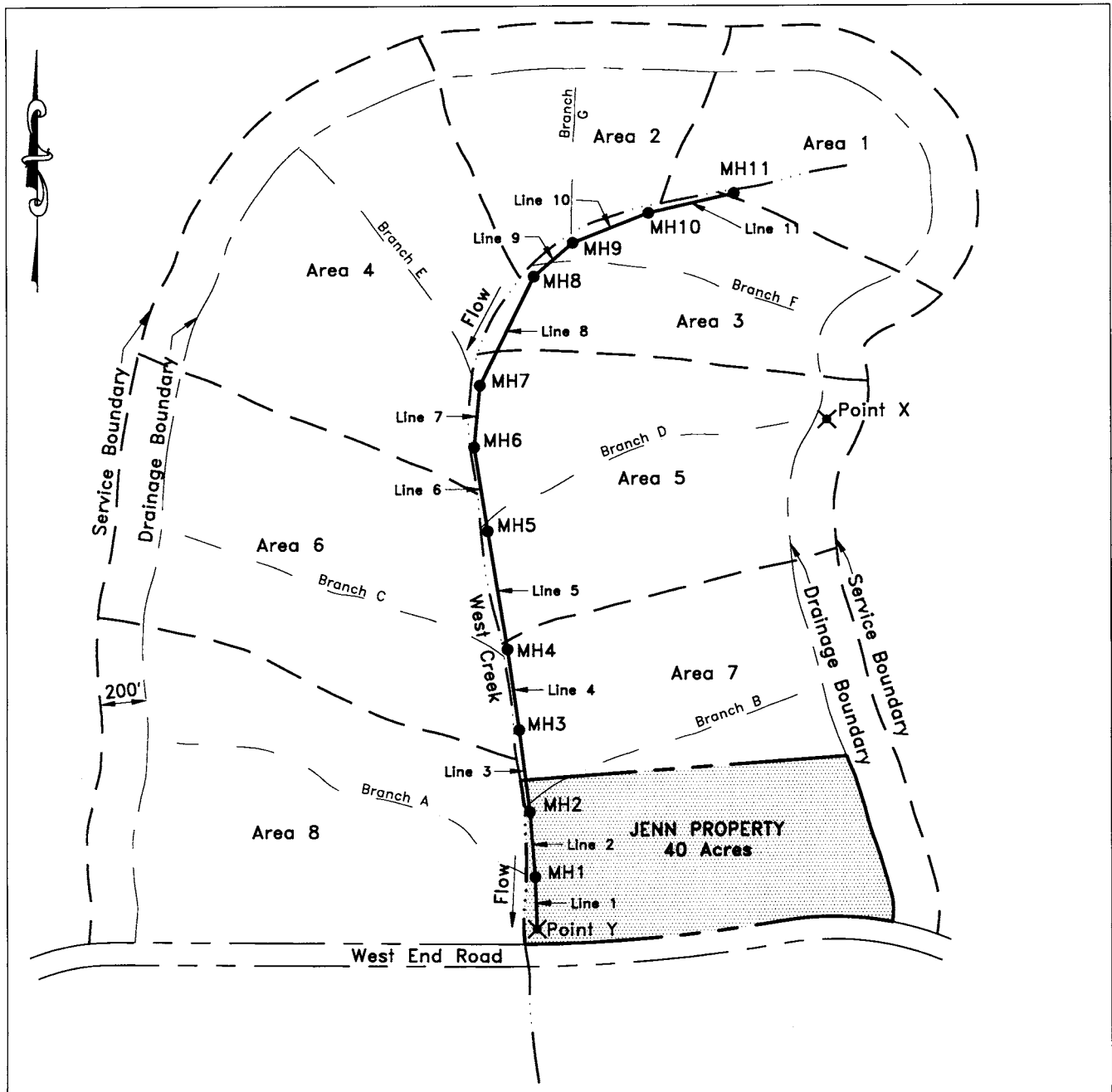
**FIGURE 25.17** Service connections showing building spurs or laterals.

able in a trunk sewer located to the west, the engineer was advised that it may be 5 to 10 years before service will be available to the property under the utility's sewer extension program. The utility does permit property owners to extend sewer service into areas approved for service at the property owner's expense. The utility advised the engineer that any design for sewerage the parcel will have to consider the needs of the entire sewer shed. The utility also advised that the comprehensive sewer plan provides for the construction of a pumping station on the adjacent sewer shed to the east for pumping the flow across into the drainage area containing the 40-acre tract. This additional 28-acre parcel is zoned single-family at four units per acre. Information on the location of the connection point for the proposed pumpover is provided. The engineer was advised that the property south of West End Road is not planned for public sewer. A pumping station is to be located at Point Y (Figure 25.18) and a force main routed along the north side of West End Road to the existing trunk sewer.

The utility has a reimbursement policy whereby if Jenn, the owner of the 40-acre tract, constructs facilities that will

serve off-site property, the utility will collect prorated funds from other land owners as their property is approved for sewer and pass these funds along to Jenn as they are collected. Jenn will only be entitled to reimbursement for any over-sizing of utilities as required to provide service for off-site property. Note that this is only one procedure for managing a reimbursement policy as the local policy may provide for reimbursement on the basis of acreage sewerage or by some other procedure. A reimbursement policy based on flow will generally be more equitable where more than one zoning applies to the area to be sewerage. The engineer should be knowledgeable in the policy for the jurisdiction where the project is proposed. The reimbursement policy may be different or there may not be one. If Jenn eventually receives full reimbursement, he will have only been responsible for the costs of the initial size facilities required to serve his property with the increased costs for over-sizing to serve off-site property qualifying for reimbursement.

Early in the project, a meeting between the engineer and the utility staff is very beneficial to both parties and should always be scheduled. The utility staff is now apprised of the



**FIGURE 25.18** Sewer shed West End Road service area.

project and the engineer has learned that the tract does lie within an area approved for sewer under the comprehensive planning and zoning for the area. The engineer has also been advised of off-site requirements that will need to be incorporated into the planning and design of the sewer system that will serve the 40-acre tract.

The cost of off-site construction in this example may be more than would be economically feasible in the development of a 40-acre parcel. The principal off-site costs will be that of a pumping station and 16,000 ft of force main. The engineer is able to apprise the owner of these costs as a

part of the preliminary report. This report is completed and presented to the owner before starting design. Thus, the owner is able to make financing decisions before incurring substantial engineering costs. Project financing is not covered in this chapter, but note that the property owner has several options that should be explored. The owner or the owner's legal counsel should meet with other property owners in the sewer shed to determine if they are interested in participating in the project. The engineer may also assist the owner by sending a letter to each affected property owner advising them of a meeting to be held at the engi-

neer's office for the purpose of determining their interest in participating in the project. It would not be uncommon to find that the other property owners have been wanting sewer service for some time and will be willing to participate in the off-site costs. The reimbursement agreement should be between the local government and the property owners, with the local government being responsible for collecting and disbursement of funds. A reasonable rate of interest should be added to the reimbursement amount and passed along to Jenn. The engineer would generally not be authorized to proceed with design until Jenn has made suitable financing arrangements for the construction of the project.

### Establishing Sewer Service Shed

A drawing of the sewer shed is shown in Figure 25.18. The drainage boundary above West End Road is found to be 255.26 acres. This is a relatively small drainage shed, and the utility has advised the engineer that the planning must include the entire shed as being developed to the limits allowed under the comprehensive plan for the area. It should be recognized that for a rural area, a 255.26-acre area may require many years for complete development, in which case something less than the entire drainage shed would be included in the planning and design associated with sewerage the 40-acre tract. This is a decision that should be agreed upon by the local government planners and the utility. Decisions of this type should be arrived at by determining what the needs of the area are projected to be over the next 20 to 25 years. It is not good practice to over-design a facility to where the capacity will not be used for many years, nor is it appropriate to be too short-sighted in planning to where a parallel sewer will be needed in five to ten years. In this example, it is assumed that the parcel is located in a major urban area where development will occur within a 10-year period once sewer service becomes available.

Contours are not shown on Figure 25.18 for clarity purposes. The figure is shown at a 1 in. = 500 ft scale in order to include the entire area on a normal sheet. The engineer would normally use a 24-in. by 36-in. layout sheet. This would permit using a scale of 1 in. = 200 ft. At this scale, a 5-ft contour interval would be used, unless the terrain is extremely flat or steep.

The 255.26-acre sewer shed above West End Road has eight subsheds, each having an unnamed intermittent stream draining to West Creek. These intermittent streams are shown as Branches A through G on the figure. It is generally possible to gravity-sewer some property outside the drainage boundary without having excessively deep sewers to cross the drainage divides. In the absence of any local government regulation governing this practice, the engineer should place a limit on the depth of sewer that will be used to sewer property beyond the drainage boundary limit. It is recommended that a maximum depth of 20 ft be used for this purpose.

A 200-ft-wide strip has been used in the example to establish a sewer service boundary. This 200-ft-wide strip extends around the three sides of the drainage boundary. Lines are drawn to show the drainage limits of the eight subsheds. Also shown on the figure is the point where it is proposed that the off-site sewer discharge to the West Creek shed. This is shown as Point X.

The engineer must determine how the area will be sewerage. In this example, it is determined that the area will be sewerage by constructing a main sewer along the east side of West Creek from Point Y to Area 1. Submains will connect at the confluence of each of the Branches with West Creek. Each of the submains will sewer the respective area with no area connecting directly to the main except through the submains. This type of decision will be made by the engineer and will be based on the topography. In this example, the flow for the main sewer is computed. The design of the submains and the sewer laterals would be designed as a part of the development of each parcel.

### Determining Average and Peak Flow

A topographic map should be prepared for the sewer shed, showing the boundaries for each planned land use. This information is taken from the comprehensive plan for the area. The area of each use for each subshed is then determined by planimetry. The area for each subshed, as determined by planimeter, is shown in Table 25.11. The planned density for each subshed is shown in Table 25.12.

A drawing similar to Figure 25.18, at a scale of not smaller than 1 in. = 200 ft, would be prepared showing all topographic features, including contours and land use.

**TABLE 25.11 Acreage for Each Subarea as Shown in Figure 25.18**

AREA	ACRES*	ADDITIONAL ACRES**	TOTAL ACRES
1	15.50	8.61	24.11
2	16.07	5.68	21.75
3	21.12	4.24	25.36
4	36.39	9.41	45.80
5	37.37	3.85	41.22
6	37.54	5.17	42.71
7	49.65	8.15	57.80
8	41.62	6.95	48.57
Totals	255.26	52.06	307.32

\* Acres within the drainage boundary.

\*\* Acres within the additional 200-foot-wide strip.

**TABLE 25.12 Planned Density**

AREA NUMBER	PLANNED DENSITY
1	100% single-family at 4 units/acre
2	40% townhouses at 10 units/acre 60% new elementary school—300 students
3	100% single-family at 4 units/acre
4	100% single-family at 4 units/acre
5	100% single-family at 4 units/acre
6	100% townhouses at 10 units/acre
7	100% single-family at 4 units/acre
8	40% commercial at 2000 gpd/acre 60% apartments at 25 units/acre

A map of this type is not provided in this example because of size limitations.

Table 25.13 is formatted for tabulating average and peak flow for each main sewer segment. The length of each segment is shown in column 3 of the table. The peak design flow is provided in both gpm and cfs for general information.

Table 25.14 provides information on elevations for the proposed sewer route and the associated West Creek streambed. This table would not be required in a normal design as the information would be taken directly from the topographic map data as the design proceeds.

General guidelines to be used in the sewer design include:

1. It is desirable to keep the sewer depth to at least 6 ft. This will minimize the impact of surface loadings.
2. Design as to allow for stream crossings. The top of the sewer for all stream crossings is to be at least 1 ft below the stream bed. The sewer, in the area of the stream crossing, will be concrete encased.

### Line Design

The sewer design is provided in Table 25.15. Notes are provided as a part of the table to show how the data was determined. In the design example, the design is started adjacent to the proposed pumping station and continued upstream. When designing sewers to serve relative flat terrain, where the ground slope along the sewer route is less than the slope required to provide minimum velocity in the sewer, the design should be started at the upper end of the system and continued downstream. This is because the sewer will continue to get deeper as the design proceeds.

If the design is started at the lower end, the designer does not know what initial depth to use so that adequate cover will be provided upstream, making several trial-and-error attempts necessary.

### Preparation of Construction Plans

The engineer now has all the information required to complete a set of construction drawings. It is important that the plans be clear and complete. These plans would normally be prepared on 24-in. by 36-in. sheets (local utility may have other requirements) and consist of:

- A cover sheet showing the project title, any identifying project number, a location map, and the name of the engineering firm. The cover sheet also often includes a sheet index for the drawings and the name of the owner. If the owner is a public body, the names of the elected officials such as mayor and council members may be shown as local practice requires.
- A general layout sheet showing streets, lots and sewer location. The layout sheet should be to a scale where the entire project can be shown on a single sheet. The design sheets should be overlain on the layout sheet as an index.
- Plan and profile sheets. These sheets normally show the design in plan view on the top half and in profile on the lower half. All topographic data related to the design construction area should be shown. The manholes are shown and numbered. The lines are shown with bearings and distances with any angles being shown at changes in horizontal alignment. Sewer size, manhole numbering and stationing is shown on both the plan and profile views. The plan view should show existing and proposed structures, houses, underground utilities, curbs, property lines, all storm drainage, and any other infrastructure. The profile shows the elevation of all critical points such as all manhole inverts and tops of covers to a hundredth of a foot. Care must be taken to show the location and elevation of all subsurface utilities, foundations, and other elements important for safe, orderly, and economical construction. Field test pits may be needed for the proper location of existing subsurface elements.
- The plan view is generally shown at a scale of 1 in. = 50 ft. The horizontal scale for the profile is shown at the same scale as the plan. The vertical scale for the profile is generally shown at a scale of 1 in. = 5 ft.
- Standard details sheets. Standard details for manhole, manhole frame and covers, special construction, such as waterproof frame and covers and stream crossings, are shown on sheets as needed following the plan and profile sheets.

TABLE 25.13 Computation of Sewage Flow

(1) FROM POINT*	(2) TO POINT*	(3) LENGTH FEET	(4) AREA, ACRES ADDED	(5) TOTAL ACRES	(6) DENSITY ADDED	(7) ADDED AVE. FLOW	(8) TOTAL FLOW	(9) PEAK FACTOR	(10) DESIGN FLOW	(11) DESIGN FLOW
8	7	700	1(24.11)	24.11	96 S.F.	35,520 gpd	35,520 gpd	6.4	158 gpm	0.35 cfs
7	6	220	2(21.75)	45.86	87 T.H.	26,100	61,620	6.0	257 gpm	0.57
6	5	470	3(25.36)	71.22	101 S.F.	37,370	98,990	5.9	406 gpm	0.90
5	4	680	4(45.80)	117.02	458 T.H.	137,400	236,390	5.4	886 gpm	1.97
4	3	470	5(41.22)	158.24	164 S.F.	60,680				
			X(28.0)	186.24	112 S.F.	41,440	338,510	5.1	1199 gpm	2.67
3	2	670	6(42.71)	228.95	427 T.H.	128,100	466,610	4.9	1588 gpm	3.54
2	1	290	7(57.80)	286.75	231 S.F.	85,470	552,080	4.7	1802 gpm	4.01
1	Y	250	8(48.57)	335.32	Commercial 728 Apts.	38,856 218,550	809,486	4.5	2530 gpm	5.63

\*Note: Points refer to confluence of subshed drainage with West Creek and correspond with area added as shown in column 4.

## Notes:

1. Allowable density obtained from comprehensive plan of local government (see Table 25.12).
2. Acres taken from mapping by planimeter (see Figure 25.18 and Table 25.11).
3. Areas are totaled to confluence of indicated streams.
4. Average flow per unit obtained from Table 25.2.
5. Flow added is unit flow times the number of units.
6. Total flow is total to indicated downstream point.
7. Peak Factor obtained from Figure 25.2.
8. Design flow is obtained by multiplying average flow times the peak factor to obtain gpd and then dividing by 1440 minutes/day to give flow in gpm. Cfs is obtained by dividing gpm by 449 gpm/cfs. MGD is obtained by dividing gpm by 695 gpm/mgd or cfs divided by 1.55 cfs/mgd gives the flow in mgd. Cfs or gpm can then be used with the nomograph shown on Figure 25.7 to establish head loss and pipe size.
9. S.F. = Single-family detached housing.
10. T.H. = Town house.
11. Note that where the distance between points is greater than 400 ft, an intermediate manhole is needed. See the design tabulations shown in Table 25.15 for more information on how this was handled.

■ Submain and building spur connections. In subdivisions it is common to locate the sewer near the middle of the street where possible and construct all building spurs to the property line and cap the end of the line. It is important to show the location and elevation of each spur on the drawings for later location. The spur should enter the sewer through a manufactured wye. House spurs generally do not connect to a manhole. Submains should enter the main at a manhole where the connection has been cast as a part of the manhole.

Note that the sewer design results in establishing the sewer invert profile. The sewer conduit is uniform and at steady flow conditions. The invert (lowest point inside the sewer conduit) is parallel to the hydraulic grade line (HGL). While the HGL will rise and fall with changes in flow conditions, it will always be within the pipe under gravity flow conditions. Since the HGL location varies, the invert slope

is established in design and utilized in constructing the sewer.

## PUMPING STATIONS

### General

Pumping stations are common to most sewage collection systems. The collection system that does not include lift or pumping stations at one or more locations is rare. The engineer has access to a wide choice of pump types, giving flexibility in selecting the type of facility most suitable for a particular application.

The term "lift station" refers to a pumping facility located within a collection system to lift the flow to a higher elevation. Sewer systems located in relative level terrain may become excessively deep, making it necessary to "lift" the flow up to minimum cover for the sewer. Note in Table 25.5, the slope of an 8-in sewer providing a velocity of 2.25 fps is 0.0042. This sewer has a head loss and, hence, a drop

**TABLE 25.14 Elevations Determined from Field Surveys**

STATION	GROUND ELEV. (ft)	STREAM INV. ELEV. (ft)	STATION	GROUND ELEV. (ft)	STREAM ELEV. (ft)
0 + 00	212.0	206.8	19 + 00	221.6	217.4
1 + 00	212.0	207.8	20 + 00	222.0	218.0
2 + 00	213.2	208.2	21 + 00	222.5	218.7
3 + 00	213.7	208.9	22 + 00	223.1	219.3
4 + 00	214.4	209.3	23 + 00	223.6	219.9
5 + 00	215.0	209.7	24 + 00	224.0	220.5
6 + 00	215.9	210.1	25 + 00	224.7	221.0
7 + 00	216.4	210.4	26 + 00	225.2	221.4
8 + 00	216.6	211.0	27 + 00	225.8	222.1
9 + 00	216.8	211.6	28 + 00	226.5	222.9
10 + 00	217.2	212.3	29 + 00	227.0	223.5
11 + 00	217.4	213.0	30 + 00	227.6	224.2
12 + 00	217.9	213.3	31 + 00	228.2	224.8
13 + 00	218.5	213.7	32 + 00	228.9	225.4
14 + 00	218.9	214.3	33 + 00	229.5	226.2
15 + 00	219.6	214.9	34 + 00	230.1	227.0
16 + 00	220.1	215.5	35 + 00	230.7	227.5
17 + 00	220.7	216.2	36 + 00	231.2	228.2
18 + 00	221.0	216.8	37 + 00	231.7	228.0

**Notes:**

1. Ground elevations were taken from field topo for the selected sewer route.
2. Stream elevations are for the lowest point in the stream cross section at the respective station along the sewer route.
3. The sewer route is within 25 feet of the stream at all points.
4. Station 0 + 00 is adjacent to proposed location of pumping station.

**General Guidelines to be used in the sewer design**

1. It is desirable to keep the sewer depth to at least 6 feet to minimize impact of surface loadings.
2. Design is to allow for stream crossings. Top of sewer is to be at least 1 foot below stream bed. Stream crossings will be concrete-encased.

in the sewer invert of 4.2 ft per 1000 ft of length. If the sewer is long or if the sewer is not running with the ground slope, the depth may become so deep that a lift station is needed. The depth where a lift station becomes economical depends on subsurface conditions. The presence of a high ground water table, poor soil or rock must be considered in making this determination. Also, the length of sewer that will be excessively deep must be considered. Utility maintenance personnel generally prefer that sewers not be over 20 ft deep if greater depths can be avoided.

A pumping station is a facility designed to lift the flow and convey it some distance, such as from one drainage area to another or from the collection system to a treatment works. A lift station is commonly referred to as a pumping station, but pumping stations are not called lift stations.

**Location**

A pumping station should be located so as to serve the entire sewer shed. In the sewer design example in the previous section, it was found as a part of the preliminary

TABLE 25.15 Sewer Design Tabulation

(1) LINE	(2) FROM	(3) TO	(4) LENGTH	(5) PEAK FLOW	(6) GROUND LOWER	(7) ELEV. UPPER	(8) STREAM ELEV.	(9) SEWER DIA.	(10) VEL.	(11) SLOPE	(12) HEAD LOSS	(13) Q FULL	(14) SEWER LOWER	(15) INVERT UPPER	(16) CON- TROL
1	Y	1	250'	5.63	212.0	213.4	208.6	15"	4.1	0.0056	1.40	5.0	204.75	206.15	Cover
2	1	2	290	4.01	213.4	215.5	209.9	15	3.9	0.0052	1.50	4.8	206.15	207.65	Stream
3	2	3	340	3.54	215.5	216.7	211.5	15	3.7	0.0047	1.60	4.6	207.65	209.25	Stream
4	3	4	330	3.54	216.7	217.9	213.4	15	3.6	0.0042	1.40	4.5	209.25	210.65	Cover
5	4	5	470	2.67	217.9	220.6	216.0	12	3.6	0.0059	2.75	2.9	210.85	213.60	Cover

## Notes:

Column 1 = Number of sewer segment—generally numbered consecutively upstream.

Column 2 = Number of manhole at downstream end of sewer segment.

Column 3 = Number of manhole at upstream end of sewer segment.

Sewer manholes are numbered consecutively upstream. Submains connecting to the main numbered from that point. For example, a submain or lateral connecting to the main sewer at manhole 5 would have manhole numbers of 5-1, 5-2, etc.

Column 4 = Length of sewer segment measured from center to center of manholes.

Column 5 = Peak flow calculated for sewer segment (see Table 25.13).

Column 6 = Ground elevation at downstream end of sewer segment.

Column 7 = Ground elevation at upstream end of sewer segment.

Note that the ground elevation is carried to the nearest 0.1 ft, whereas the sewer invert is carried to the 100th of a ft.

Column 8 = Stream elevation at upstream end of sewer segment. Must be able to cross stream with one ft of cover over top of pipe. Column 16 is added to show if stream elevation is controlling.

Column 9 = Diameter of sewer segment in inches.

Column 10 = Velocity of flow in sewer segment.

Column 11 = Slope of invert for sewer segment.

Column 12 = Feet of head loss in sewer segment.

Column 13 = Full capacity of sewer.

Column 14 = Sewer invert elevation at downstream end of segment.

Column 15 = Sewer invert elevation at upstream end of segment.

Column 16 = Condition controlling depth. This column is not shown on sewer design tabulations, but included here for instructional purposes. The design criteria employed in this example is that the sewer cover is to be at least 6 ft and that at least 1 ft below the stream invert. The invert of a sewer is the lowest elevation on the inside of the pipe. The invert elevation is maintained in sewer construction and design.

## Design procedure:

A. For line 1 running from manhole Y to manhole 1.

1. Check ground slope for segment. Difference in elevation is found to be 1.4 ft and slope is head loss divided by segment length =  $1.4/250 = 0.0056$  as shown in column 11.

investigation that the limit of the sewer shed was to be West End Road. This may have been a political boundary or a decision made by the local government because of land planning. The station must be located so as to receive the sewage flow from the designated sewer shed. Some general considerations to be used in locating a pumping station include:

- A lift station or pumping station should never be located in a public street.
- The station must be accessible by an all-weather service road, where the utility either owns the right-of-way or has a permanent access easement.
- The station must be located to provide protection from flood inundation. The station should, as a minimum, remain fully operational at the 25-year flood ele-

vation and receive no damage at the 100-year flood level. The criteria to be used in determining flood protection depends on local policy and conditions related to the site, such as the potential for unacceptable environmental pollution when overflows or sewer backups occur. In no case should sewer backup due to pump station failure result in flooding of buildings with sewage.

- Consideration should be given to problems with noise and odor. Odor is most likely to be a consideration with flow from large, long main or trunk sewers where the sewage becomes septic in the sewer before reaching the pumping station. The discharge from a force main will also usually be septic. Equipment is available for removing odors from exhaust air. Screenings are another source of odors, and they should not



TABLE 25.15 Sewer Design Tabulation (Continued)

(1) LINE	(2) FROM	(3) TO	(4) LENGTH	(5) PEAK FLOW	(6) GROUND LOWER	(7) ELEV. UPPER	(8) STREAM ELEV.	(9) SEWER DIA.	(10) VEL.	(11) SLOPE	(12) HEAD LOSS	(13) $Q$ FULL	(14) SEWER LOWER	(15) INVERT UPPER	(16) CON- TROL
6	5	6	340	1.97	220.6	222.1	218.2	12	3.1	0.0044	1.50	2.5	213.60	215.10	Cover
7	6	7	340	1.97	221.1	223.9	220.3	12	3.4	0.0053	1.80	2.8	215.10	216.90	Cover
8	7	8	470	0.90	223.9	226.7	223.1	8	2.6	0.0054	2.54	0	217.16	219.70	Cover
9	8	9	220	0.57	226.7	227.9	224.5	8	3.0	0.0070	1.53	1.1	219.70	221.23	Cover
10	9	10	350	0.35	227.9	230.1	227.0	8	2.8	0.0063	2.20	0.9	221.23	223.43	Cover
11	10	11	350	0.35	230.1	232.0	228.2	8	2.7	0.0054	1.90	0.93	223.43	225.33	Cover

2. Manning's  $n$  is 0.013 (generally always used for design).
3. Knowing  $n$  and  $S$ , from Figure 25.7 using a straight edge, the two points will establish a point on the pivot line.
4. Maintaining the pivot point, rotate the straight edge to the required  $Q$  as shown in column 5. Note that the straight edge falls between 12 in and 15 in pipe diameter. The straight edge is rotated about the pivot point to the next larger pipe diameter of 15 in. At this alignment of the straight edge  $D$ ,  $V$ , and  $Q$  full is established. The most economical condition is to lay the sewer on the same slope as the ground surface. In this example the ground slope is adequate to provide the minimum velocity of 2.0 fps. A 6-ft cover requires that the sewer invert be 6.0 ft + the 1.25-ft pipe diameter, 7.25 ft from the ground elevation at each end of the sewer segment. In some instances it is necessary to check cover at low point between manholes and the designer should always make this check. Columns 9, 10, and 13 data is obtained from the nomograph shown as Figure 25.7. Columns 14 and 15 are based on cover to be provided and head loss as shown by column 12.
5. Check upper end invert for cover at stream crossing when submain is installed.
6. The rules for connecting sewer segments at manholes are:
  - a. The crown of the entering and exiting pipe are at the same elevation.
  - b. The 0.8th depth of the entering and exiting pipe are at the same elevation. This option has been used in this example. Note Manhole 4 where the entering sewer is 12 in and the downstream sewer is 15 in in diameter. In this case the drop in invert elevation across the manhole is 0.20 ft.
  - c. If the crowns had been placed at the same elevation the drop would have been the difference in the diameter of the two sewers or 0.25 ft.
  - d. Past practice has been to provide a 0.10-ft drop across the manhole where there is no change in pipe diameter. This practice is no longer considered to be necessary. If a smooth chsnrl having the same diameter as the sewer is constructed across the manhole when the bench is made, there is no need for this drop. In fact, the change in velocity resulting from a 0.10-ft drop is likely to cause problems. Note in the data in Table 25.15 that no loss is shown across the manhole where there is no change in pipe diameter.
7. In cases where the sewer is at a depth greater than what is needed to provide minimum cover, the sewer slope to be used is generally the slope required to provide the minimum velocity, which is usually 2.0 or 2.25 fps. This permits minimizing the depth of construction. Never use a greater pipe size than what is required to carry the flow. Designers sometimes do this when designing sewers for flat terrain to reduce the slope required to provide the minimum velocity (Note in Eq. (25.7) that the larger the pipe diameter, the smaller the slope required to maintain a velocity). This practice will lead to maintenance problems because the actual velocity of the partially filled pipe will result in the deposition of solids.

be stored at locations where odors will affect the surrounding area. Accumulation of solids in a poorly designed wet well may also cause odors.

Noise will not normally be a problem at a properly designed facility. The noise level from the operation of pumps and motors is low and should not be noticeable outside the station. There will be some minor noise from the maintenance truck and crew that checks the station on a daily basis. If the station design includes on-site emergency power, the generator will need to be exercised weekly, but residential type mufflers are available and should be used. The exercise period is usually one-half hour and should be scheduled during the normal workday.

■ A buffer area of at least 100 ft around the station should be owned by the utility. A cyclone type fence should be installed around the station to minimize problems with vandalism and, more importantly, to reduce the possibility of accidents involving children.

■ The exterior architecture of the station should not detract from the appearance of the neighborhood. A pumping station can be located in any residential neighborhood if it is designed and sited in an attractive manner. Windows should be avoided to reduce vandalism. False windows can be used where needed for architectural reasons. The owner should always be consulted about the exterior appearance of the station. The grounds should be properly landscaped for appearance and drainage. Use slow growing shrubbery, gravel areas and pavement to reduce maintenance requirements.

### Design Types

The engineer should be familiar with the requirements of the utility that will own and operate the station. Utility personnel prefer to have equipment that they are familiar with for ease in maintenance. If the utility staff does not like the type of equipment provided, it probably will not be a suc-

cessful project. It is also important that the equipment be procured through a manufacturer's representative who will be available and responsive if problems occur with the equipment.

A small lift station may serve a few houses, a business, church or school. This type of station can be as simple as placing one or two small pumps in a manhole. It is widely used with submersible pumps, with the manhole housing the pumps serving as the wet well. Small grinder pumps are often used, in which case the force main may be a 1 $\frac{1}{4}$ -in. plastic pipe. Pneumatic ejectors may also be used for small lift stations, but ejectors are generally located inside buildings to lift the flow from floors lower than the public sewer up into the sewer. Ejectors located inside buildings are part of the building plumbing. In most instances, these small lift stations should include two pumps with automatic alternation for station reliability.

Moving up in capacity, the second type of pumping or lift station may be of the same general type as described above, except for the size or capacity of the equipment and the reliability of the station. This size of station would always pump into a force main 4 in. in diameter or greater. The station should always have two pumps and generally have on-site emergency power. The engineer has many options available for selecting the type of pumping facility to be used. However, the submersible station design will normally offer the most economical and functional choice for use in systems serving small communities.

Figure 25.19 shows a design employing two submersible pumps placed in a properly sized concrete manhole. The bottom of the manhole is filleted and sloped to direct the flow to the pump intake and to prevent solids accumulation in the manhole. The pump mounting design provides for the pump to be removed and replaced by a lifting chain without the need to enter the manhole. A lifting hoist is provided as part of the station design. All valves and controls are located in a separate vault for ease in maintenance, thus eliminating the need to enter the wet well. This type of pumping station can be used for designs of any size. Where the average daily flow is greater than 0.5 mgd, the two pumps may be placed in separate chambers that are interconnected with isolation valves so that either chamber may be dewatered for maintenance.

There are several manufacturers of reliable submersible pumps. No alternative offers the simplicity, reliability and economy that is achieved with a submersible pump station. Four-ft diameter manholes are normally used as the wet well in collection systems; however, manholes are generally available in diameters up to 10 ft. When a large-diameter manhole is needed for locations where they are not available, reinforced concrete pipe can be placed vertically on a concrete pad. When concrete pipe is used, the design must connect the pad and pipe together in a manner that will prevent leakage at the joint.

While the submersible pumping station is recommended, there are hosts of "package pump station" designs available. Generally, the entire pumping station is shipped

to the site already assembled and is placed on a concrete pad, connected to a wet well discharge, force main piping, and to an electrical supply.

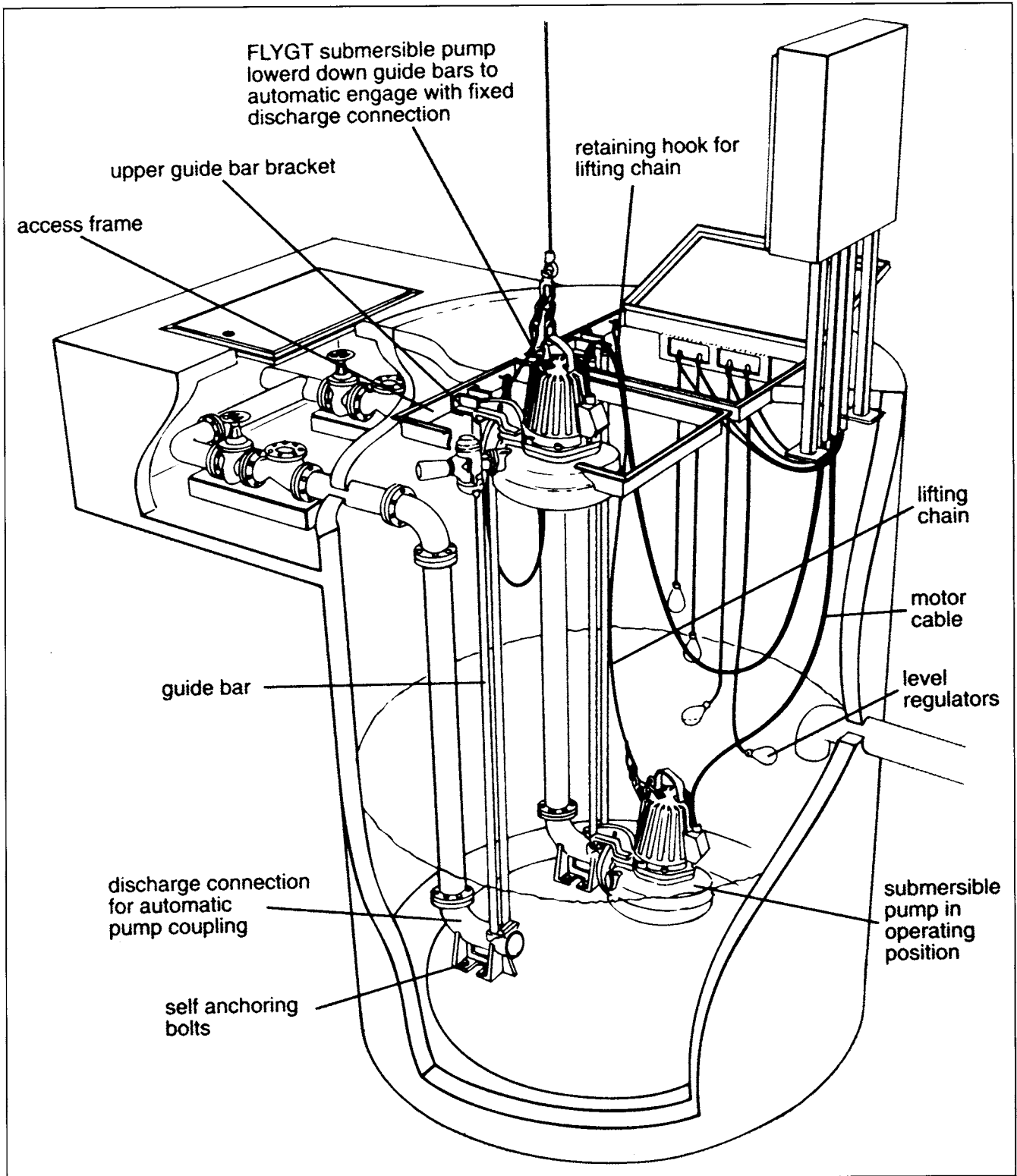
The engineer always has the option of designing a constructed-in-place wet well-dry well design where reinforced concrete basins are constructed and the pumping equipment, piping and associated appurtenances are assembled in place. The wet well is the receiving chamber for the incoming flow. The wet well serves as a flow surge basin and as short-term storage during periods when the pumps are not operating. The dry well is isolated from the wet well except for the pump intake pipes. The dry well houses the mechanical equipment. The electrical equipment, including the pump motors, is generally placed in the ground floor housing located over the dry well. Both the wet well and dry well must be suitably lighted, ventilated and have proper access for maintenance personnel. This type of pumping station can be designed to incorporate all specific requirements of the operating utility, but it is the most expensive to construct and to operate and is normally not an option for a station serving a land development project.

With designs where the maintenance personnel must enter the wet or dry wells, a minimum two-man crew must be present, whereas only one person can carry out the routine maintenance at a submersible station as no below-ground tasks are required. It is important to note that all pumping station designs be checked for flotation. While more engineers and utilities are using submersible pump stations as described above, the older accepted practice of constructing a built-in-place wet well-dry well pumping station remains the standard design for the larger installations. Submersible pumps can be used in dry well installations.

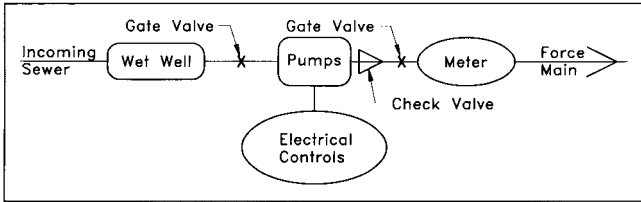
### Components of a Pumping Station

The flow schematic shown in Figure 25.20 is common to all pumping stations, except the gate valve is not needed between the wet well and the pump in a submersible design because the pump is installed in the wet well.

**Screening.** Screenings are the larger particles of floating and suspended matter that may clog pumps and other equipment. The quantity of screening normally does not exceed 0.5 ft<sup>3</sup> per million gallons of flow for residential sewage. Rags are the most troublesome items in pumping stations. The quantity of screenings to be removed will depend on the type of housing and other sources of incoming sewage. The screening device should be located such that all incoming flow passes through the chamber before entering the wet well. The device should only remove material that will interfere with the station operation because of the difficulty of handling removed material. Disposal methods include grinding and returning the material to the flow, burial and incineration. Burial and incineration should be off site from the pumping station. Maintenance crews generally carry the screenings from small pumping stations back to the treatment plant for treatment and disposal. Macerators are available for locating on the incoming sewer and should



**FIGURE 25.19** Typical submersible pump station. Reprinted with permission from FLYGT Corporation, 1993, *Wastewater Pumps*, Catalog, Trumbull, Conn.



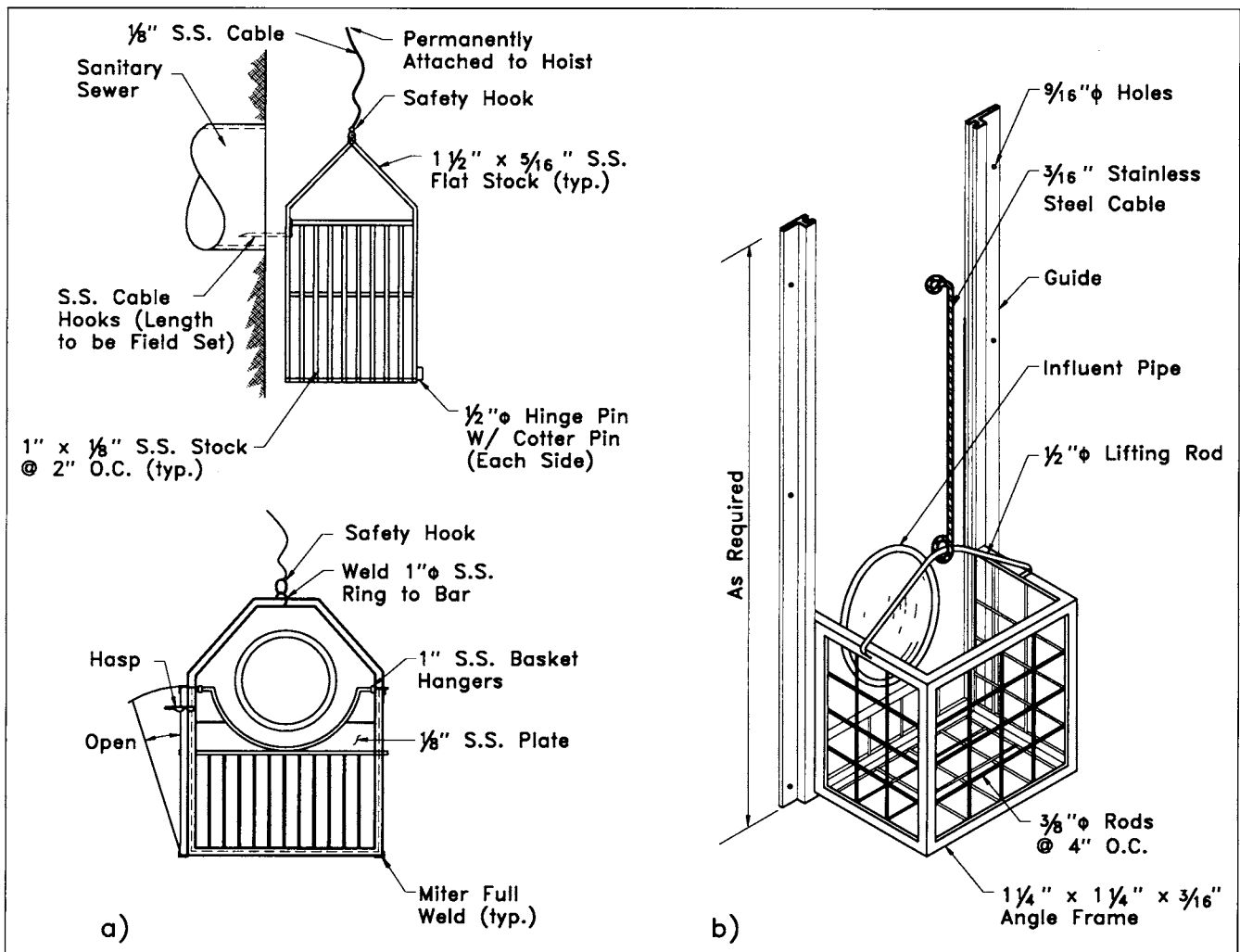
**FIGURE 25.20** Schematic of a pumping station.

be considered for use on sewers serving industrial areas producing significant fibrous wastes.

The screening device for small to moderate-size pumping stations, serving primarily residential areas, may be a basket located such that the flow passes through the basket as it enters the wet well. Daily cleaning should be scheduled until experience shows that a less frequent schedule is adequate. The design permits overflow in case of stoppages without causing a sewer backup. The basket should be designed for easy removal of the screening. With submersible

stations, the basket is lowered into place and removed by a cable hoist. The basket is held in place by guides or by hanging hooks as shown in Figure 25.21. Hanging hooks are generally more satisfactory because guides are a maintenance problem.

Fixed bar screens are used in larger pumping stations. The bar screen is placed in a straight channel, usually located at the entrance to the wet well. The screen is inclined from 30 to 60° from the horizontal. The corners of the channel should be filleted to minimize problems with solids accumulation, particularly grit. The bars generally have a face dimension of 1/4 to 3/8 in. and a depth of 2 to 2 1/2 in. The bar size for larger screens is determined by the structural requirements, with the face dimension limited to 3/8 in. The clear space between the bars is normally 2 to 2 1/2 in. The bars are held together with a welded strip located on the downstream side at the bottom and top of the screen. A rake, having teeth size and spacing compatible with the screen, should be provided for cleaning the screen. The screenings are raked to the top of the screen where



**FIGURE 25.21** Hook-mounted basket (a); Guide-mounted basket (b). Part b is reprinted with permission from Washington Aluminum Company, *Components for Water and Wastewater Plants*, Catalog, Baltimore, Md.

they fall into a can having a perforated bottom for drainage or onto a drain rack. The top of the bars may be curved over to facilitate cleaning. Fiberglass racks and slide gates are available and should be considered for a design. Mechanically-cleaned screens are used at large stations.

The approach velocity to the screen should not be greater than 2.5 fps at peak flow. The design must allow for the energy loss in the channel and through the screen so that the incoming sewer is not surcharged. Also, the maximum operating level in the downstream wet well must not surcharge the screen. When the screen is located in a subsurface chamber, suitable hoisting equipment should be provided for lifting the screening to the surface. A typical manually cleaned bar screen is shown in Figure 25.22. Calculations are shown for sizing a screen to handle the flow from the West Creek sewer.

The average and peak flows at Point Y are 1.25 cfs (809,560 gpd) and 5.63 cfs respectively (see Table 25.13). The approach velocity is to be 2.5 fps at peak flow. The bars are to be  $\frac{1}{4}$  in.  $\times$  2 in. with 2-in. clear openings between bars.

Cross-section area of flow =  $Q/V = 5.63 \text{ cfs}/2.5 \text{ fps} = 2.25 \text{ ft}^2$ .

**Select Channel Depth.** The incoming 15-in. sewer at 0.8 depth of flow = 1 ft; therefore, a 1-ft depth is selected for the approach channel. Width of channel =  $2.25 \text{ ft}/1.0 \text{ ft} = 2.25 \text{ ft} =$  width of screen. Note in Figure 25.25 that free fall of about 3 in. is provided at the influent pipe and 6 in of fall is provided through the screen chamber. The energy

loss through a clean screen is small, being less than 1 in., but most of the time the screen is partly clogged, and this can materially increase the loss. Note the curved bars that aid in handling the screening. The screenings are raked onto a drain pad. After draining, the screenings are removed for disposal.

**Sizing of the Wet Well.** The wet well is a chamber that has been sized to receive and accumulate the incoming flow when the pumps are not operating. The wet well also provides some flow equalization, thus reducing the peak pumping rate from the peak influent flow rate. Some common design criteria for pumping stations include:

1. Pumps are selected to pump 2.5 times the average daily flow rate with the largest pump out of service. For example, if only two pumps are provided, each must have a pumping capacity of 2.5 times the average daily flow rate. If three pumps are provided that are not of equal size, the two smaller capacity pumps must be capable of pumping 2.5 times the average daily rate. All pumping stations should have at least two pumps so that any single pump can be removed for maintenance.
2. There is a heat build-up in pump motors if the frequency of starting is too great. This can shorten the life of the motor. Most heavy-duty motors of the type used with pumps can be started up to about ten times per hour without overheating occurring. Starting frequency is controlled by the size of the wet well, and

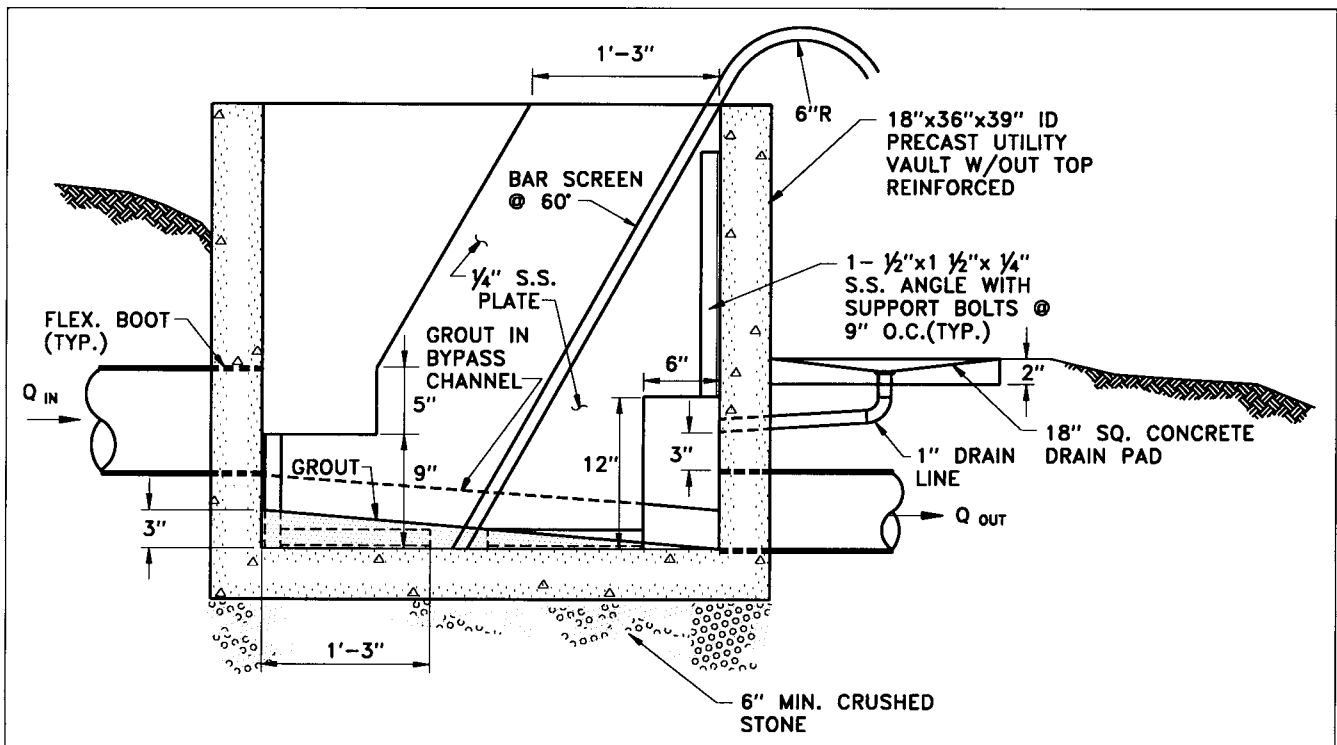


FIGURE 25.22 Manually cleaned bar screen.

this should be coordinated with the electrical designer to ensure that the proper class of motor is specified.

3. The wet well should be filleted to eliminate corners where grit can accumulate.
4. The wet well should be divided into sections at larger stations so that the section supplying each pump can be isolated and dewatered for maintenance. This can be accomplished with submersible pumps by placing each pump in a separate manhole that is suitably interconnected and valved.
5. If maintenance crews are required to enter a chamber for routine maintenance, proper ventilation must be provided. The ventilation fan should be sized to provide 30 air changes per hour. Air is usually exhausted from near the floor so that fresh air is brought into the top of the chamber as the crew descends. The exhaust fan switch should be located at the top of the stairs or ladder so the fan can be started before entering the subsurface chamber.
6. A potable water hose bib should be available at each structure for use in cleaning. The potable source should be protected by installing a backflow preventer on the service line serving all outlets except the drinking fountain and bathroom.
7. A flow-measuring device should be included in the design. Ultrasonic meters are reliable and widely accepted.
8. The short intake piping and intra-station effluent piping may be designed for velocities of 6 to 8 fps, but force mains are not normally designed for velocities greater than about 5.0 fps at the peak pumping rate. Higher velocities scour and erode the pipe, and the energy loss increases exponentially with an increase in velocity. A minimum velocity of 2.0 fps should be provided at the minimum pumping rate.
9. Pump stations, except for the very small ones, should have an alarm system for the following malfunctions: power supply failure, pump fail to start, high level in wet well, failure of sump pump to operate, and failure of on-site generator to start. Other alarms may be needed at specific locations. The alarm should be relayed, by telephone line or otherwise, to a central manned location.
10. A check valve should be placed in each pump discharge pipe to prevent back flow through the pump. A gate valve should follow the check valve so that the pump and check valve can be isolated for maintenance.
11. The top of the pump bowl on horizontal impeller shaft pumps should contain an air release valve that is vented back to the wet well.

12. No air should enter the dry well from the wet well.

13. Pumping stations located in an area where sewage back-up or overflow would result in a health hazard should include an on-site power generator. These generators are normally diesel engine driven and designed to start automatically upon a failure of the off-site power supply and to return to standby conditions when the off-site power supply is restored. The units are designed to keep the facility completely operational. An on-site fuel supply for 36 to 48 hr of operation is provided as a part of the design.

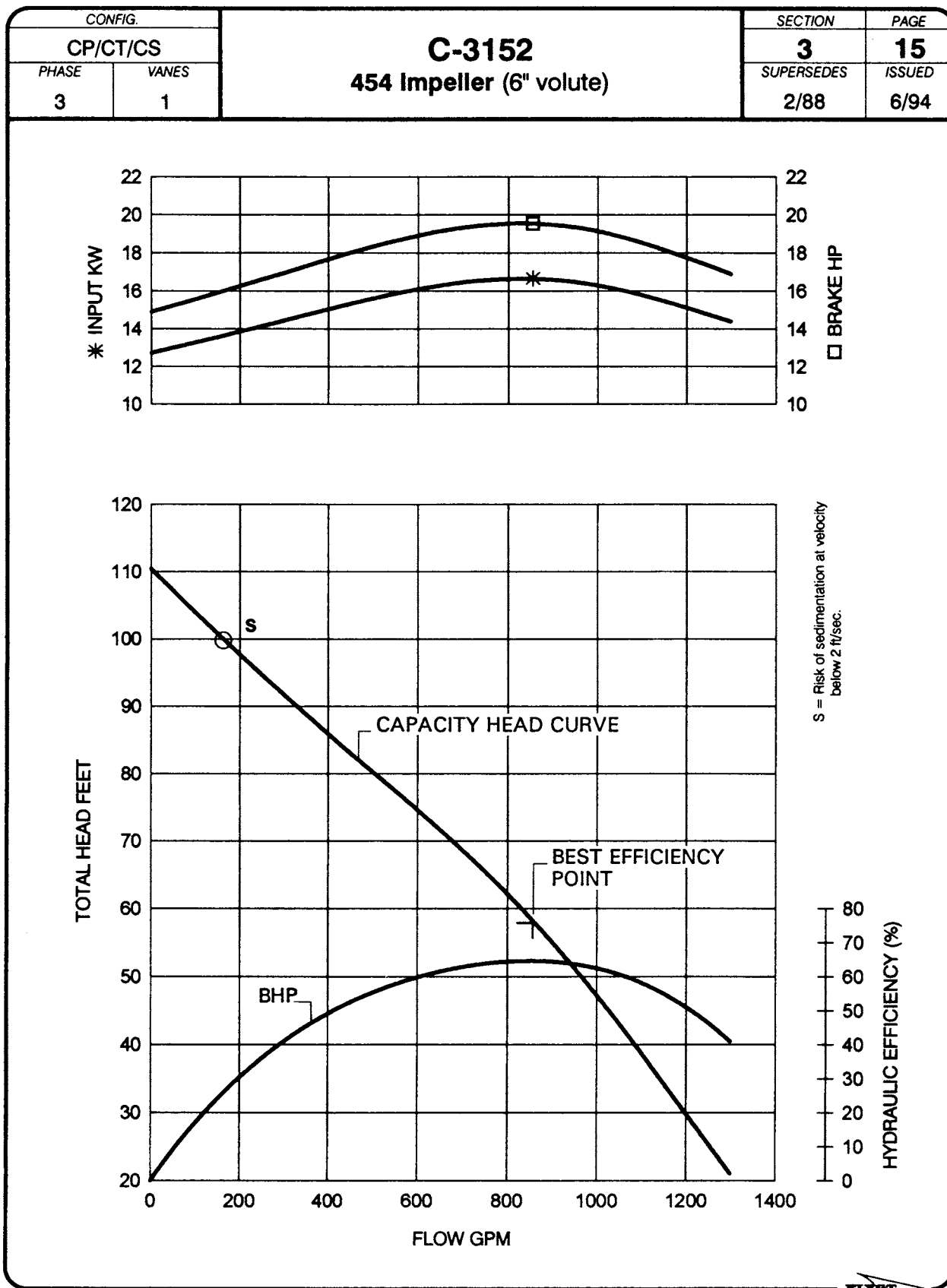
### Pump Selection

Sewage pumps are centrifugal, volute, non-clog impeller design. The impeller should be designed to pass a 2½-in. diameter sphere. The pump manufacturer provides performance curves for each impeller available for the specific pump. A typical performance chart for a submersible pump is shown in Figure 25.23.

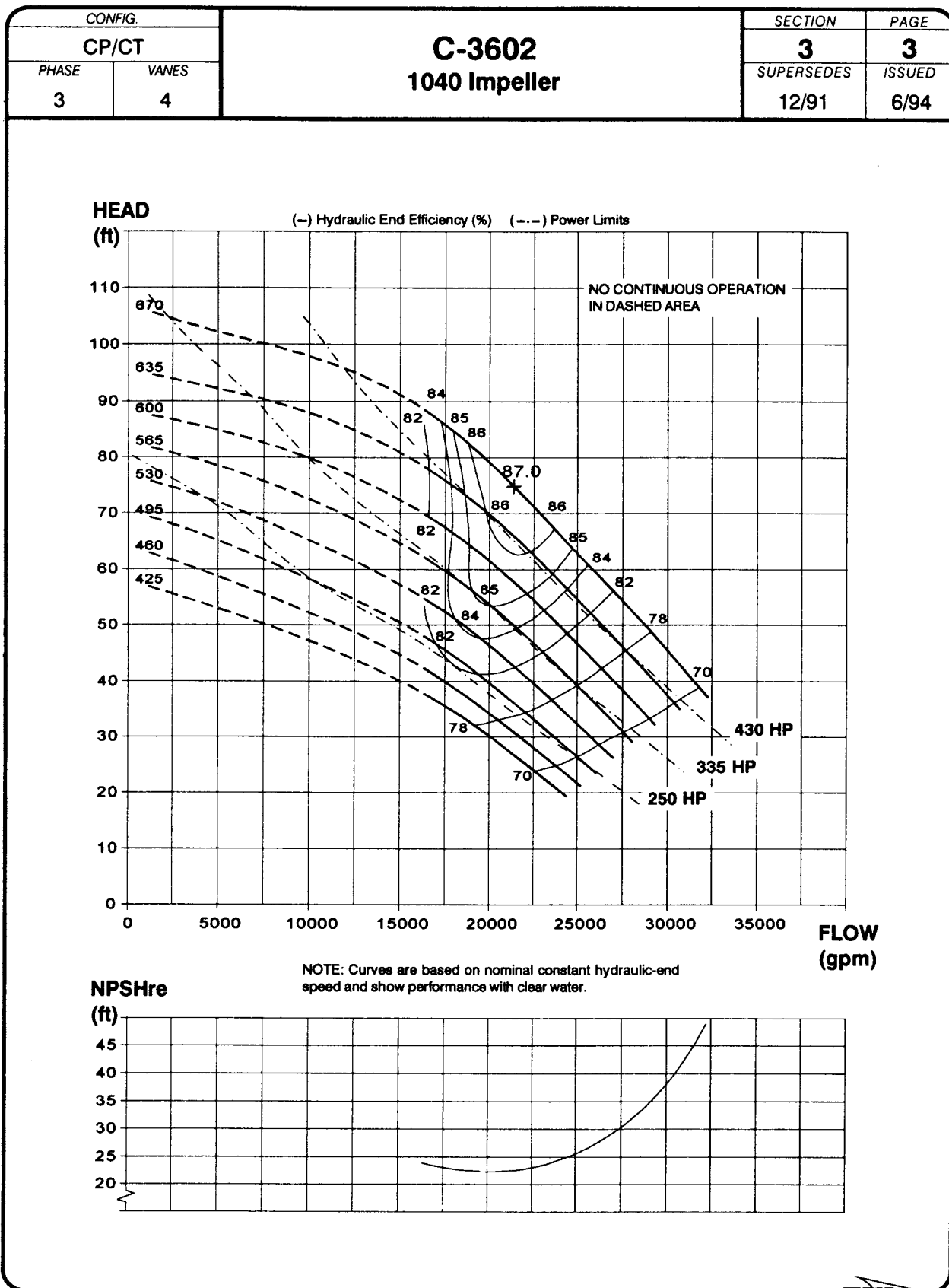
The performance curve shown in Figure 25.23 only shows the characteristics of the particular pump for a single impeller. The manufacturer casts the largest impeller that can be used with the pump. Impellers are then trimmed to the diameter required to provide the head and flow needed.

Figure 25.24 provides performance data for eight impeller diameters. The diameters, in millimeters, are shown along the ordinate of the graph. Efficiency curves are shown for the operational range of the pump, with the peak efficiency being 87% for the 870-millimeter impeller. Additional data available from catalogues include intake and discharge connection diameter, brake horsepower, and rotational speed. Rotational speed is also an important characteristic in pump selection. Higher rotational speed will increase flow and head, but bearing life is generally materially reduced. A lower rotational speed pump may cost more, but reliability will be significantly improved. The pump selected should operate at less than 1700 and specify any diameter between 7 in. and 10 in. Net positive suction head (NPSH) is the difference between the absolute pressure at the suction point and the vapor pressure of the liquid. Failure to maintain the NPSH can result in vaporization of the liquid being pumped, with the resulting cavitation causing damage to the pump. The low water pump shut off level is usually set no lower than the midpoint of the pump bowl so that pump suction is not a problem. Pump efficiency is shown to vary between 58 and 72%. Brake horsepower is shown for each of the four impeller sizes.

The total head imparted to the flow is shown as the ordinate for the indicated flow rate. The head imparted is not the same as the discharge pressure. The imparted head is the discharge energy minus the intake energy of the flow. Note that for centrifugal pumps, as the discharge head decreases, the motor horsepower requirement increases.



**FIGURE 25.23** Pump performance curve. Reprinted with permission from FLYGT Corporation, 1993, *Wastewater Pumps*, Catalog, Trumbull, Conn.



**FIGURE 25.24** Centrifugal pump performance curve. Reprinted with permission from FLYGT Corporation, 1993, *Wastewater Pumps*, Catalog, Trumbull, Conn.

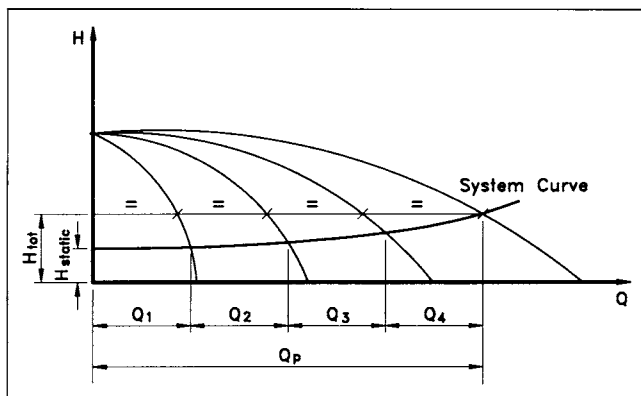


Never oversize a centrifugal pump as this may lead to an overloaded motor.

It is sometimes necessary to install two centrifugal pumps in series to provide the required energy input. When centrifugal pumps are installed in series, the flow through each pump is the same, and the energy input is the sum imparted by the two pumps. The two pumps are designed as separate pumps and piped to operate in series. With water pumps, series impellers can be installed in a single multi-stage pump housing (multi-stage pump has two or more impellers on the same shaft within a single housing). However, this can not be done with sewage pumps because the pump must be designed to pass solids. Therefore, sewage pumps are designed as separate pumps where series operation is required to induce the required head.

When centrifugal pumps are installed in parallel, the flows can not be summed to obtain the flow from the two (or more) pumps. Figure 25.25 shows a pump curve that has been constructed for four pumps installed in parallel. The pump curve is constructed by selecting a head and drawing a line parallel to the abscissa. Each pump discharge for that head is added along the line. In the figure, all four pumps are identical, but the same principle applies if the pumps are different. When pumps having different performance characteristics are installed in an installation, the smaller pump will not function if the discharge head of the larger pump is greater than the shut-off head of the smaller pump. Shut-off head is the head produced by a pump at zero flow and is shown by the ordinate-intercept of the pump curve.

A system curve is then constructed starting with the static head at the ordinate intercept and adding the system energy loss at two or more flow rates to permit drawing the curve. The interception of the system curve with the performance for the indicated number of pumps shows the capacity for the respective number of pumps. Note that in Figure 25.25, since four pumps are identical, the distance between the curves at the total head value are the same, as each pump has the same pumping capacity at any given



**FIGURE 25.25** Performance curve for up to four identical pumps operating in parallel.

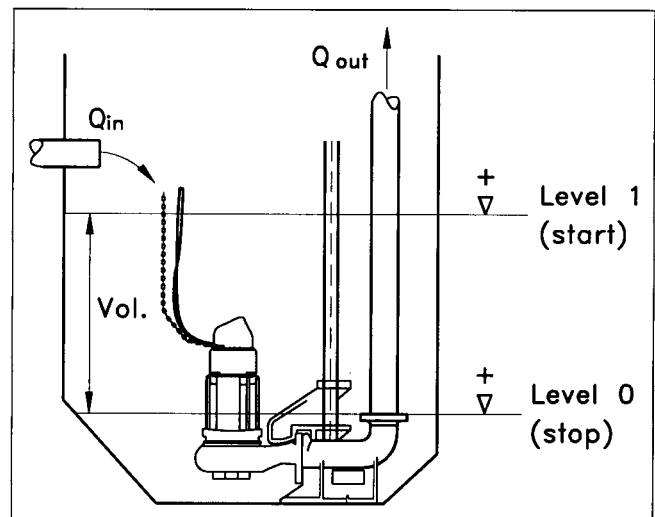
head. However, as more pumps come on line, the energy loss increases as defined by the Hazen-Williams equation and shown by the system curve. Thus, the flow capacity of two pumps is less than twice that of the single pump operating. The capacity of each pump is the same and is one half, one third or one fourth of the total, depending on the number of pumps operating.

Pumping stations serving a long force main should have a surge relief valve to relieve the surge from water hammer. This valve is located on the station discharge piping and is vented back to the wet well.

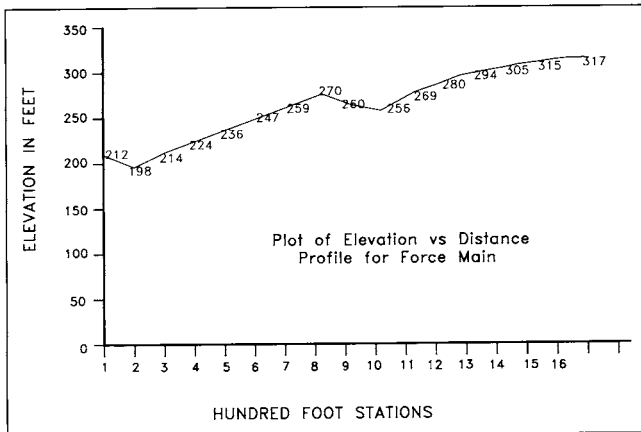
The incoming 15-in. sewer at Point Y has an invert elevation of 204.75 ft (see Table 25.15). The fall in the piping and bar screen is 1.00 ft, giving an invert elevation entering the wet well of 203.75 ft. Each of the two pumps are to be installed in an 8-ft diameter manhole (volume per vertical foot = 375.8 gallons per manhole = 100 ft<sup>3</sup> for the two manholes). The minimum inflow rate is taken as about 10% of the average rate or 50 gpm. The peak inflow rate is 2530 gpm (see Table 25.13) and the pumping rate will be 2.5 times the average daily inflow rate ( $2.5 \times 562.2 \text{ gpm} = 1405 \text{ gpm}$ ).

The wet well should be sized to provide at least a 4-minute run time at minimum flow. Therefore, the volume of the wet well is the pumping rate minus the inflow rate =  $(4 \times 1405) - (4 \times 50) = 5420 \text{ gallons} = 724.6 \text{ ft}^3$ . The operating depth of storage in the wet well =  $724.6 \text{ ft}^3 / 100 \text{ ft}^3 \text{ of volume per ft} = 7.25 \text{ ft}$ . Therefore, the pump cut-off depth =  $203.75 - 7.25 = 196.50 \text{ ft}$ . These depths are shown in Figure 25.26. Note that the bottom of the manhole is the distance from the pump connection flange to the bottom of the pump mounting plate, which is lower than the pump stop level. This dimension is obtained from pump catalog information.

A suction lift exists when the hydraulic grade line drops below the center of the pump bowl. Suction lifts should be



**FIGURE 25.26** Schematic showing pump wet well elevations.



**FIGURE 25.27** Profile of ground surface along force main route.

avoided with sewage pumps unless the installation is specifically designed to be self-priming. Total dynamic suction lift is the vertical distance in feet from the center line of the pump to the free liquid level in the wet well plus all energy losses in suction pipe and fittings.

The total dynamic head imparted by a pump is the difference in energy across the pump and is given by the  $E$  term in Eq. (25.16).

$$\frac{V_1^2}{2g} + P_1 + Z_1 + E_p = \frac{V_2^2}{2g} + P_2 + Z_2 + H_{1-2} \quad (25.16)$$

Points 1 and 2 are located on the suction and discharge side of the pump respectively. Point 1 may be the water surface level in the wet well and point 2 the free discharge point at the end of the force main. Points 1 and 2 must be the same for all parameters in Eq. (25.16).  $V$  is the velocity,  $Z$  is the vertical distance from the same datum to the respective points,  $E$  is the energy imparted by the pump and  $H$  is the energy loss between the two points. Each term in the equation has the dimension of feet.

Brake horsepower is the shaft input to the pump and is given by Eq. (25.17).

$$BHP = \frac{(\text{gpm})(\text{Head in Feet})}{(3,960)(\text{Pump Efficiency})} \quad (25.17)$$

Brake horsepower is the work done by the pump expressed in foot pounds per minute. Raising a gallon of water 10 ft represents  $8.34 \text{ #/gal} \times 10 \text{ ft} = 83.4 \text{ ft-#}$  of work. One horsepower is doing work at a rate of 33,000 ft-# per minute. In Eq. (25.17), 3960 is obtained by dividing 33,000 by 8.34. Pump efficiency is expressed as a decimal and not in a percentage (obtained from pump performance curves). Velocity head is usually ignored in pump selection because it is insignificant. For example, at a velocity of 5 fps, the velocity head (see Eq. (25.5)) =  $25/64.4 = 0.39 \text{ ft}$ .

The profile for the force main to be constructed between the West Creek pumping station located at Point "Y" in Figure 25.18 to the existing gravity trunk sewer is shown in Figure 25.30. The low water level in the station wet well is 196.50 ft. The invert of the force main at the discharge point is 308.15 ft (obtained from field survey or taken from as-built drawings available at utility), giving a static head of 111.65 ft.

The energy loss due to pipe roughness for pressure flow is generally determined by the Hazen-Williams Equation. The terms in Eq. (25.18) are:  $V$  = velocity in fps,  $C$  is the Hazen Williams Coefficient,  $R$  is the hydraulic radius =  $D/4$  in feet for circular pipe and  $S$  is the hydraulic gradient = head loss/length. Eq. (25.19) is the continuity equation for non-compressible flow. Eqs. (25.20) and (25.21) are rearrangements of Eq. (25.18). Note that the flow varies directly as  $C$  for a constant hydraulic gradient and pipe diameter. The technology of pipe manufacture is such that all pipe is near smooth wall. Most information supplied by pipe manufacturers indicates that a  $C$  value of 140 to 150 should be used in design; however, engineers generally use a value between 100 and 120. This allows for normal minor losses in the pipe and also accounts for a somewhat deteriorated pipe wall condition after the pipe has been in use for several years.

$$V = 1.318 CR^{0.63} S^{0.54} \quad (25.18)$$

$$Q = AV \quad (25.19)$$

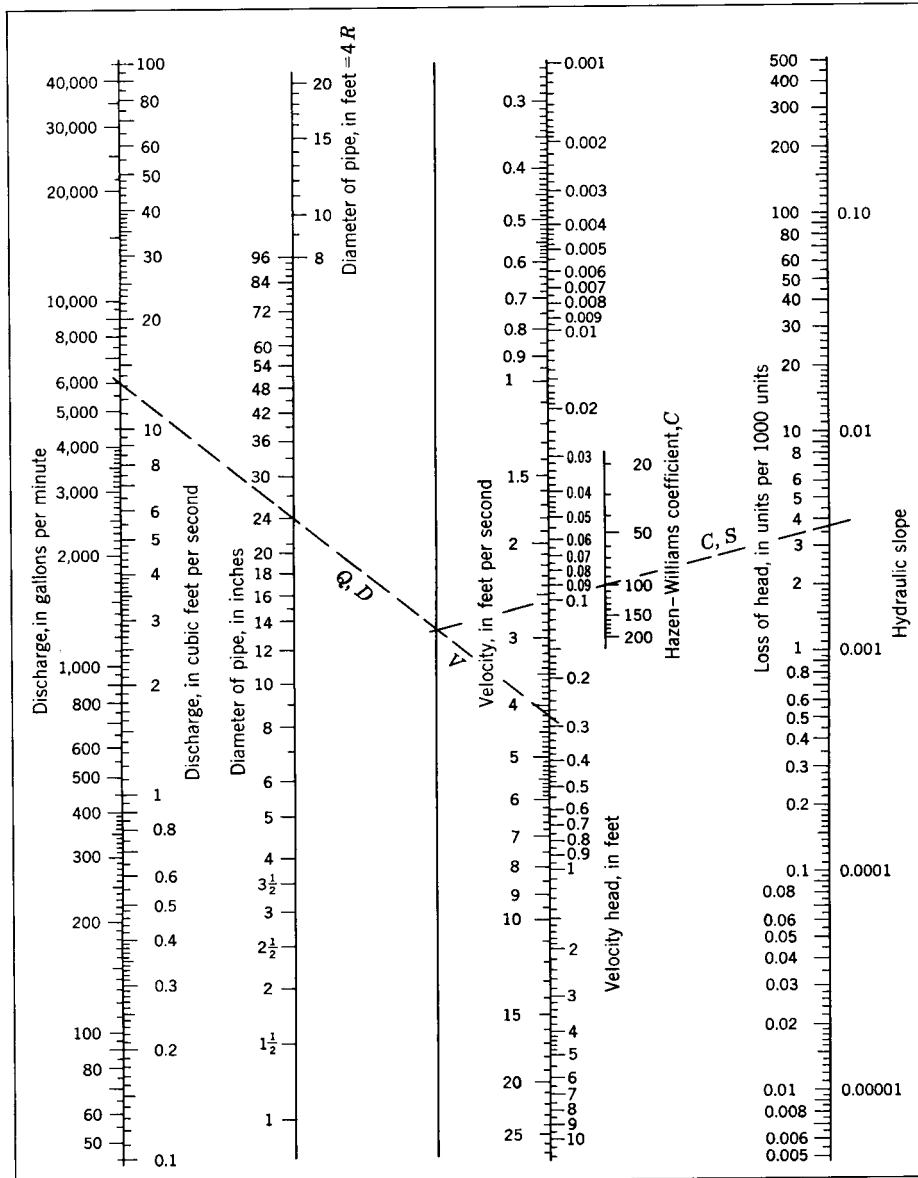
$$\frac{Q_1}{Q_2} = \frac{C_1}{C_2} \quad (25.20)$$

$$H_f = \left[ \frac{V}{1.318 CR^{0.63}} \right]^{1.85} L \quad (25.21)$$

Various types of nomographs and slide rules are available for solving the Hazen Williams Equation. One type of nomograph is shown as Figure 25.28.

As determined above, the static head for the pumping station is 111.65. The friction loss is based on 1600 ft of pipe plus the minor losses in the pumping station. The pumping rate is 1405 gpm or 3.13 cfs. Force mains are generally designed for a velocity at peak flow of around 5 fps. From the continuity equation and Figure 25.31, it is found that a 12-in. pipe at a velocity of 5.0 fps will have a capacity of 1737 gpm and that a 10-in. pipe requires a velocity of 5.7 fps to carry 1405 gpm. The 12-in. pipe is selected for the force main because the friction loss will be less and because there is another area at station 10+00 that may use capacity in the force main at a later date.

Since the flow from the pumping station will be small for several years, it is decided to install less than the full pumping capability at this time. A force main should have a minimum velocity of 2 fps when the pumps are operating. Thus, the initial pumping capacity will be at least 710 gpm. Figure 25.28 shows a hydraulic gradient of 0.0016 when



**FIGURE 25.28** Alignment chart for Hazen-Williams Formula for pipe flow. From *Design and Construction of Sanitary and Storm Sewers, Manual of Practice No. 9*, ASCE 1969. Reprinted with permission from the American Society of Civil Engineers, New York, New York.

using a design C of 120 and a flow of 710 gpm in the 12-in. pipe. The hydraulic gradient at 1405 gpm is 0.0056.

Minor losses are losses in valves, bends and segments of a conduit that produce turbulence over that for straight pipe. These losses are usually neglected in a pipeline, but they can become significant in a pumping station. One method of accounting for minor losses is by use of Figure 25.29. Knowing the source of the minor loss, the information from the figure can be used to determine an equivalent length of straight pipe. For example, a check valve is equivalent to 80 ft of 12-in. pipe. The following minor losses apply to the West Creek Pumping Station: one check valve, one gate valve, two elbows and one tee, giving the following equivalent lengths: 80 ft + 6 ft + 33 ft (2) + 70

ft, giving a total equivalent line length of 1600 + 222 = 1822 ft. At the hydraulic gradient of 0.0056, the initial friction head loss will be 10.2 ft, which is added to the static head giving a pumping head of 111.65 + 10.2 = 122 ft. The pump selected, as shown by the performance curve in Figure 25.26, will deliver about 750 gpm at the required head. This pump will operate at about 53% efficiency and require a 40-Hp motor.

The pump selected will accept larger impellers and motors as shown by the catalog data. These can be changed when the need arises without having to replace the pump. The daily cost of pumping can be determined as follows:  $BHp = (750 \text{ gpm})(122 \text{ ft}) / (3960)(0.53) = 43.6$  (see Eq. (25.15)). Electric Power = BHp/Efficiency of

motor =  $43.6/0.92 = 47.4$  Hp. Cost = (Hp)(0.746 kW/Hp) (Hours)<sup>1</sup> (\$/kWh) = (47.4) (0.746) (24) (0.40<sup>2</sup>) (\$0.08<sup>3</sup>) = \$27.16/day.

Other appurtenances to be included in a force main design include air relief valves and blow-off valves. Air relief valves must be placed at high points in the force main, as an accumulation of air will restrict the area of flow and result in a decreased carrying capacity of the line. It is seen from the line profile in Figure 25.27 that an air valve is needed at about station 8+00. An air relief valve for sewage must be used. The valve is placed in an enclosure such as a manhole. Blow-off valves are located at low points in the line to permit draining the line for maintenance. A blow-off valve is usually a gate valve or a plug valve. The profile of the main shows that a blow-off valve is needed at about station 10+00. Plug valves are generally preferred over gate valves because of the ease of operation. A plug valve only needs to be turned 90 degrees to open or close, whereas a gate valve requires much more effort and time to open or close.

Solid state frequency controllers are very reliable means of varying the speed of the pump motor. Variable speed pumps should be considered for pumping stations as their use will minimize surges in the force main from the constant speed pump cycle. The variable speed feature will result in the pumping rate matching the incoming flow rate. Variable speed pumping stations at sewage treatment plants will greatly reduce surge waves through the plant.

## DESIGN OF SANITARY SEWERS WITHIN SUBDIVISIONS

### General

The previous section presents procedures for the design of a main or trunk sewer such as those needed for sewerage an entire sewer shed. This section presents the principal elements that are generally applicable to the layout and design of a sewage collection system to serve a subdivision or development.

The design of a sewage collection system for a subdivision generally requires fewer hydraulic calculations than is needed in the design of a main sewer, such as presented in Table 25.13 and Table 25.15. There are many important aspects in the layout and design of any sewage collection system, and an experienced engineer should be involved in the process. A poorly conceived layout of a collection system will increase the construction costs and generally result in additional maintenance requirements. Care must also be taken to insure that all lots can be properly sewerage.

The hydraulics of sewers as presented, along with the data in Table 25.5, apply to all sewer designs. However, the calculations required in the design of collector sewers are generally much less complicated than what are required for larger sewers. As shown in Table 25.5, an 8-in. sewer flowing full at a slope of 0.0042 ft/ft, and a roughness coefficient  $n$  of 0.013 has a capacity of 354 gpm (0.51 MGD). At this flow, the “peaking factor” is 5.0 (see Figure 25.2), giving an average daily flow for design of 0.1 MGD (hydraulic design flow divided by peaking factor = average daily flow from source). Thus, an 8-in. sewer will sewer 270 single-family houses (270 SF units times 370 gpd/unit = 99,900 gpd) or 333 townhouses (see Table 25.2 for average daily flow per unit). Likewise, a 10-in. sewer will serve 476 single-family units or 587 townhouses (peak factor = 4.5). A 12-in. sewer will serve 700 single-family units (peak factor = 4.4) or 863 townhouses.

The sewer capacities shown above are based on a minimum velocity of 2.25 fps. When a sewer is placed on a slope greater than required to provide the minimum velocity, the sewer capacity is increased in accordance with the Manning equation (see Figure 25.7). Generally, the sewer design for the normal subdivision does not require the calculations of flow and tabulation of data as shown for the larger capacity main sewers by Tables 25.13 and 25.15. For subdivisions where the number of units is less than 500, the engineer is concerned with selecting the most appropriate sewer layout and insuring that the sewer depth is sufficient to provide gravity drainage from the units served and that the sewer slope is equal to or greater than required to provide a velocity of 2.25 fps (slope greater than 0.0042 ft/ft). When designing the sewer collection system for single-family subdivisions greater in size than 500 units or townhouse developments greater than about 800, the design engineer should perform the design calculations as shown for large sewer mains. The upstream terminal segment of a sewer line should be placed on a 1% slope because this segment will only serve a few houses, and the greater slope is needed to ensure that adequate scour is provided at the low flow. The minimum diameter for a public sewer is 8-in. Also, as noted previously, infiltration should essentially be zero in a properly constructed sewer. The unit flow data given in Table 25.2 includes an allowance for infiltration.

### Feasibility Investigation

Often times an engineering firm will become involved in a feasibility investigation. This investigation, generally conducted prior to the developer purchasing the property, is important in that it will confirm the zoning of the parcel, the availability of an adequate water supply and sewer service. The investigation should also discover any limitations to development, whether environmental or physical.

The engineering firm should be staffed to provide complete engineering and surveying services as needed to carry

<sup>1</sup> Note: 24 hours is used in the example. Pumps may operate fewer hours per day, and this would be used in the equation period.

<sup>2</sup> Since the pumping rate is 2.5 times the average daily flow, the pump will only operate 40% of the time when the flow reaches the design rate. The flow and pump run time will be less until the sewer shed is developed to design density.

<sup>3</sup> Power costs are assumed as \$0.08 per kilowatt hour.

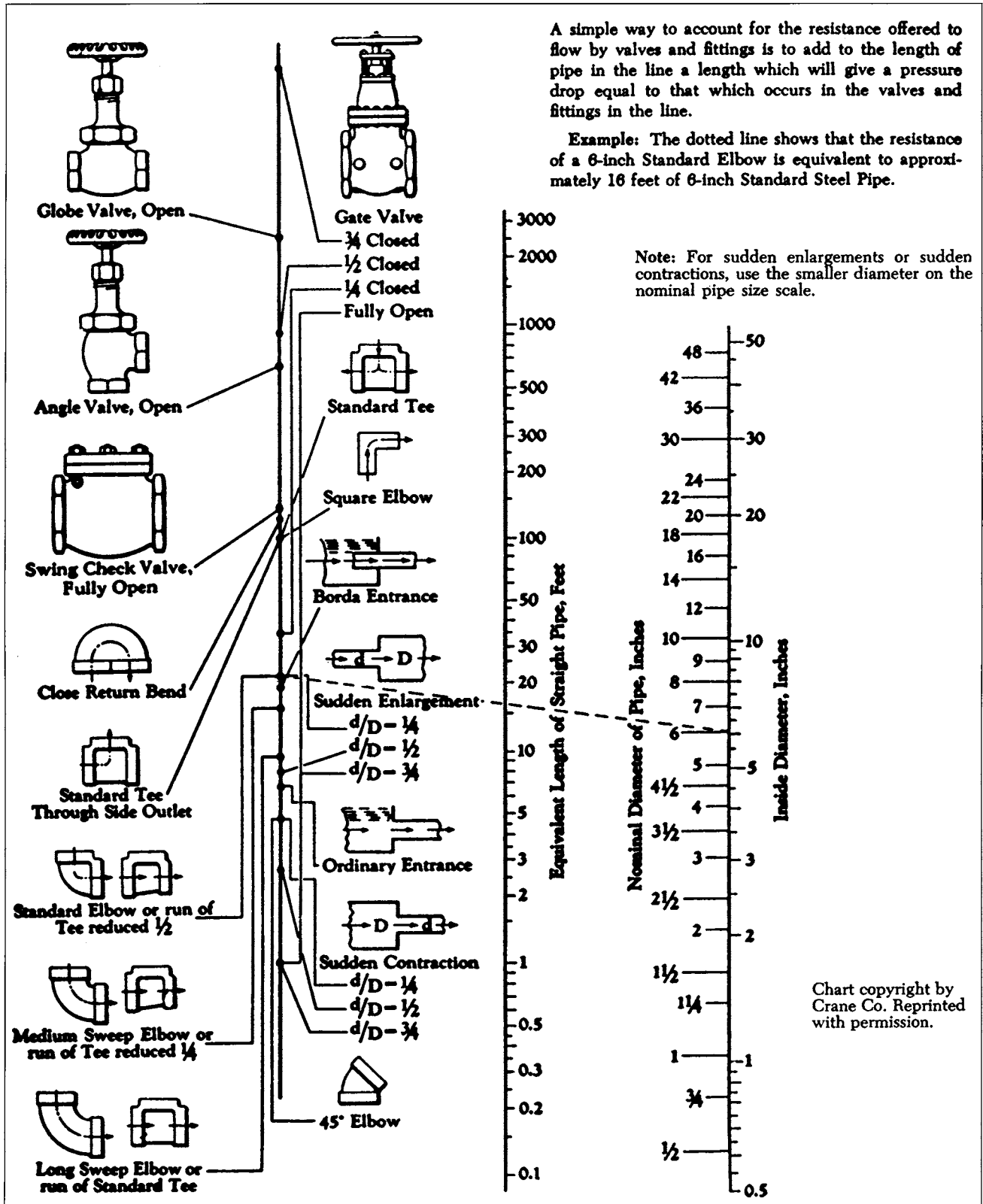


FIGURE 25.29 Equivalent lengths for minor losses. Reprinted with permission from Crane Co. Stamford, Conn.

the project from inception to completion of construction. The staff should have a thorough knowledge of local and state ordinances, laws and regulations, in addition to a broad experience in the technical aspects of land development.

Some of the items to be researched as a part of the feasibility investigation include:

- Location of the parcel.
- Metes and bounds of the parcel. Confirm title to the property.
- Zoning for the parcel or comprehensive plan if not zoned.
- Availability of utilities, including location and capacity.
- Are off-site easements needed to bring utilities to site? Local public utility should be willing to acquire any needed off-site easements if condemnation is required.
- Mapping available. Existing mapping may be adequate for developing a preliminary plan, but field surveys should be conducted for establishing the site boundary and topography of the site for final design.
- Existing easements on the property.
- Environmental concerns—wetlands, noise, endangered species, flooding, drainage problems.
- Will sections of sewer system need to be designed to provide future service to off-site properties.
- Any special local policy or regulation such as storm water management or open space requirement.
- Traffic considerations.

Zoning for the parcel will dictate several features of the development that will affect the sewer layout. These features include: street width, lot size, sidewalk and curb and gutter requirements, and building setbacks from property lines. This chapter is only concerned with items related to the sewer design. A more complete description of items associated with a feasibility investigation is included in the chapter on Feasibility Studies. A second meeting should be scheduled with the staff of the sewer utility and other agencies having jurisdictional authority after most of the material for the preliminary plan has been accumulated to ensure that all factors are given consideration.

If the results of the initial investigation are favorable for the planned development, the engineer should proceed with the preparation of the preliminary plan. If problem areas are discovered that may limit the proposed development, or if an unexpected development expense, either on site or off site, is discovered, the developer should be advised before proceeding further. If the developer determines

that the additional costs are acceptable, he should so advise the engineer so that work can proceed.

### The Preliminary Plan

An experienced engineer should direct the preliminary plan design. Many important aspects leading to the success of the development are related to the preliminary plan. These include: the lot yield of the tract, the grading required, the length of roadway to be constructed, the layout, costs and ease of construction of the sewer and other utilities and the overall appearance of the completed development. The engineer's task is to select a sewer layout that will most effectively serve an area at the least cost while providing long-term reliable service with minimum maintenance. The principal items affecting construction costs are the length of sewer, sewer depth and subsurface obstacles to construction. Deep sewers are particularly expensive when construction is in rock. The principal items affecting maintenance costs include infiltration and stoppages. Both items are related to the quality of the material specified by the engineer and to the quality of the construction. A properly designed and constructed sewer should require little if any maintenance. The engineer has suitable material available for use that will provide relatively carefree service for many years. Good engineering includes specifying the proper materials and construction requirements and providing competent inspection of the construction.

Items such as the boundary shape of the parcel, topography, and the location of existing connecting roads and utility lines will influence the location of streets and planimetric design. While it is preferable that utilities such as sewer, water, and gas be located within the public street for ease of maintenance, this may not be possible for all sewer lines because of topographic considerations. Water and gas lines are almost always located within the public streets, with the water meter being placed at the front property line. When the sewer is located within the street, it is generally located along the center line, except for curvilinear street segments, where the manholes are located so as to keep the sewer within the paved street area. Locating the sewer as near as feasible to the center of the street will result in equal lengths of building spurs to the units on each side of the street. The sewer should never be located under the curb because access for maintenance is thereby limited. Most organized utility departments have regulations on the location of utility lines because uniformity in location and construction methods throughout the system aids in maintenance.

A public or common sewer is a sewer that serves two or more building units. A private-ownership segment of sewer should never serve more than one customer. All segments of common or public sewers that are not constructed within public right-of-ways should be within dedicated easements. A 15-ft-wide easement is desirable with the sewer line located at its center. Easements should not straddle lot lines because homeowners often plant shrubbery or construct fences along property lines, making access to the sewer dif-

difficult. Easements should extend to the development boundary at locations where the sewer will need to be extended in the future to serve off-site property.

Proper separation must be maintained between water and sewer lines. Regulatory agencies generally require a minimum of 10 ft of horizontal separation between the two conduits. Where field conditions preclude a 10-ft separation, some agencies will permit construction with a 5-ft horizontal plus an 18-in. vertical separation, with the water line being the higher. Special design requirements may be used where suitable separation cannot be maintained. Special conditions may include the use of ductile iron pipe for the sewers or other safeguards to insure that the water lines do not become contaminated in case of line leakage or breakage. Sewer lines placed within 20 ft of buildings may require special construction, such as concrete encasement, to ensure that any leakage will not seep around the building foundation. Also, if a building is located near or over a sewer, it may not be possible to access the sewer for any required maintenance.

Sewer depth is established by the elevation of the buildings to be served. The sewer should be sufficiently deep to permit gravity drainage from all buildings if possible. For residential subdivisions, the sewer depth is based on the building construction. If the homes have basements, the sewer should be at sufficient depth to gravity-sewer the basement. Basement floors will generally be about 6 ft below finished grade at the highest point next to the house. An additional 2 ft of depth is needed to allow for the basement foundation as the house sewer should be located below the wall foundation and not through the foundation. The building spur (sometimes referred to as the house lateral or house sewer) should be placed on a minimum slope of  $\frac{1}{4}$  in. per ft or 2 ft per 100 ft (slope = 0.02 ft/ft). Plumbing codes allow a slope of 1 ft per 100 ft (slope = 0.01 ft/ft) for sewers under conditions where the greater slope cannot be conveniently provided. Thus, where the public sewer is located in the center of a 50-ft street and the setback for the house is 50 ft, the invert of the house spur at the connecting point to the public sewer is 9.5 ft lower than highest point of the finished grade at the house wall (from above 6 ft + 2 ft = 8 ft at outside of building foundation plus 1-ft drop in elevation in the 50 ft distance between the house and front property line plus  $\frac{1}{2}$ -ft drop in elevation in the 25-ft distance from the property line to center of street). To allow for the wye connection, the invert of the public sewer is the sewer diameter plus 1 ft, lower than that of the incoming house spur (see Figure 25.17 for typical spur connection to public sewer). Thus, a total vertical drop of about 12 ft between the highest proposed grade at the house wall and the invert of the public sewer is needed to ensure gravity drainage when the house is constructed.

In relatively level terrain, there will be a drop across the lawn of about 2 ft between the front building wall and the street, making the sewer about 8 ft to 10 ft lower than the

street. Sewers should not be constructed deeper than 20 ft except where alternatives are not available because of the difficulty in maintenance of deep sewers. Topographic conditions may make it necessary to locate sewer lines along the back of some lots in order to provide gravity drainage at a reasonable sewer depth.

A portion of a typical preliminary plan is shown in Figure 25.5. This is part of a plan for a proposed development that was submitted to a local government for review.

### Final Plan

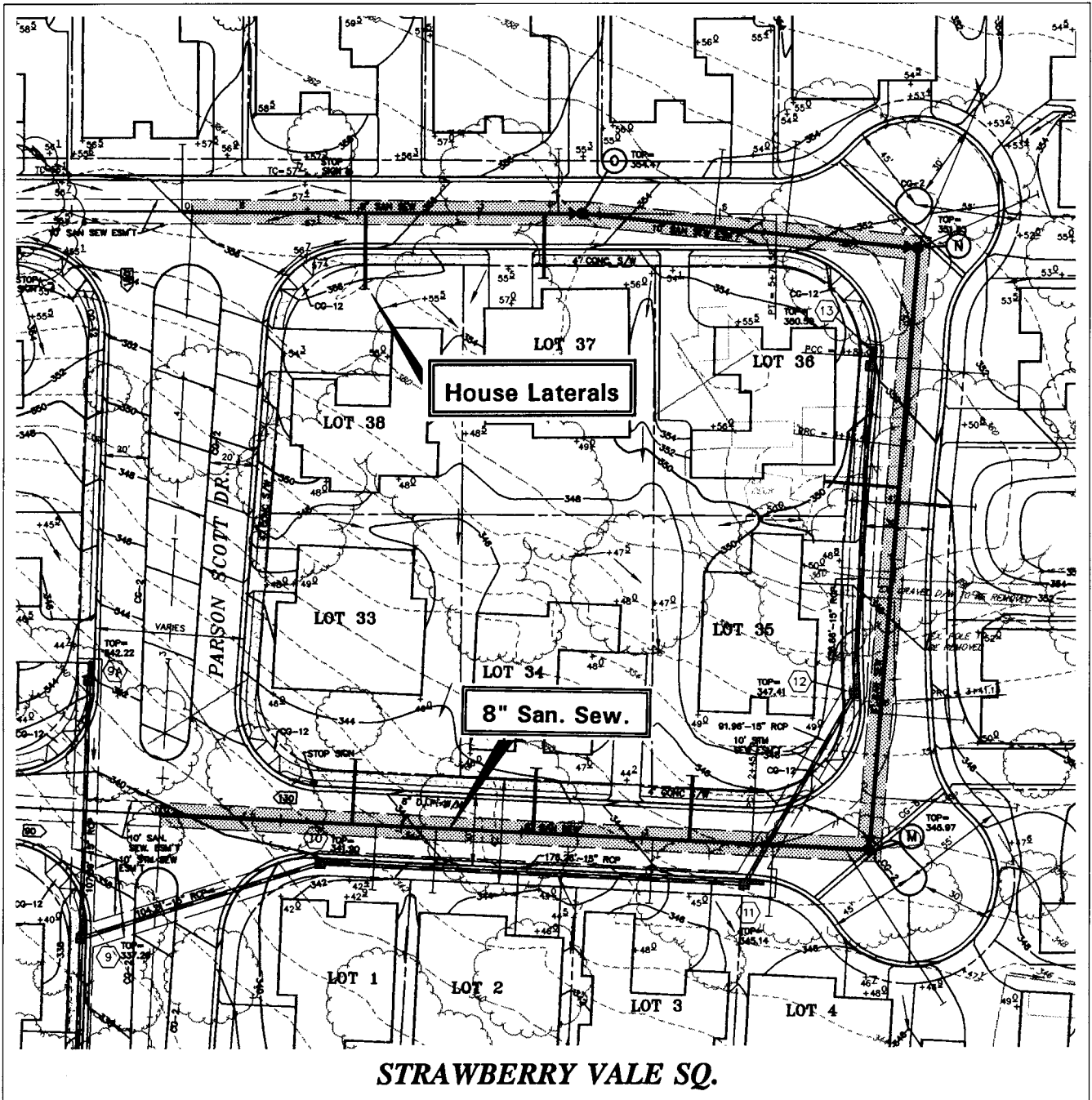
The engineer should not proceed with the preparation of the final plans for a development until the preliminary plan review has been completed and all departments of the local government have approved the plan. In most instances, one department will act as the control department and distribute the plans to the necessary departments, and all review comments will come back through the control department. While the sewage utility, whether it be the Department of Public Works or an Authority, will review and comment on the sewer system proposed, the preliminary plan will be reviewed by a number of departments and agencies.

Since, as noted previously, design calculations are seldom required for sewers within a subdivision, sewer slopes are shown on the profile for each segment of sewer. The slope is constant between manholes.

The design engineer should follow certain rules in selecting sewer slope. The slope for each segment of sewer running between adjacent manholes is selected on the basis of design conditions. The overall design objective is to design the collection system that can be constructed most economically while maintaining good engineering practice. This generally means keeping the sewer as shallow as possible while complying with the criteria given in items 1 and 2 below.

1. Minimum sewer slope—the slope required to provide a velocity of 2.25 fps, or the slope required to carry the required flow, whichever is greatest.
2. Minimum cover
  - a. Sewers located in streets should have at least 6 ft of cover. If less than 6 ft of cover is provided, special bedding may be needed because of the superimposed load from traffic.
  - b. Sewers located within off-street easements should be placed below the freeze depth or at least to a depth of 3 ft.
  - c. Sewer segments serving a building lateral should be at sufficient depth to provide gravity drainage from the building served.

Figure 25.30 shows a portion of the final plan for the same section of a proposed development as the preliminary plan shown in Figure 25.5. The planimetric sewer layout is



**FIGURE 25.30** Final plan.

highlighted on this drawing. The final grading contours are shown along with the storm drainage. The manholes have been numbered and the locations of the house laterals are shown.

### Plan and Profile

A plan and profile is prepared showing all utilities, including the sewers. If the sewer is located within a street, the lines are a part of the street plan and profile. All existing and proposed underground utilities should be shown on

these drawings. If underground utilities are not shown, the construction contractor is likely to damage them, leading to additional costs.

A plan and profile sheet is used for presenting this design. The plan view is shown on the top half of the sheet and the profile is shown directly below on the bottom half of the sheet. The plan and profile view for the final plan of Figure 25.30 is shown in Figure 25.31. The scale (of the original drawing) is  $1" = 50'$ . In the profile view, the horizontal scale of the profile is the same as that of the plan



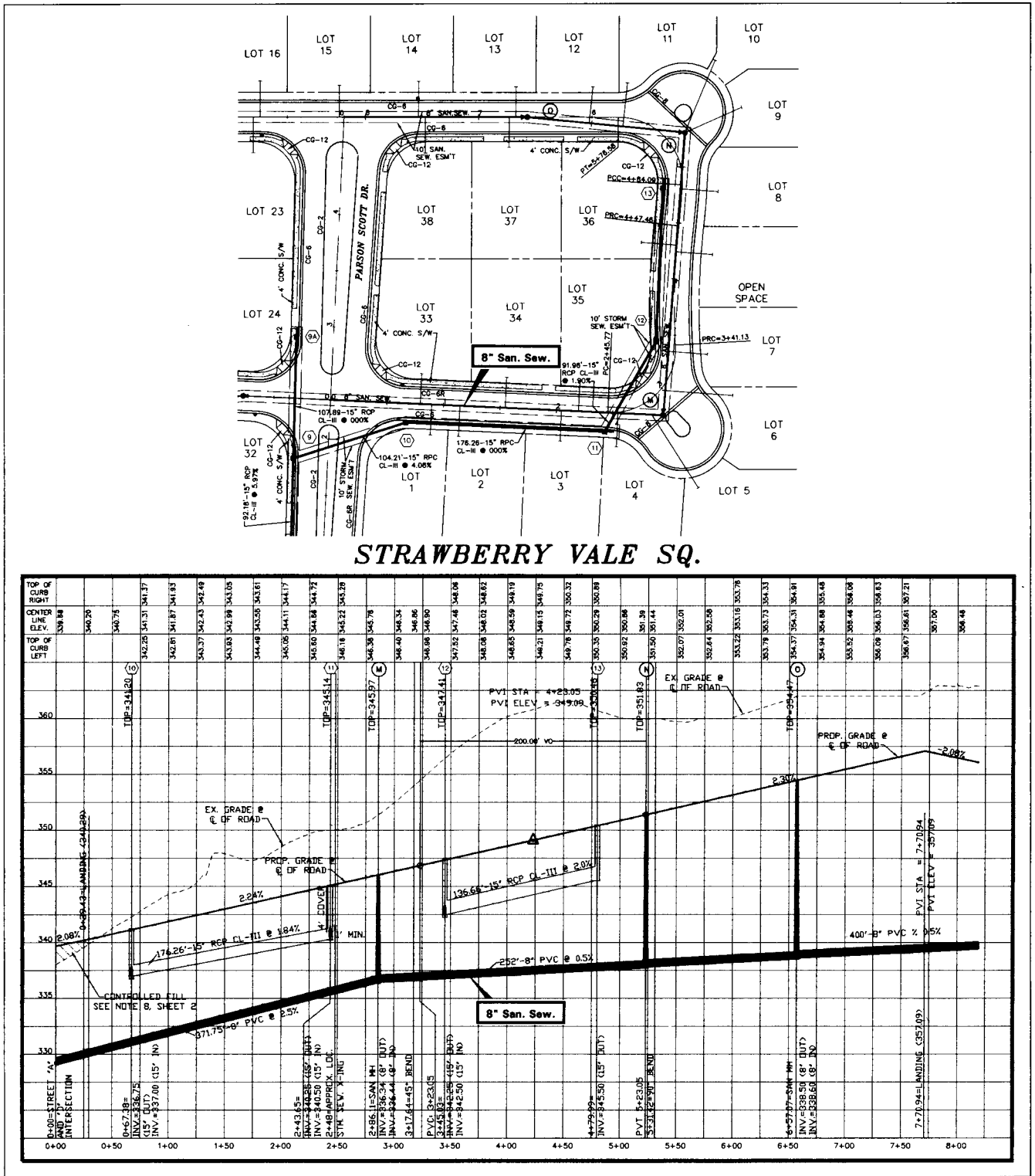


FIGURE 25.31 Example of a plan and profile view for a sanitary sewer line.

view. The vertical scale is 1" = 5'. The larger scale is used to show the detail needed for construction. The drawing is carefully constructed, as the contractor may need to scale some dimensions from the drawing during the project construction.

Corresponding manhole numbers are shown on both the plan and profile views. The building lateral locations are shown on the plan view, but note that the stationing and elevation of each building lateral at the lot line is also shown on the profile view. The contractor must construct the sewer as shown. Sewers located in a street are constructed to just inside the property line and marked at the surface with a stake so that the house plumber can access the sewer without disturbing the street. He keeps a set of plans in the field with current field conditions noted thereon as the construction progresses. This field-set of plans is used to construct as-built drawings that become record drawings. The utility crews can then use these drawings in future years in maintaining the sewers.

The manhole stationing, along with the sewer slopes and diameter, is shown for each segment of sewer. Note that the sewer grades are considerably greater than the 0.0042 ft/ft required to provide the minimum velocity of 2.25 fps. The water line is placed to provide 4 ft of cover, whereas the sewers are required to be much deeper in order to provide gravity drainage from the buildings. All information needed by the contractor to construct the sewer, roads and other utilities is shown on the plan and profile. The engineer must be thorough in ensuring that all underground conditions are shown on the final plan and profile. Existing underground utility location can be obtained from as-built drawings and supplemented through field surveys and test pits. Specifications are prepared to establish the allowed type of pipe material, the type of bedding required and the acceptable infiltration. This information can also be included on the plans as notes.

## PRESSURE SEWERS

A pressure sewer collection system is used to reduce costs relative to the cost of a conventional gravity system. The technology may be the only feasible means of sewerage in some areas. Pressure sewers are particularly applicable for sewerage in less populated areas and developments or communities located in hilly or rocky terrain. Conventional gravity sewers may need to be deep and costly to construct in areas where the topography is undulating. Also, a high water table may make the construction of gravity sewers economically unfeasible. Pressure sewers also have merit other than lower construction cost. Reliable pumping equipment is available. The technology requires no modification to the house plumbing, and, therefore, the use of pressure systems causes little inconvenience to the homeowner.

The two major types of pressure sewer systems are the grinder pump (GP) system and the septic tank effluent pres-

sure (STEP) system. Figure 25.32 shows the basic elements of both systems. A typical septic tank pump installation is shown in Figure 27.5. A typical grinder pump is shown in Figure 25.33. The GP system consists of a grinder pump that receives the flow from a dwelling or activity and pumps the flow into a pressure force main. The GP system may also be installed to replace an existing septic tank system as shown in Figure 25.32. The STEP system follows a conventional septic tank where the flow is pumped into a pressure force main.

Pumping equipment for the GP technology is well-developed. The pump shown in Figure 25.33 is available in several sizes and capacities to serve uses from a single-family dwelling to a small commercial or industrial flow. The unit is supplied in a pre-piped and wired fiberglass enclosure that can be installed in a basement, crawl space, or below ground level. The units are available with one or two pumps for reliability. The unit is installed and the 4-in. building sewer is connected. Pumps are available for pumping against discharge heads of more than 100 ft. Pressure systems are designed similar to force mains as discussed in the section on pumping stations, except a determination of the number of pumps operating on the main at any one time and the resulting flow. GP suppliers have developed this type of statistical data, and it can be found in their catalogs.

The normal wet well size for a single-family dwelling is 60 gallons (pump enclosure provides wet well volume).

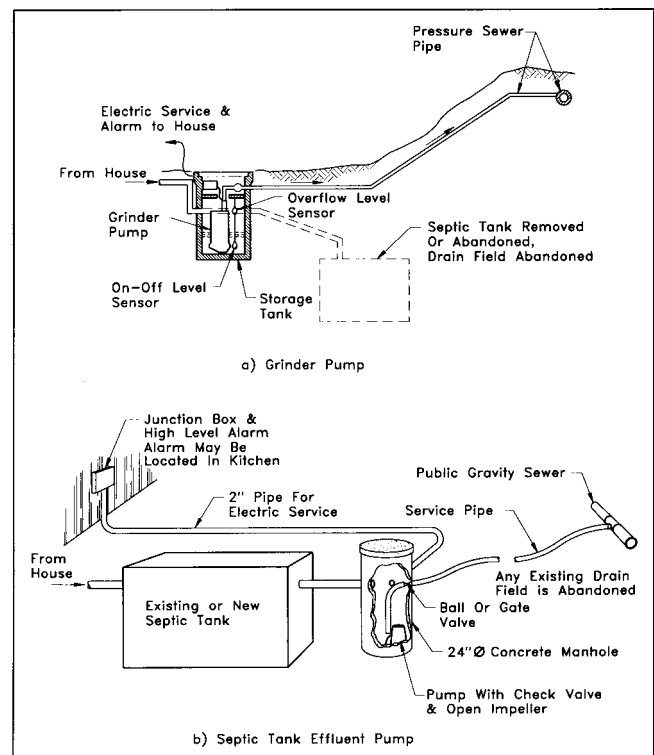
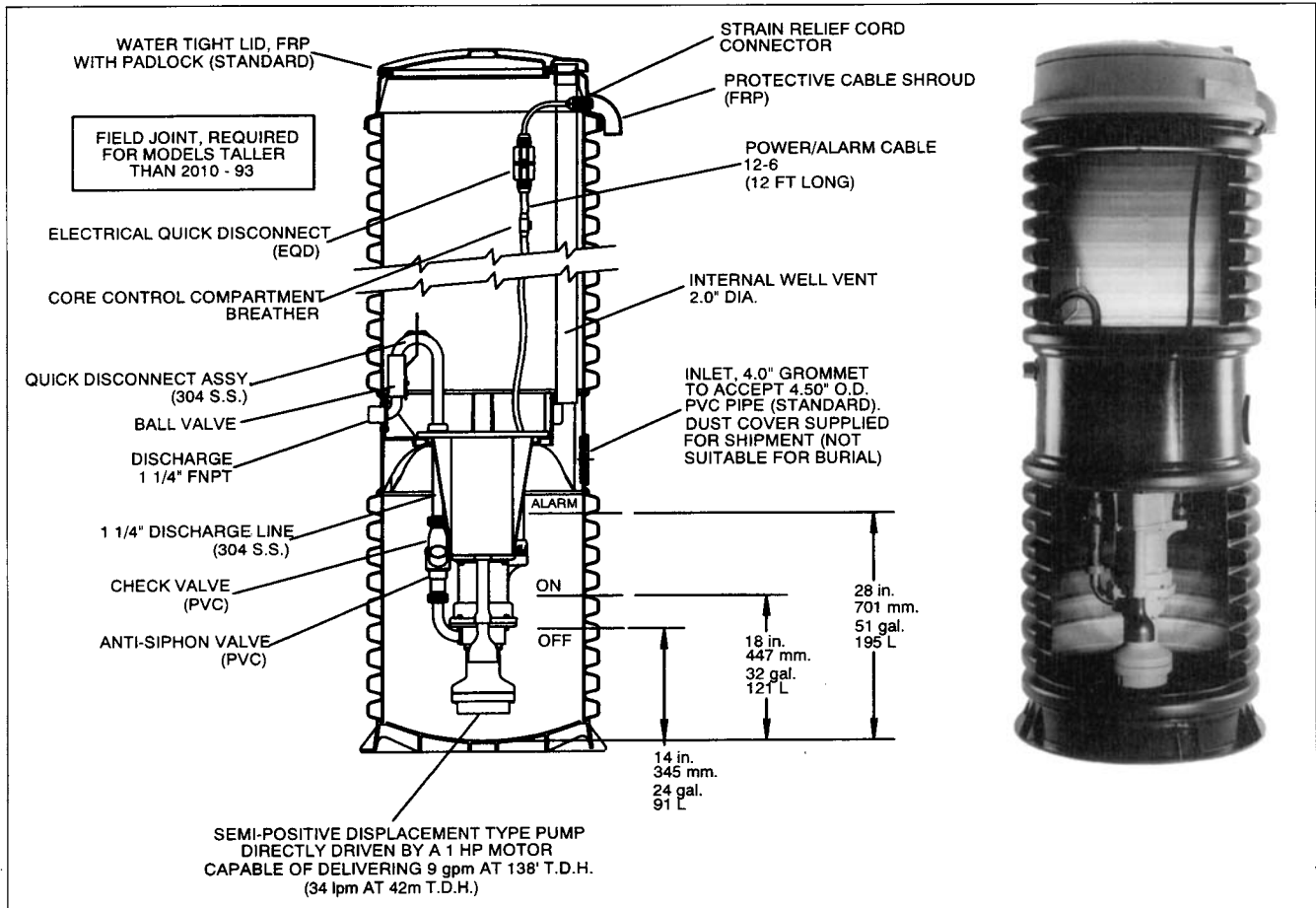


FIGURE 25.32 Flow diagram for pressure pumping systems.



**FIGURE 25.33** Typical grinder pump complete with housing. Reprinted with permission from Environment One Corp. 1994, *Grinder Pumps*, Environment One product catalog, Schenectady, N.Y.

This constitutes some wastewater storage availability during periods of power outages. The units are wired to permit placing a high water alarm at an appropriate location in the home, usually in the kitchen, so that the homeowner is made aware that the pump is not operating and that the wet well is full. The pumping units are designed for easy removal of the entire unit in case of pump failure. The pumps should give ten or more years of service under normal conditions. It is desirable for the community with a GP system to have service provided by a utility or by a private plumbing company that will keep spare pumps in stock for rapid installation. The non-functioning pump is then taken to the shop for repair or replacement and kept in stock for the next replacement need. If rapid service is not available, the homeowner should install the duplex unit so that a back-up pump is always available, allowing time to have the non-functioning unit repaired.

The septic tank effluent pressure system (STEP) is a means of eliminating the need for on-site treatment and disposal, such as the soil absorption field. In past years, septic tank absorption fields have been installed at locations that are no longer environmentally acceptable. Criteria for siting absorption fields have improved as more knowledge

on soil percolation and potential for groundwater pollution has become available. The STEP systems are being installed to eliminate failing absorption fields and as a means of providing central sewerage service to both existing and new communities. The septic tank located ahead of the pump eliminates most of the grease and solids in the flow to be pumped. The force main design is similar to the GP system except for the type of pump required. The homeowner must continue to maintain a septic tank as a part of the STEP design.

#### REFERENCES

- Babbitt, Harold E., *Sewerage and Sewage Treatment*, John Wiley and Sons, Inc., New York, 1953.
- Certainfeed Products Corporation, *Certainfeed Product Catalog*, Certainfeed Products Corporation, Ambler, Pa., 19002.
- County of Fairfax, *Public Facilities Manual*, County of Fairfax, Fairfax, Va., 22031, 1988.
- Environment One Corp., *Grinder Pumps*, Environment One Product Catalog, Environment One Corp., 2773 Balltown Road, Schenectady, NY, 12309.
- Flygt Corporation, *Wastewater Pumps*, Catalog of Flygt Corporation, 129 Glover Avenue, Norwalk, Conn., 06856.

Hydromatic Pumps, *Sump, Sewage and Wastewater Pumps*, Catalog of Hydromatic Pumps, 5800 Foxridge Drive, Mission, Ks., 66202.

ITT A-C Pump, *Pumps*, Catalog of ITT A-C Pump, 1150 Tennessee Avenue, Cincinnati, Ohio, 45229.

Johns-Manville Manufacturing Company, *Pipe and Pipe Products Division Catalog*, Johns-Manville Manufacturing Company, Inc., 1051 Sperry Road, Stockton, Ca., 95206.

Multi-Flo Waste Treatment Systems, *Engineering and Technical Manual*, Multi-Flo Waste Treatment Systems, Inc., 2324 East River Road, Dayton, Ohio, 45439.

National Association of Home Builders, *Alternatives to Public Sewer*, National Association of Home Builders, 15th and M Streets, N.W., Washington, D.C., 20005, 1978.

Pollution Control Federation, *Design and Construction of Sanitary and Storm Sewers*, WPCF Manual of Practice No. 9, Water Pollution Control Federation, Alexandria, Va., 1969.

Portland Cement Association, *Design and Construction of Concrete Sewers*, Portland Cement Association, Old Orchard Road, Skokie, Il., 60076, 1968.

Prince William County Service Authority, *Sewer Standards and Design Manual*, Prince William County Service Authority, P. O. Box 2266, Woodbridge, Va., 22193-0266.

Uni-Bell Plastic Pipe Association, *Handbook of PVC Pipe*, Uni-Bell Plastic Pipe Association, 2665 Villa Creek Drive, Suite 150, Dallas, Tx., 75234.

Washington Aluminum Company, *Components for Water and Wastewater Plants*, Catalog of Washington Aluminum Company, Knecht Avenue, Baltimore, Md., 21229.



## CHAPTER 26

# WATER DISTRIBUTION

*John Matusik, P.E. / Revised/Updated: Gary L. Nickerson, P.E.*

### OVERVIEW OF A WATER SUPPLY SYSTEM

A water-supply system includes the water supply, treatment facilities, pumping facilities, transmission lines, and the local distribution network. The distribution network consists of pipes, fittings, valves, and other appurtenances designed to convey potable water at adequate pressures and discharges. Adequate discharges are necessary to cover the range of flows caused by the fluctuation in demand. Adequate pressures are necessary for fire fighting, general service to residents, and economic considerations such as leakage and energy losses. The pressures within a water-supply system must be high enough to overcome the energy losses in the system, yet maintained below the point at which damage to fittings and other appurtenances occur. Pressures and discharges are a function of the hydraulic characteristics of the system, as well as the system's service characteristics. Service characteristics include such items as demand, as it relates to present and projected population, economic base, fire flow, and climate. Hydraulic characteristics include the length, size and condition of the pipe, types and sizes of fittings and appurtenances, and variations in elevation of the system. Since service and hydraulic characteristics constantly change, the design must incorporate allowances for these changes. Dynamic demographics of the users and deterioration of the system affect how and when the changes are made. The changes that can be made include relocating or replacing the existing water line, increasing the pipe sizes, addition of auxiliary pumps and storage, or adding more distribution lines to accommodate new development.

The water utility company is responsible for the water quality and operation of the distribution system. A water

utility company can either be a public entity that, like many other public bureaus, exists for the health, safety and welfare of the public, or it may be a privately-owned utility providing water for profit. In some of the larger metropolitan and suburban areas, the water utility company heavily controls the design of a water distribution system. Personnel within the water company perform most of the design, analysis, and layout. In this case the site designer of the land development project only performs the lesser water-works design aspects and incorporates his analysis, along with the water company's information, into the construction drawing set. In smaller locales, the analysis and design may be performed entirely by the private consultant working for the developer. Typically, the water-supply company, to ensure conformance with local standards, reviews the private consultant's design. Regardless of whether the water-supply company is public or private, the design must conform to the state health department's criteria, as well as the criteria of other controlling public agencies. Design parameters and regulations for water works are available from state boards of health and local city/county health departments. Additional information on water quality, as well as material and construction standards, are available from the Environmental Protection Agency (EPA), the American Water Works Association (AWWA) and the American National Standards Institute (ANSI).

The overall water-works system consists of the following elements:

- The *water source*, usually a lake, river or aquifer, serves as the municipality's main supply. Larger municipalities may have more than one source. Lakes and reservoirs are located in outlying areas to take advan-

tage of runoff from large catchments and where there is less pollution. Along rivers, water is usually extracted on the upstream side of the population centers. The minimum supply volume at the source must coincide with the present and projected demand that it is to serve.

- A *treatment facility* purifies the water for safe use. Since pumping clean water is less expensive than pumping sediment-laden water, the location of the filtration plant should be near the source. The treatment facility treats and disinfects the water to meet the water-quality standards set by government regulations. The law mandates that the water-supply company provide quality potable water and consumers expect and rely on the water company to do so.
- *Transmission lines* convey the water leading from the source to the treatment facility, as well as from the treatment facility to the distribution network. For moderate to large population areas, transmission lines can be 6 ft in diameter (and larger) and operate at pressures in excess of 200 psi. They serve as the link between the source, the treatment facility, and the distribution network. Therefore, transmission lines have limited branches and service taps.
- *Pumping facilities* provide necessary energy to move the potable water to consumers. Pumping facilities are needed to convey the water through transmission lines to the distribution system. These pumping facilities can be simple, such as the well pumps in small systems, or they can be complex, high-capacity pumping stations needed for large municipalities. Pumping stations within the distribution network, called booster stations, are used to maintain required minimum pressures.
- *Intermediate storage facilities* (e.g., water tanks) are located near the distribution network. Intermediate storage facilities stabilize the line pressures, serve as reserve for peak demand periods and storage for fire flow requirements. All domestic water line systems operate under pressure. In times of high demand, the pressure in the network is decreased, requiring the water supply to be supplemented from the storage tanks along with the water from the treatment plant. During low-demand periods, when supply pressure in the distribution lines is high, the storage tanks refill. Discounting fire flow storage, water-storage volumes fluctuate 40% to 70% on a daily basis.
- *Distribution lines* are used to link the intermediate storage facilities and the feeder lines that connect the residential, commercial, and industrial service areas. Typically, distribution networks are laid out as interconnecting loops. The looped (grid) layout allows for bi-directional flow of water. Since water travels towards the area with the lowest pressure, which is often the

area of highest demand, looping delivers twice the volume of water for fire flows and other heavy demands. Additionally, grid layout allows for isolation of small areas during repairs. A branching type of configuration consists of a main feeder line with single dead-end branches to service areas. In comparison to the grid system, the branch system is not as economical for several reasons: (1) closing down a branch for maintenance disrupts service to a larger area; (2) poor water quality resulting from stagnation and sedimentation in the branch ends means periodic flushing at the ends. Although dead ends cannot be avoided, they should be minimized. Fire hydrants, flushing hydrants, or blow-off valves at the dead ends allow for such periodic flushing.

- *Appurtenances* such as fire hydrants, valves, auxiliary pumps, and fittings (wyes, elbows, crosses, and tees) augment the operation of the distribution system. The arrangement of the appurtenances and fittings is specific to the local area they serve. Valves allow the system to isolate small service areas when repairs are needed. Other fittings are used to change flow direction and provide economic flexibility in sizing pipes.

The design of the entire system requires an evaluation of numerous factors. Location and size of main water supplies depend on general characteristics of the region such as climate, hydrology, geology, and topography. Design of treatment facilities, transmission and distribution lines depends on existing water quality at the source, existing and projected population and its spatial distribution. Additionally, as the municipality grows, the increasing population and expanding development inevitably influences hydrologic and topographic factors as well as others, and these in turn affect water demand.

## WATER USE AND DEMAND

### Demand Forecasting

Water use is categorized as either consumptive use or non-consumptive use. Consumptive use includes municipal, agricultural, industry and mining, whereas non-consumptive use refers to water used for hydropower, transportation, and recreation. Water demand is the quantity that consumers use per unit of time. Land development projects involve analysis of municipal water uses such as residential, commercial, institutional, industrial, fire fighting, swimming pools, and lawn and park watering.

Demand forecasting projects the future water use based on previous water use, socio-economic trends, climatic factors, and other parameters. Various types of forecasting models may be used to estimate water demand. The least complex of these is a single coefficient method, whereby the projected demand is based on a factor such as per capita, per connection, or by land use. Complex models may factor in water pricing, income, and even statistical analysis.

The selection of the model depends on the type and size of the project and the availability of the data.

Part of the design of a water-distribution system includes estimating the amount of water needed for a service area. Water demand for an area depends on population, climate, industry, and economic factors. Additionally, the design analysis must include fire flow requirements. Although the overall volume of water used for fighting fires is quite low relative to most other uses, the amount of water required to fight a fire, even if only for a few hours, puts a heavy strain on the system. Since fire flow requirements are so high relative to other uses, they are usually the controlling criteria in the design of distribution piping and storage. In part, the water-works system design is based on an estimated average demand and maximum daily demand. However, variations in demand must also be included as part of the design analysis.

Variation in water use becomes apparent by considering hourly and daily individual personal water use. Each individual drinks, bathes, and generally uses varying daily amounts of water. On an hourly basis, this variation fluctuates substantially. Extend the individual fluctuations in hourly and daily use over a population of tens of thousands and it becomes apparent why water demand projection is difficult. Fortunately, individual daily use tends to an average value exhibiting only slight variations over longer periods. Variations in short-term use are further reduced as population and development stabilizes. However, the capricious variables (e.g. climate, demand for consumer goods and services) inherent in commercial, manufacturing and agricultural industries can add to wide variations in use and complicate demand forecasting.

The types of use for water demand include domestic, commercial, industrial, mining, agricultural, thermoelectric power, public use, and many others depending on where the interest is. Collecting and sorting the data to determine "average use" is difficult, cumbersome, and inexact. Water-supply companies keep numerous records of water withdrawals and metered use. Inconsistencies in how each supplier defines various uses, or categorizes the various uses, leads to the inexactness in determining "average use" figures, when compared over state-wide areas. The point here is that it is difficult to say confidently that a certain value is applicable as a blanket design value for all areas of the U.S. For preliminary estimates, a blanket design value is appropriate, but for specific design, regional data is more appropriate.

### Residential Demand

Domestic water use refers to the water used for household purposes, including water for drinking, cooking, sanitary needs, landscape watering, swimming pool maintenance, street cleaning, fire fighting, leakage, and system maintenance. Residential water demand widely fluctuates, on both a daily basis and a yearly basis. Daily, the water demand has two peaks. Once in the morning hours of 7–11 A.M.

and another peak in the evening between 4–8 P.M. Although the peaks remain fixed for the time periods, the amount varies according to season and yearly rainfall. Average daily demand for domestic use ranges from 60 gallons per capita per day (gpcd) to 130 gpcd. However, studies have shown that average water use decreases as population increases.

The USGS presents data on water use in the report "Estimated Use of Water in the United States in 1995." The data from this report shows public-supplied domestic water use ranging from 53 gpcd in Wisconsin to 212 gpcd in Nebraska, with the average water use for all states, as well as Washington D.C., being 179 gpcd.

### Commercial Demand

Commercial water use refers to the water used in motels, hotels, restaurants, office buildings, shopping centers, and other commercial establishments. Compared to residential demand, commercial demand is considerably less than residential peak demands and not as varied. Note: most of the water use for office buildings is for air conditioning purposes. Restaurants, hotels, and motels use water mainly for cooking and cleaning. Water-use estimates for design purposes are usually based on number of square feet of space, potential number of occupants, or the number of water fixture units in the building. Utilizing data available in the above-noted USGS report, it can be deduced that commercial use, relative to total use, ranges anywhere from 1% in Iowa to nearly 41% in Alaska. Table 26.1 provides values for water use for various types of uses. At first glance the values might appear to be high relative to what an individual perceives to be his actual use. However, these values account for unmetered water use such as fire fighting, maintenance (e.g., flushing the system), leakage and illegal connections.

### Industrial Use

Industrial use accounts for the water used for fabrication, processing, washing and cooling associated with such industries as food processing, steel production, chemical processing, mining, and petroleum refining. Note: some industry plants install their own water-supply systems, rather than rely on the municipal water supply. Depending on the industry and the capacity of the municipal water supply, it may be necessary for the industry to furnish its own water supply. The amount of water used varies greatly with the type of industry. An industry such as manufacturing of woolens uses 140,000 gallons per ton of woolens produced, whereas a ton of tanned leather requires 16,000 gallons. The USGS data indicates that the state with the highest industrial use (as a percent of total use) is Kentucky, which uses 41% towards industry.

The wide fluctuations in water use for these three categories are evident from the data listed in Tables 26.2 and 26.3.



**TABLE 26.1 Daily Water Consumption Rates**

TYPE OF LAND USE	WATER CONSUMPTION (gal/day)
Dwellings, per person	100
High schools with showers, per person	16
Elementary schools without showers, per person	10
Community colleges per student and faculty	15
Motels @ 65 gals/person, minimum per room	120
Trailer courts @ 3 persons/trailer, per trailer	300
Restaurants, per seat	50
Interstate or through highway restaurants, per seat	180
Interstate rest areas	5
Shopping centers, per 1,000 sf of ultimate floor space	200–300
Theaters, auditorium type, per seat	5
Hospitals, per bed	300
Laundromats, 9 to 12 machines, per machine	500
Factories per person per 8-hour shift	15–35

Source: Virginia Dept. of Health, Waterworks Regulations (1993).

### Other Uses

Additionally, water use can be identified with other types of industries. Depending on the region, the water use associated with mining, irrigation, livestock, and thermoelectric power may heavily influence design considerations for a water system. Note: this section of the book will only be concerned with residential and commercial demand; information on other types of water demand dependent on the characteristics of specific localities.

### Peak Demand

Distribution system design must account for the peak periods of daily use. The maximum daily use for domestic water ranges between 1.5 to three times greater than the

average daily use, whereas the maximum hourly use for domestic water use is two to five times greater than the average annual use. Since peak factors are a function of present population, population growth rate and land use, the older stabilized areas tend to have peak factors that are two to three times lower than rapidly expanding areas. Table 26.4 shows the relationship of population on peaking factors.

Typically, distribution lines are sized for design flows of maximum daily consumption plus the required fire flow. However, if the maximum daily consumption plus fire flow is less than the estimated maximum hourly amount, the design may be based on the latter value. Local regulations and state waterworks design manuals dictate minimum design flows.

**TABLE 26.2 States with Highest and Lowest Water Use**

	3 HIGHEST STATES (gpcd)			3 LOWEST STATES (gpcd)		
Domestic	Nevada (212)	Utah (184)	Idaho (181)	Wisconsin (53)	Ohio (54)	Pennsylvania (62)
Commercial	D.C. (90)	Nevada (81)	Utah (62)	Kentucky (7)	Texas (7)	Missouri (14)
Industrial	Alabama (62)	Kentucky (59)	Wisconsin (42)	D.C. (1)	Nevada (2)	Montana (2)
Total	Nevada (324)	Utah (269)	Washington (266)	Massachusetts (130)	Rhode Island (132)	W. VA (134)

**TABLE 26.3 Public Supply Freshwater Use by State (from USGS Circular 1200)**

<b>STATE</b>	<b>DOMESTIC (gpcd)</b>	<b>COMMERCIAL (gpcd)</b>	<b>INDUSTRIAL (gpcd)</b>	<b>PUBLIC USE AND LOSSES (gpcd)</b>	<b>TOTAL PER CAPITA USE (gpcd)</b>
Alabama	117.7	35.6	62.1	27.4	236.7
Alaska	99.7	60.3	31.5	21.0	211.6
Arizona	134.2	34.4	16.8	20.7	206.1
Arkansas	96.5	29.0	28.5	36.5	190.5
California	121.6	32.6	9.3	20.6	184.1
Colorado	141.9	29.8	5.6	29.6	203.8
Connecticut	75.5	35.2	16.6	27.7	154.9
Delaware	76.2	35.5	28.4	19.5	159.6
D.C.	171.5	90.3	1.3	0	263.1
Florida	103.3	31.6	8.4	25.6	168.9
Georgia	106.6	28.5	32.9	27.3	195.3
Hawaii	117.0	42.0	5.0	27.7	191.7
Idaho	180.8	23.1	8.6	29.5	241.9
Illinois	90.0	42.3	11.4	31.2	174.8
Indiana	76.2	27.8	29.2	23.1	156.3
Iowa	64.7	30.2	36.3	40.9	172.1
Kansas	82.3	28.9	16.0	32.0	159.2
Kentucky	69.9	6.9	58.6	12.5	147.9
Louisiana	121.6	14.3	9.1	20.8	165.7
Maine	65.0	35.3	19.8	19.8	139.8
Maryland	103.8	20.4	10.6	65.0	199.8
Massachusetts	64.9	33.7	15.4	15.8	129.8
Michigan	90.3	36.7	39.1	22.3	188.4
Minnesota	71.6	30.8	12.3	30.8	145.5
Mississippi	109.7	14.6	8.9	17.7	150.9
Missouri	86.4	13.6	32.3	28.9	161.2
Montana	119.4	40.3	1.6	60.5	221.7
Nebraska	120.2	61.3	20.2	20.2	221.7
Nevada	212.5	80.6	1.5	29.2	323.8
New Hampshire	81.8	30.1	18.7	9.6	140.2
New Jersey	77.6	25.8	13.1	29.3	145.9

**TABLE 26.3 Public Supply Freshwater Use by State (from USGS Circular 1200) (Continued)**

STATE	DOMESTIC (gpcd)	COMMERCIAL (gpcd)	INDUSTRIAL (gpcd)	PUBLIC USE AND LOSSES (gpcd)	TOTAL PER CAPITA USE (gpcd)
New Mexico	136.2	56.5	10.9	21.7	225.4
New York	111.7	25.3	22.0	26.2	185.1
North Carolina	69.9	29.1	40.6	22.1	161.7
North Dakota	81.8	30.7	5.1	30.7	148.3
Ohio	53.6	38.3	38.3	23.0	153.2
Oklahoma	82.3	58.0	41.6	11.6	193.5
Oregon	135.8	36.7	33.0	28.8	234.4
Pennsylvania	61.8	24.1	21.3	63.4	170.6
Rhode Island	64.9	22.8	13.7	29.6	131.0
South Carolina	135.3	18.4	16.2	29.8	199.6
South Dakota	86.4	34.9	13.1	11.8	146.2
Tennessee	80.3	48.2	29.4	17.7	175.8
Texas	139.2	7.4	15.2	23.4	185.2
Utah	183.8	62.2	9.2	13.5	268.7
Vermont	82.6	24.4	24.4	17.5	148.9
Virginia	85.5	30.7	17.7	24.4	158.3
Washington	127.6	36.3	74.7	27.5	266.1
West Virginia	72.7	17.4	10.6	33.3	134.1
Wisconsin	53.1	31.2	42.4	41.6	168.3
Wyoming	157.0	46.5	7.0	49.4	259.9
<b>AVERAGE USE</b>	<b>101.8</b>	<b>30.0</b>	<b>21.4</b>	<b>26.2</b>	<b>179.4</b>

Reference: U.S. Geological Survey, Estimated Water Use in 1995 (2000).

Since the estimated water-use design values are based on immediate as well as future expectations in growth and development, economic assessments are made to determine the long-range plan of the system. The decision may be to design the system for the short-term future and improve the system later rather than size and build the system for long-range future projections.

## DESIGN REQUIREMENTS

### Pressure Requirements

Pressure requirements in the system depend on the combined normal service requirements and fire flow demand.

Pressures must be high enough to overcome energy losses within the distribution and service lines, as well as the losses incurred from hydrants, nozzles, hoses, and other fire fighting equipment. Additionally, pressure requirements are a function of the topography. Since most water lines are installed at minimum depth, they closely follow the ground surface topography. Consequently, pressures in the system must be increased in the hilly service areas to overcome elevation differences. A balance must be struck between the pressures required for normal service operations and the pressures required for short-term high demand and emergency occasions. Although the water-supply system can be

**TABLE 26.4 Estimated Peak Ratios for Residential Water Systems**

POPULATION	RATIO OF MAXIMUM DAY TO AVERAGE ANNUAL USE	RATIO OF MAXIMUM HOUR TO AVERAGE ANNUAL USE
0–500	3.0	4.50
500–1000	2.75	4.13
1000–2000	2.50	3.75
2000–3000	2.25	3.38
3000–10,000	1.90	2.85
10,000–25,000	1.80	2.85
25,000–50,000	1.80	2.70
50,000–75,000	1.75	2.62
75,000–150,000	1.65	2.48
over 150,000	1.50	2.25

Reprinted by permission of R. V. Anderson Associates Limited, Willowdale, Ontario.

Reference: Recommended Guidelines for Residential Servicing in Ontario, Ontario Housing Advisory Committee (1973).

designed to operate at higher pressures, sustained excessive pressures add to system costs by increasing leakage volumes and the potential for damage to fixtures.

A municipality's exact pressure requirements are established by a state agency, such as the Health Department, with supplemental requirements imposed by agencies within the municipality such as the Fire Marshall. Typically, street water-main pressures of 20 psi are the minimum when providing either maximum daily demand (including fire flow requirements) or the peak hour demand. This is a precautionary measure, designed to prevent potential cross-connection backflows from house fixtures. This minimum pressure also represents the minimum pressure required to overcome frictional losses in the service line and push the water up three stories—the upper height limit typical in residential areas.

Preferable normal pressures in the street should be between 40 psi and 60 psi in residential areas. In most cases, a pressure of about 80 psi is recommended as the upper limit. Above 80 psi, special pressure-reducing valves may be necessary to prevent residential fixture damage. Typically, commercial areas require a minimum pressure of 75 psi. Booster pumps for tall buildings (e.g., more than ten stories) eliminate the need for excessively high pressures in the street mains.

### Fire Flow Requirements

Each municipality establishes its own parameters for fire flow requirements based on local conditions. The municipality may, however, instead refer to the fire flow recom-

mendations provided by the Insurance Services Office (ISO). The ISO recommends criteria for establishing insurance rates and for classifying municipalities with reference to their fire defenses and physical conditions. The criteria provides a means for rating a municipality based on type of fire fighting equipment, proximity of equipment, water distribution flows, storage and other factors. One such document by the ISO is the "Guide for Determination of Required Fire Flow." Although the guide only recommends fire flows, many areas defer to it for fire flow considerations, while others have established their own requirements. Minimum fire flows vary widely among jurisdictions. The design engineer should consult with the local water authority and Fire Marshal's Office for specific criteria.

### Estimating Fire Flow

The ISO uses the following equation to estimate the required fire flow:

$$F = 18 \times C \times \sqrt{A} \quad (26.1)$$

where

$F$  = required fire flow in gpm,  
 $C$  = coefficient related to the type of construction,  
 $A$  = total floor area in sf; including all stories but excluding basements.

For fire-resistive buildings, consider the six largest successive floor areas if the vertical openings are unprotected; if the vertical openings are properly protected, consider

only the three largest successive floor areas. Coefficient  $C = 1.5$  for wood-frame construction, 1.0 for ordinary construction, 0.9 for heavy timber type buildings, 0.8 for non-combustible construction, and 0.6 for fire-resistive construction. Coefficients shall not be greater than 1.5 nor less than 0.6 and may be determined by interpolation. Such interpolation shall be between consecutive types of construction as listed above. Additionally, these guides recommend a minimum fire flow of 500 gpm and maximum fire flows under the following conditions: 8000 gpm for wood-frame construction, ordinary and heavy timber construction; 6000 gpm for non-combustible construction and fire-resistive construction. For a normal one-story building (of any type of construction), the fire flow shall not exceed 6000 gpm. Table 26.5 identifies required fire flows for areas of single-family and small two-family dwellings not exceeding two stories in height.

Adjustments to recommended fire flow values might be applicable for certain occupancy conditions. Equation 26.1 may be reduced up to 25% for occupancies having a low fire hazard or increased up to 25% for occupancies having a high fire hazard. Examples of occupancies considered low fire hazard are residential dwellings, churches, schools, hotels, hospitals, and other public buildings. Occupancies of high fire hazard are manufacturing and processing plants using explosives and high combustibles, such as oil refineries, paint shops, and aircraft hangers. Other modifications that affect the required fire flow rate include the existence of sprinkler systems, non-combustible construction, and building separation. However, the range of adjustment to the initial fire flow estimates cannot be greater than 75%, and the final adjusted flow cannot be less than 500 gpm, nor exceed 12,000 gpm.

Fire flow estimates resulting from applying the above criteria are only a guide and warrant the judgment of experienced and knowledgeable persons in fire protection.

Typically, local jurisdictions require fire flow approval by controlling agencies, such as the Fire Marshal's Office, public water supply agency, and health department. Exact requirements by local jurisdictions for fire flows may not be the same as what is recommended by the ISO. In fact, there can be a wide range of fire flow requirements even among adjacent jurisdictions.

**Fire Flow Tests**

Before new projects ultimately get approval for construction by the Fire Marshal's Office, fire flow tests are performed to determine the fire flow capacity and adequacy of the existing system. Fire flow tests are used to determine available flow rates and pressures at various locations for fire fighting purposes. Although, in the larger municipalities, the water utility company has extensive data on the existing water system, uncertainties in the hydraulic variables and service variables requires further data to corroborate existing information. On commercial sites and small subdivisions, the test is performed on hydrants near the point where the proposed water main will connect to the existing water main. On larger sites, where the project is built in phases, fire flow tests may be required near the connection points at each subsequent phase of the development.

The following discussion on testing procedures and analysis is extracted from the National Fire Protection Association (NFPA). The exact method and analysis may deviate according to local regulations.

The test consists of discharging water at a measured flow rate from one or more hydrants and observing the resulting pressure drop in the main through another fire hydrant. Typically, a minimum residual pressure of 20 psi in the main is required (when delivering fire flows) for two basic reasons: first, fire department pumpers require 20 psi in order to operate effectively; second, a minimum residual pressure is needed to prevent a vacuum (negative pressures) from developing in the system that can cause collapse of pipes and inadvertent back-siphonage of polluted water.

For the test, one base point hydrant is selected as the residual hydrant. The residual hydrant is chosen such that the hydrants to be flowed are between it and the larger mains. Typical patterns for selecting the test hydrants are illustrated in Figure 26.1. Note: the residual pressure at this location is measured before other hydrants are opened. The pressure at this location is then determined when the other hydrants are fully opened. The test is made during a period of ordinary demand such that the pressure drop in the residual hydrant is 25% or less of what it would be at the total demand necessary for fire fighting purposes.

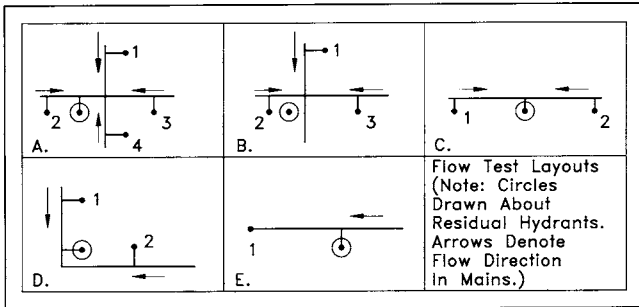
A pressure gage is placed on one of the hose outlets on the residual hydrant and the hydrant valve is fully opened. After the barrel air is exhausted, the pressure is recorded. The other hydrants are opened full in succession one at a time. After each hydrant is opened and the flow stabilized, the pressure is recorded at the residual hydrant and the discharge at each open hydrant is measured using a pitot gage. The discharge  $Q$  (gpm) is determined from

**TABLE 26.5 Required Fire Flows for Single-Family and Small Two-Family Dwellings**

DISTANCE BETWEEN BUILDINGS	SUGGESTED REQUIRED FIRE FLOW
Over 100 ft	500 gpm
31-100	750-1000
11-30	1000-1500
10 or less	1500-2000*

From the publication "Guide for Determination of Required Fire Flow," 2nd ed. Reprinted with the permission of Insurance Services Office, Inc. Copyright 1974.

\*If the buildings are continuous use a minimum of 2500 gpm  
Where wood shingle roofs could contribute to spreading fires, add 500 gpm



**FIGURE 26.1** Suggested test layout for hydrants. From the publication “Grading Schedule for Municipal Fire Protection.” Reprinted with the permission of Insurance Services Office, Inc. Copyright 1974.

$$Q = 29.83 cd^2 \sqrt{p} \tag{26.2}$$

where

$c$  = is the coefficient of discharge of the pitot tube (depends on the manufacturer’s specifications—typically around 0.9),

$d$  = is the diameter of the outlet hose connection in inches, and

$p$  = is the velocity pressure in psi (i.e., the pitot tube reading).

After all observations are recorded, the discharge (in gpm) at the specified residual pressure or for any pressure drop is

$$Q_R = Q_F \times \left( \frac{h_r}{h_f} \right)^{0.54} \tag{26.3}$$

where

$Q_R$  = flow predicted at the desired residual pressure,

$Q_F$  = total flow measured during the test,

$h_r$  = pressure drop to the desired residual pressure, and

$h_f$  = pressure drop measured during test.

As the equation is written, any consistent units for  $Q$  and  $h$  will work.

**Fire Duration Requirements**

In addition to providing adequate discharges and pressures for normal consumption, a water supply system must be designed to provide water for sustained periods for fire fighting purposes. Since the magnitude, duration, location, and frequency of a fire are quite unpredictable, the design of a system should be based on a reasonable worst-case scenario.

The ISO defines the average daily consumption as the total amount of water used each day during a 1-yr period considered as a rate for a 24-hr period. The maximum daily consumption is defined as the maximum total amount used during any 24-hr period in a 3-yr period, disregarding any unusually high circumstantial amounts caused by filling storage tanks. An adequate water supply, according to the

ISO, can deliver the required fire flow for the duration given in Table 26.6 at the maximum daily rate.

**Distribution Storage**

Distribution storage, which consists of small reservoirs located near service areas, acts as service storage to compensate for widely fluctuating demands and to provide storage for fire fighting and emergency reserve. Distribution storage is used to normalize operating pressures, eliminate the need for non-uniform pumping, reduce the need for constructing large mains at the periphery of the system, and provide increased reliability for fire protection and other emergencies.

Whether ground storage or elevated storage tanks are used depends on such factors as topography, size of service area, reliability of main water supply, and economics. Auxiliary pumps at the ground storage facility or higher water surface elevations in the elevated storage reservoir can be utilized to equalize pressures during high-demand periods. Since short and frequent high-demand periods can require pumps to turn on and off regularly to maintain system pressures one advantage to elevated storage tanks (over auxiliary pumps) is the ability to supply a sustained pressure without the cyclic pump operations. However, the selection of the type of distribution storage should be considered on a case-by-case basis.

Although the need for design and construction of storage facilities is infrequent in moderately-sized land development projects, very large projects, or areas of unanticipated rapid growth, may warrant the need for distribution storage facilities. Nonetheless, the project engineer should be aware of the major factors governing the design of the facilities. In many cases, an engineer specializing in the design of

**TABLE 26.6 Required Fire Flow Duration**

REQUIRED FIRE FLOW (gpm)	REQUIRED DURATION (hr)
10,000 and greater	10
9500–9000	9
8500–8000	8
7500–7000	7
6500–6000	6
5500–5000	5
4500–4000	4
3500–3000	3
2500 and less	2

Source: Virginia Dept. of Health, Waterworks Regulations (1993).

water works will be called on to perform the specific design. In the projects where such facilities are required, the developer and jurisdiction may enter into some kind of joint venture where the costs would be shared by the two. This practice is common in instances where users beyond the project boundaries would share the facility.

The size of a storage tank depends on the characteristics of the service zone, fire demand requirements, and the local standards governing such design. General rule-of-thumb storage requirement is given as one day's supply plus fire flow. The 1-day supply is based on the maximum-day usage, while the fire flow storage is based on the duration as suggested by the ISO for the design fire flow (see Tables 26.5 and 26.6). Note: local design standards may supersede this method of storage demand design. Preferably, the tank should be sized to provide 2 to 3 days supply, although economic and risk analyses will dictate the uppersize limit.

Another consideration for storage tank sizing deals with the retention time of the water in the tank. Since a storage tank is designed to supplement the added demand during peak periods and to re-fill during off-peak periods, a properly designed tank under most operating conditions never completely empties. However, because of this fluctuation, there is a somewhat continuous exchange of water within the tank. With the residual effects of disinfectant chlorine limited to 6 to 10 days, the maximum residence time of water in the tank should be less than 8 days to prevent stagnation and possible health problems.

Formal design of a storage facility should incorporate the hourly-use hydrograph or mass diagram for the maximum day. A description of the design can be found in most basic water resources textbooks. One way to estimate storage requirements based on land use follows.

**EXAMPLE 1:** Table 26.7 shows the land use for a new service area and the corresponding demand for the average day for the type of land use. For this service area, fire flow requirements are 4 hours duration for a flow of 2500 gpm. Local ordinance requires a reserve of 20% of the maximum-day demand and the maximum-day demand is assumed to be equal to 1.6 times the average-day demand. Use a safety factor of 1.25. What are the storage requirements given these parameters?

Storage Required:

$$\begin{aligned}
 4\text{-hr fire flow @ } 2500 \text{ gpm} \times 60 \text{ min/hr} &= 600,000 \text{ gal} \\
 2,068,910 \text{ gal} \times 1.6 \times 0.20 &= 662,051 \text{ gal} \\
 \text{Subtotal} &= 1,262,051 \text{ gal} \\
 \text{Storage} = 1,262,051 \times (1.25) &= 1,577,564 \text{ gal} \\
 &\quad (26.4)
 \end{aligned}$$

Further considerations in this analysis would be to determine if one or several facilities would supply the required storage, the size of each facility, and the location.

## PUMPING FACILITIES

Pumping Facilities are necessary to transport treated water through the transmission and distribution lines to fill storage tanks and maintain system pressure. For small water utilities, the well pump or the single finished-water pumping arrangement at a treatment plant may be all that is required. For large municipal distribution systems, a complex network of pumping stations, monitored and controlled from a central point, may be needed.

### Pump Selection

Small water utilities commonly employ three types of pumps for water-distribution service: centrifugal, positive displacement, and jet (ejector).

**Centrifugal Pumps.** Centrifugal pumps are the most commonly used of the three. A centrifugal pump involves a casing containing a rotating impeller mounted on a shaft turned by a motor power source. Water that enters the suction side of the rotating impeller is thrown at high velocity against the casing to convert velocity head into pressure. Centrifugal pumps can be designed to provide stages in series, where each impeller and matching casing constitutes a stage, to realize pressures not attainable with a single-stage pump. Pumps having more than one stage are called multi-stage pumps, of which there are two types: submersible and turbine.

The submersible pump consists of one or more pump stages driven by a closely coupled motor designed for submerged operation (see Figure 26.2). It is commonly used in wells but may also be used in finished-water clear wells at treatment facilities.

Turbine pumps consist of one or more centrifugal pump stages driven by a vertical shaft, connecting the pumping assembly to a motor mounted at the surface. Figure 26.3 shows a turbine pump used for well service. Turbine pumps in this configuration are called vertical turbine pumps. Turbine pumps can also be mounted on treatment plant clear wells for distribution of finished water.

**Positive Displacement Pumps.** Positive displacement pumps, as the name implies, displace a set volume of water with each turn of the pump. The pumping rate varies with the speed of the pump. Although these pumps are not that common in water distribution service, especially in high-volume applications, they are well-suited for intermittent pumping at high pressure. Positive displacement pumps are available in several configurations:

- *Reciprocating pumps.* This pump consists of a plunger driven back and forth in a closely fitted cylinder, with check valves at both the inlet and pump outlet.
- *Helical rotor pump.* The helical pump consists of a spiral rotor that rotates in a sleeve. As the spiral rotor turns, it traps water between the rotor and the sleeve, forcing it to the outlet end of the sleeve.

**TABLE 26.7 Demand Requirements for New Service Area (Example 1)**

LAND USE	UNIT OF MEASUREMENT	No. OF UNITS	AVERAGE DEMAND (gpd)	TOTAL DEMAND (gpd)
Single-family detached	per dwelling	1682	350	588,700
Townhouse	per dwelling	519	350	181,650
School	per student	750	16	12,000
Daycare center	per student	200	16	3200
Recreation center	per person	1528	10	15,280
Hotel	per room	120	168	20,160
Restaurant	per seat	800	50	40,000
High-rise commercial	per acre	89.1	2000	1,782,000
Low-rise commercial	per acre	362	1500	543,000
			Sub-Total =	1,582,190
Existing demand				486,720
			TOTAL =	2,068,910

■ *Rotary gear pump.* This type of pump uses two meshing gears housed in a sealed casing that traps water in cavities between them. As the gears mesh, they carry water from the inlet to outlet port.

**Jet (Ejector) Pumps.** Jet pumps are actually a combination of centrifugal and ejector pumps (see Figure 26.4). A portion of the centrifugal pump discharge is diverted through a nozzle and venturi tube near the water level of the source. This creates a low-pressure zone that draws flow upward toward the surface, where the centrifugal pump suction can further lift it into the distribution system. These jet installations are economical for low-volume facilities and are commonly used in wells.

The fundamental considerations for pump selection within a particular installation are as follows:

- Yield of the well, or other water source
- Total daily needs and instantaneous demand of the system
- Total operating head pressure of the pumps at normal delivery rate, including lift and all friction losses
- Elevation difference between the ground level and the water level in the well during pumping
- Availability of power
- Ease of maintenance and availability of parts
- Initial and operations costs

- Reliability of pumping equipment

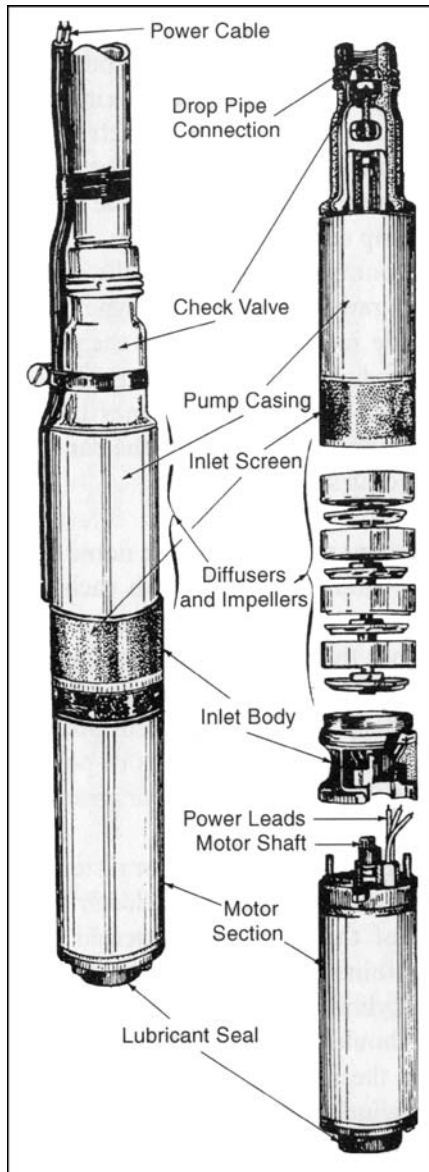
Table 26.8 provides information useful for selecting the type of pump needed for typical small system applications. Selecting the proper pumping unit size must be done by carefully evaluating the peak demand, available storage in the system, and capacity of the water source. Increased storage capacity is needed when source-yield is low in comparison with peak demand (as often happens with well sources). This is needed to avoid starting and stopping pump motors more than four or five times per hour in order to maintain optimum service life of the equipment.

### System Head Curve

The system head curve is an essential tool for use in determining the pump's type, size, and number of units to be applied for the most economically feasible installation. It graphically presents hydraulic head requirements for various flow characteristics of a particular system, making it easier to determine the appropriate configuration. Figure 26.5 shows an example of system head curve including a typical centrifugal pump curve.

The concept of total dynamic head must be understood to construct a proper system head curve for a potable water distribution pumping system. Figure 26.6 is a graphical representation of total dynamic head for a well pump installation. Static head, friction head, and pressure head are the three components of total dynamic head. Static head is the difference in elevation of the source water and the point

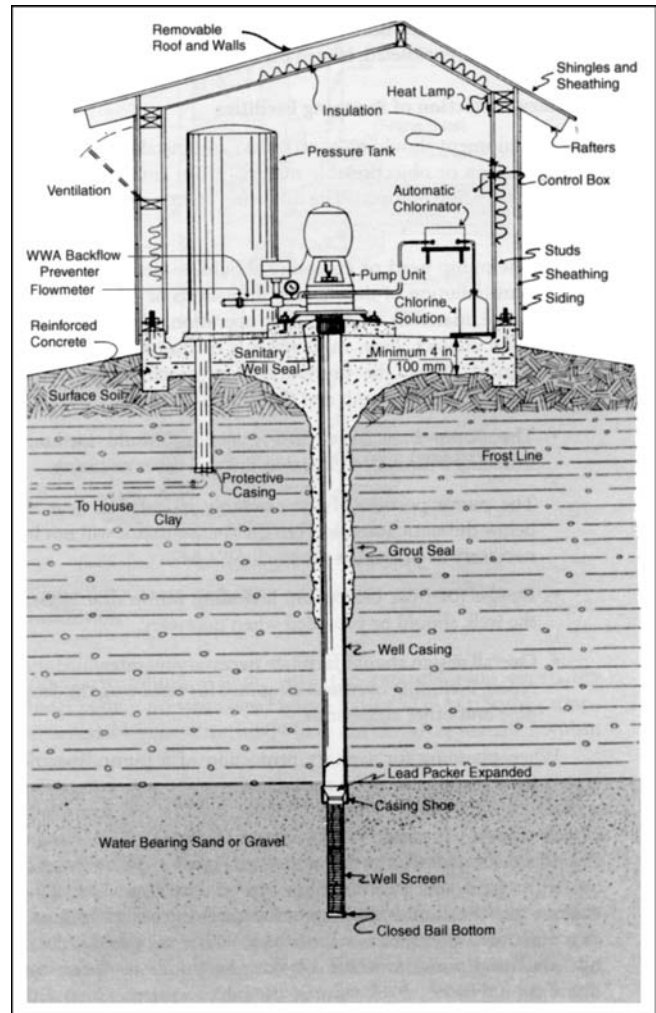




**FIGURE 26.2** View of a multistage submersible pump. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, AWWA).

of discharge. Friction head is the loss that occurs due to valves, fittings, and piping (discussed later in this chapter under System Analysis, Minor Losses). The pressure head includes the velocity head as well as the minimal residual pressure required in the system by state health departments (usually no less than 25 psi at the outlet fixture).

Water hammer, or hydraulic surge, occurs when water flowing in a pipe comes to an abrupt stop. A common example of this is when a pump shuts off suddenly because of a power failure. On such an occasion, the resulting surge can cause system pressure that is many times the normal working pressure. These high pressures, if not adequately accommodated by the design, can damage piping, valves, and pumping equipment. Lack of proper surge protection



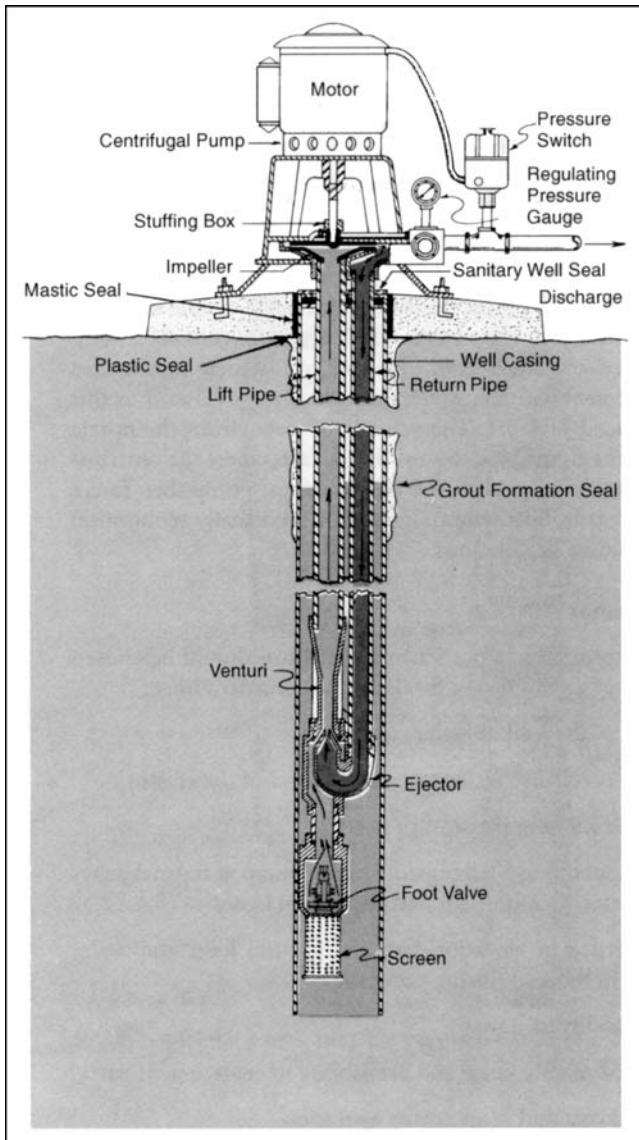
**FIGURE 26.3** Turbine pump in well service. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, AWWA).

can result in catastrophic failure, requiring several days of repair, while the utility customers are without water service. Surge protection must be carefully designed, even for small water utilities. Common surge protection methods include surge relief valves and pressure surge tanks located on the discharge main near the pumps, and vacuum relief valves located at key points on long force mains.

### Pump Drives

Pumps in the water utility industry are normally driven by alternating current (AC) motors. The power source for AC motors can be single-phase or three-phase. However, single-phase is generally limited to small motors of 10 hp (0.75 KW) or less. On a first cost basis, establishing a three-phase power source for a pumping facility is more expensive than it is with single-phase, although the three-phase is more efficient.

AC motors can be provided as either squirrel-cage induction, synchronous wound, or wound-rotor induction



**FIGURE 26.4** Combined centrifugal and ejector pump. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, AWWA).

types. However, squirrel-cage inductor types are most commonly used because they are less expensive than the other two and they have proven operation. In general, lower-speed motors are larger and more expensive than higher-speed motors, but they have a longer service life. Normally, the needs of small water utilities can be met through properly designed, constant-speed pumping arrangements of multiple pumps. However, two-speed, squirrel-cage motors are available.

Note that fairly recent improvements in technology, reliability, and costs of variable frequency drives (VFDs) make them worthy of consideration for small utility service. VFDs work by converting AC to DC and then back to AC but at a different frequency. Since the speed of a synchronous motor is directly proportional to frequency, varying the fre-

quency varies the motor speed. Because of this conversion process, VFDs produce considerable heat that must be properly vented or even addressed by the use of air conditioning.

### Packaged Booster Stations

Packaged booster pumping stations are very popular with small water utilities because of their economy and ease of installation. These units, which are delivered assembled to the construction site complete with controls and electrical, can be installed with minimal labor and time. Although these packaged pumping units can be used for primary finished-water service at treatment plants, they are more often used as booster stations out in the distribution system. These packaged stations can be ordered for either above or below-ground-level installations, as shown in Figures 26.7 and 26.8, respectively.

### Emergency Power

Electrical power to a treatment facility or pumping station can be disrupted by a storm, or other natural disaster, for many hours, or even for days. Many state health departments require that critical waterworks and pumping stations be protected against power failure by a standby power source. Normally, the standby power source is a connection to a second independent public power source (served by a separate power grid that serves the primary source), or a standby generator set.

As a minimum, generator sets should be sized to provide power for the average system demand, plus basic lighting and heating. Generator sets must be maintained and regularly exercised to ensure they will operate properly when called upon during an emergency. Although these sets normally operate on diesel fuel, units can be obtained that operate on propane, liquefied natural gas, or natural gas, if available. Both the generator set and its auxiliary fuel tanks must be properly secured against vandalism, and the fuel tank and its feed lines must be contained to prevent contamination from leakage.

There are several available options for securing and containing the generator set and its fuel tank. The first option is a custom-designed, built-in-place building. The advantages of such a building are that it can match the aesthetics of nearby buildings and surroundings, and it can provide ideal conditions for maintenance and repair of the set. Care must be taken to properly ventilate the building to prevent air-starving of the operating generator set. Typically, this is done with properly sized louver systems that are opened either by the force of the air being pulled toward the generator set or by electric motors that activate when the generator set is started.

Generator sets can also be provided by the manufacturer with a sound-attenuated enclosure made of steel or fiberglass, with removable panels to enable repairs (see Figure 26.9). The disadvantage of this is that repair personnel must stand in the elements to do their work. To facilitate repair,

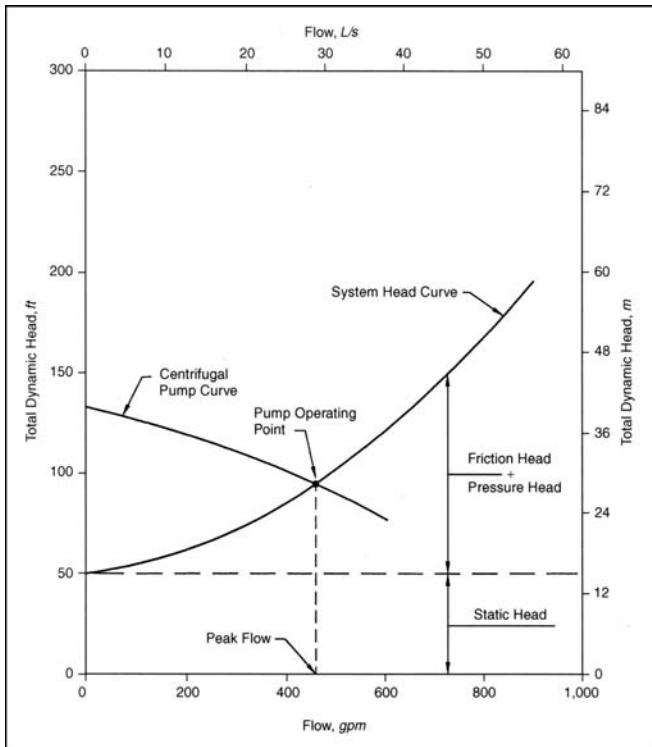
**T A B L E 2 6 . 8 Specifications for Various Types of Pumps**

TYPE OF PUMP	PRACTICAL SUCTION LIFT*	USUAL WELL-PUMPING DEPTH	USUAL PRESSURE HEADS	ADVANTAGES		DISADVANTAGES		REMARKS
Reciprocating: Shallow well	22–25 ft (7–8 m)	22–25 ft (7–8 m)	100–200 ft (30–60 m)	Positive action Discharge against variable heads	Pulsating discharge Subject to vibration and noise			Best suited for capacities of 5–25 gpm (19–95 L/min) against moderate to high heads Adaptable to hand operation Can be installed in very small diameter wells (2-in [51-mm] casing) Pump must be set directly over well (deep well only)
Deep well	22–15 ft (7–8 m)	Up to 600 ft (183 m)	Up to 600 ft (183 m) above cylinder	Pumps water containing sand and silt Especially adapted to low capacity and high lifts	Maintenance cost may be high May cause destructive pressure if operated against closed valve			
Centrifugal Shallow well Straight centrifugal (single-stage)	20 ft (6 m) maximum	10–20 ft (3–6 m)	100–150 ft (30–46 m)	Smooth, even flow Pumps water containing sand and silt Pressure on system is even and free from shock Low starting torque Usually reliable; good service life Same as straight centrifugal except not suitable for pumping water containing sand or silt Self-priming	Loses prime easily Efficiency depends on operating under design heads and speed			Very efficient pump for capacities above 60 gpm (227 L/min) and heads up to about 150 ft (46 m)
Regenerative vane turbine type (single-impeller)	28 ft (7 m) maximum	28 ft (7 m)	100–200 ft (30–60 m)		Same as straight centrifugal, except maintains priming easily			Reduction in pressure with increased capacity not as severe as with straight centrifugal
Deep well Vertical line shaft turbine (multi-stage)	Impeller submerged	50–300 ft (15–91 m)	100–800 ft (30–244 m)	Same as shallow-well turbine. All electrical components are accessible aboveground	Efficiency depends on operating under design head and speed Requires straight well large enough for turbine bowls and housing Lubrication and alignment of shaft critical Abrasion from sand			

\*Practical suction lift at sea level. Reduce lift 1 ft (0.3 m) for each 1,000 ft (305 m) above sea level. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, AWWA).

Jet									
Submersible turbine (multi-stage)	Pump and motor submerged	50–400 ft (15–122 m)	50–400 ft (15–122 m)	Same as shallow-well turbine Easy to frostproof installation Short pump shaft to motor Quiet operation Well straightness not critical	Repair to motor or pump requires pulling from well Sealing of electrical equipment from water vapor critical Abrasion from sand	3500 RPM models, although popular because of smaller diameters or greater capacities, are more vulnerable to wear and failure from sand and other causes			
Shallow-well	15–20 ft (5–6 m) below ejector	Up to 14–20 ft (5–6 m) below ejector	80–150 ft (24–46 m)	High capacity at low heads Simple operation Does not have to be installed over the well No moving parts in well Same as shallow-well jet Well straightness not critical	Capacity reduces as lift increases Air in suction or return line will stop pumping				
Deep-well	15–20 ft (5–6 m) below ejector	25–120 ft (8–37 m) 200 ft (61 m) maximum	80–150 ft (24–46 m)		Same as shallow-well jet Lower efficiency, especially at greater lifts	The amount of water returned to ejector increases with increased lift—50% of total water pumped at 50-ft (15-m) lift and 75% at 100-ft (30-m) lift			
Rotary									
Shallow-well (gear type)	22 ft (7 m)	22 ft (7 m)	50–250 ft (15–76 m)	Positive action Discharge constant under variable heads	Subject to rapid wear if water contains sand or silt Wear of gears reduces efficiency				
Deep-well (helical rotary type)	Usually submerged	50–500 ft (15–152 m)	100–500 ft (30–152 m)	Efficient operation Same as shallow well rotary Only one moving pump device in well	Same as shallow-well rotary, except no gear wear	A cutless rubber stator increases life of pump Flexible drive coupling has been weak point in pump Best adapted for low capacity and high heads			

\*Practical suction lift at sea level. Reduce lift 1 ft (0.3 m) for each 1000 ft (305 m) above sea level.

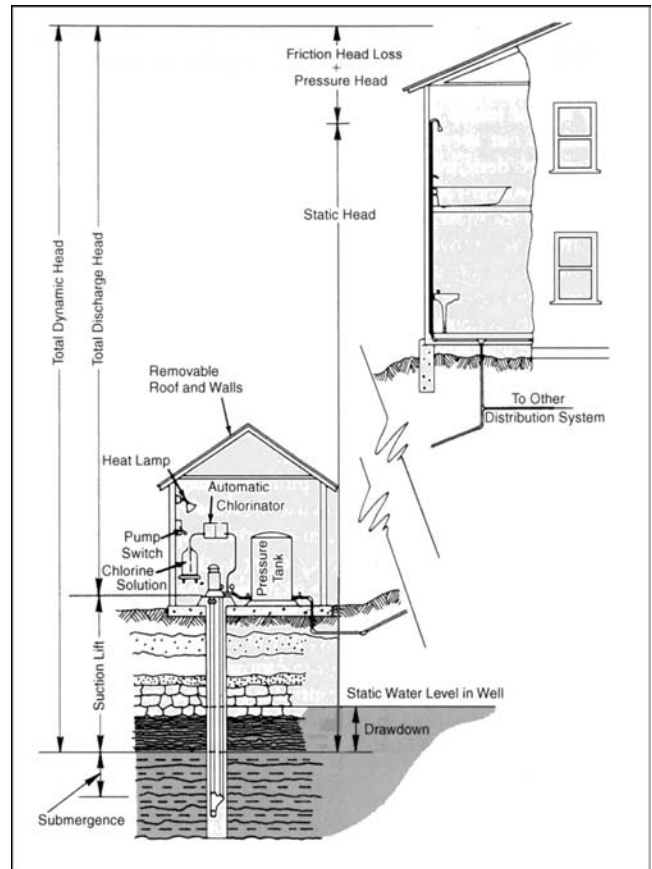


**FIGURE 26.5** System head curve. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, AWWA).

the generator manufacturer can also provide a sound-attenuated shelter to cover the set and to provide space for repairs to be done under protected conditions. These shelters are essentially small, prefabricated buildings made of steel or fiberglass that fit compactly over the set with clearances to allow repairs out of the weather. They are generally less costly than custom-designed buildings but are more costly than a simple generator set enclosure (see Figure 26.10).

Until recently, the normal arrangement for fuel tanks serving generator sets was to use buried tanks. However, as with buried tanks serving gasoline stations, there have been many instances of failure of the tanks and their feed lines causing contamination of the surrounding soils and groundwater. Buried tanks are still a viable option, but they must meet much higher standards concerning leak monitoring and double walled containment for the tank and buried lines (state and local regulatory agencies can provide detailed standards for buried tank installations). This has greatly increased the cost of buried tanks, such that most public utilities are choosing other options.

The other available options for fuel tanks are above-ground tanks that are protected by the buildings in which they are situated or that are otherwise protected. One approach is to use a protected skid-mounted tank that is designed to fit securely under a skid-mounted generator set. These tanks are double-walled for containment and are protected by the generator shelter or enclosure and the skid



**FIGURE 26.6** Components of total dynamic head in well pump. (Reprinted from *Design and Construction of Small Water Systems*, by permission. Copyright © 1999, AWWA).

framing. Their feed lines are inside the generator enclosure and are, therefore, also protected (see Figure 26.11).

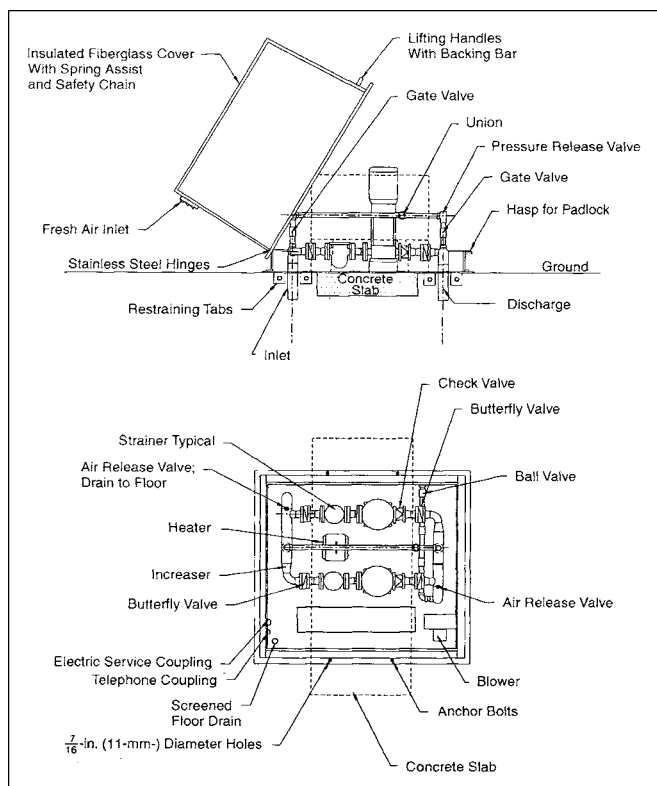
Another option for an above-ground tank is a concrete-enclosed tank such as that provided by Convault (see Figure 26.12). The feed lines from the protected tank must also be protected. If buried, the lines will need to be double walled for containment. Some utilities also require the buried double-walled lines to be installed in a lined trench (see Figure 26.13).

**SYSTEM COMPONENTS**

Essential to a distribution system are such components as valves, fittings, fire hydrants, thrust blocks, service lines and meters to control, direct, and measure the flow. The location and size of these components affects the system's hydraulic efficiency and cost-effectiveness, as well as the health, safety, and welfare of the public.

**Valves**

Approximately 16.7% of all valves in use are for water and sewerage purposes (Merrick, 1991). In a water-works system, valves are used in controlling flow directions, regulating flow rates, pressure control, isolating flows, and transient wave suppression. Typically, faucets, bibs, cocks,



**FIGURE 26.7** Above-ground packaged booster pump station. (Reprinted with permission of Universal Sanitary Equipment Manufacturing Co., Tumah, WI.)

stoppers, and plugs are types of valves used in the water-supply lines within a building, while gate valves, butterfly valves, and check valves are those used, typically, in water distribution lines serving the buildings.

A valve is completely closed when some type of operating mechanism, such as a wheel, forces the gasket or plug tightly against the fixed seat. The operating mechanism (i.e., actuator) for a valve is either manual or automatic. Manual actuators include wheels and levers; automatic actuators can be electric, pneumatic, or hydraulic. Whereas automatic valves are more commonly used at the treatment facilities or pump stations, most valves used in the distribution pipe network are manual.

In addition to their function, valves are also described by their numerous characteristics such as size, material, fitting end, pressure rating, and actuator. The five most common types of valves used in water lines, along with their specific characteristics, provide an extensive list for specifying a particular valve.

The five most common types of valves used in water lines are as follows.

- A *ball valve* (Figure 26.14) has a slotted spherical or section of a spherical closure element that rotates within the casing. The valve is opened when the opening is parallel to the flow direction. A  $\frac{1}{4}$ -turn of the opening closes the valve. Typically, ball valves are used for on-off service and throttling the flow.

■ A *plug valve* (Figure 26.15) is similar to the ball valve in that a cone or cylinder attached to a shaft, with a rectangular slot or circular orifice, opens and closes the flow by rotating the slot parallel to the direction of flow. Eccentric plug valves use a seat that consists of only part of the cone or cylinder to close the valve. The shaft rotates the seat over the pipe opening to regulate the rate of flow. Typically, plug valves are used for control and isolation purposes. Smaller plug valves are used on service connections and are referred to as service or corporation cocks. Plug valves have low head losses and typically cost more than gate, globe, and butterfly valves.

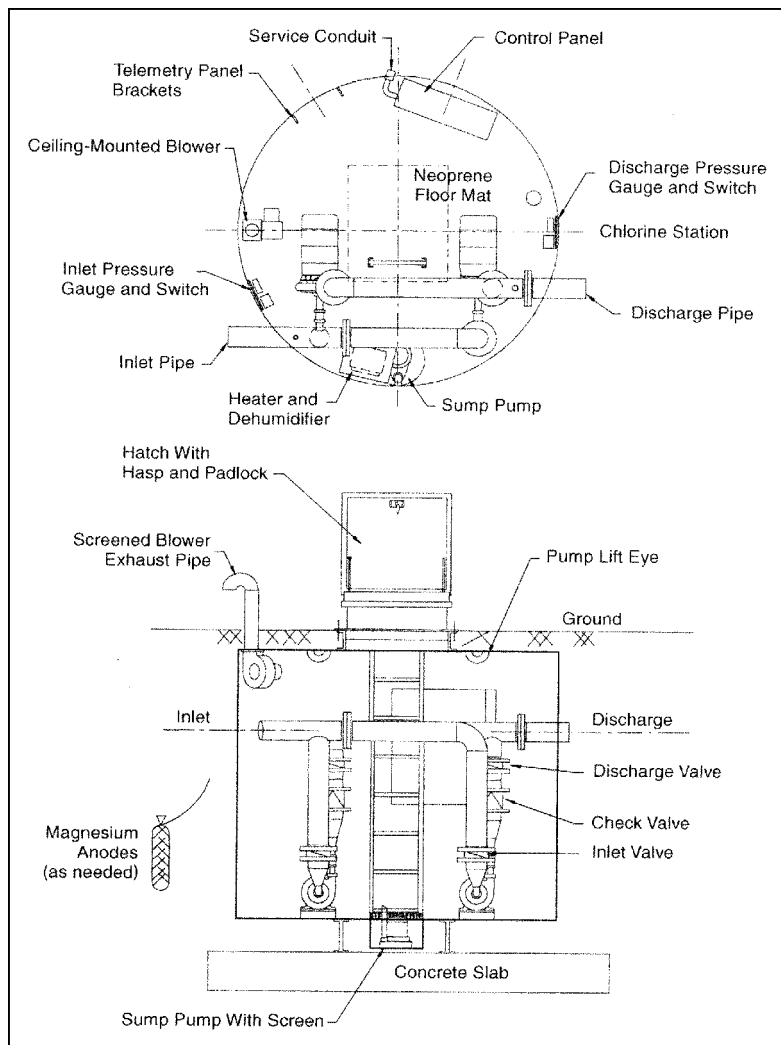
■ *Gate valves*, a type of the broader classification of slide valves, are the most frequently used when trying to isolate flows in the pipe network (see Figure 26.16). The gate valve (shutoff valve) is used to completely stop the flow through the pipeline. It operates by raising or lowering a plate or disc into the flow path. Note: a shut-off valve's intended use is to operate in the full-open or full-closed positions only, and not as a throttling valve—that is, partially opened. Considerable wear and tear on the mechanism occurs in the partially-opened position, and the head loss, at this position, is very high. Typically, gate valves are placed in valve boxes that extend to the ground surface to permit access. The valve is operated with an extension wrench that reaches down through the box to turn the operating nut on top of the valve stem.

■ As shown in Figure 26.17, a *butterfly valve*, also a type of rotary valve, consists of a thin disk that rotates about a thin shaft. When the face of the disk is parallel to the flow directions the valve is fully opened. Butterfly valves can be used for either shut-off or throttling purposes. Although these types of valves have relatively low head loss, they present problems when cleaning, since the disk is in the flow stream.

■ The *globe valve*, which is another type of slide valve, consists of a flexible disk attached to a screw-operated stem. A schematic diagram of a globe valve is shown in Figure 26.18. Raising and lowering the disc onto a horizontal seat controls the flow. Use of globe valves is most common in smaller domestic water lines; a familiar globe valve is the household faucet. High-pressure losses are typical of most globe valves.

A summary of the important characteristics for these particular valves appears in Table 26.9. In addition to the basic valves just described, there are other specialty valves used in waterworks systems:

- *Check valves* are pressure-activated valves. Typical water-main systems are bi-directional in that water flows according to the pressure differential in the pipes. Check valves are inserted to restrict flows to one direction. For example, a check valve on the discharge



**FIGURE 26.8** Below-grade packaged booster pump station. (Reprinted with permission of Universal Sanitary Equipment Manufacturing Co., Tumah, WI.)

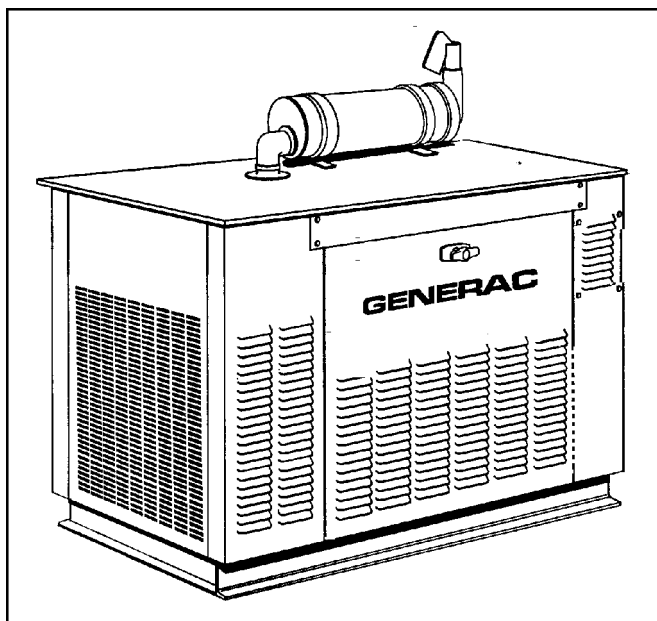
end of a pump or treatment plant prevents the water from reversing directions and flooding out the facility when the facility shuts down operation. Check valves are also used to control flows at the boundary of different water-supply districts and service areas of different pressures.

- *Pressure reducing valves* (PRV) are automatic valves that protect against excess pressure in the water lines. Pressure-reducing valves act by controlling the flow and intentionally causing high head losses. Large pressure-reducing valves are operated by a smaller pilot valve. A wheel on the pilot valve sets the internal spring tension, which in turn sets the activating pressure. Larger PRVs are placed on-line between service areas of different elevations. Smaller PRV (<2 in.) valves are used to protect residential plumbing from excessive pressures in the main line. The valve acts as the transition between high-pressure zones and low-pressure zones. For example, if line pressures are

greater than 70 psi, a pressure-regulating valve installed on a domestic service line reduces the service line pressure so that the pressure will not damage household fixtures.

- *Altitude valves* are automatically activated according to the pressure differential on either side of the seat. This type of valve is usually used to control the flow to and from an elevated storage tank. The valve opens, allowing the tank to refill, when the pressure in the distribution line is greater than the static pressure of the water in the tank. In times of high demand, when the pressure in the main decreases, the valve opens to allow the water in the storage tank to equalize the distribution system.

- *Blow-off valves* and *drain valves* are used to de-water lines for repairs or remove accumulated sediment. These valves are most commonly used at the end of branches and at low points of the water main (see Figure 26.19).

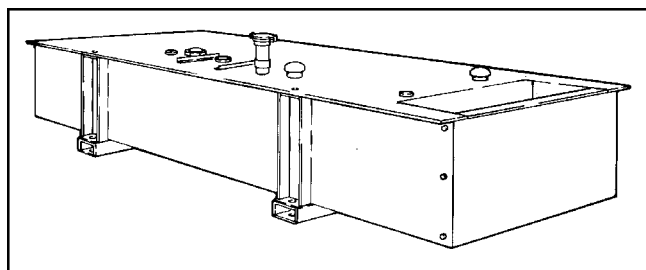


**FIGURE 26.9** Typical generator enclosure. (Reprinted with permission of General Power Systems, Waukesha, WI.)

■ *Air-release valves.* Water at standard conditions contains about 2% to 3% dissolved air by volume. The amount of dissolved air in a water-line system is governed by the turbulence before it enters the system and the pressure and temperature within the system. Pressurized distribution systems change elevations as the pipe follows the terrain. In effect, the changes in elevation change the pressure in the system. Consequently, the air dissociates from the water and collects at the higher points in the pipeline. Since the return of air back into the water does not occur as readily as it leaves the water, air pockets, which add 10% to 15% more resistance to the flow, are formed. Additionally, these pockets can lead to air lock of the system, which can completely stop the flow. Typically, air-relief valves are installed at the local peaks in the pipe system. As shown in Figure 26.20, a peak is any section of pipe that slopes up towards the hydraulic grade line or runs



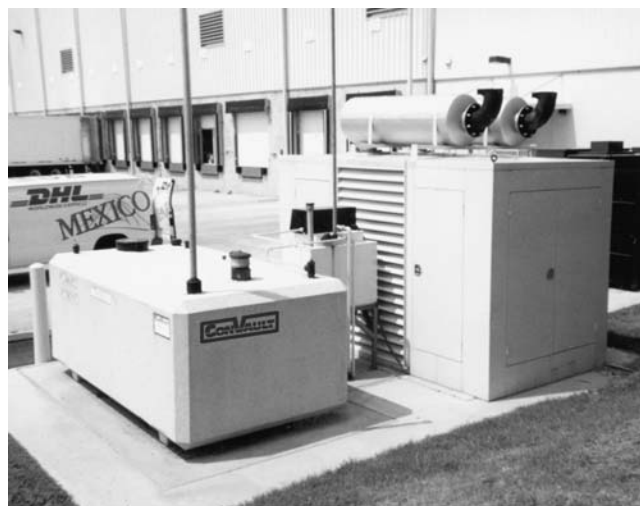
**FIGURE 26.10** Sound attenuated walk-in enclosure. Copyright © Tramont Corporation 2001.



**FIGURE 26.11** Skid-mounted fuel tank for location beneath a generator set. (Reprinted with permission of Cummins Power Generation.)

parallel to it. Air-release valves should also be considered in sections where there is an increase in the downward slope or decrease in the upward slope<sup>1</sup> of the pipe.

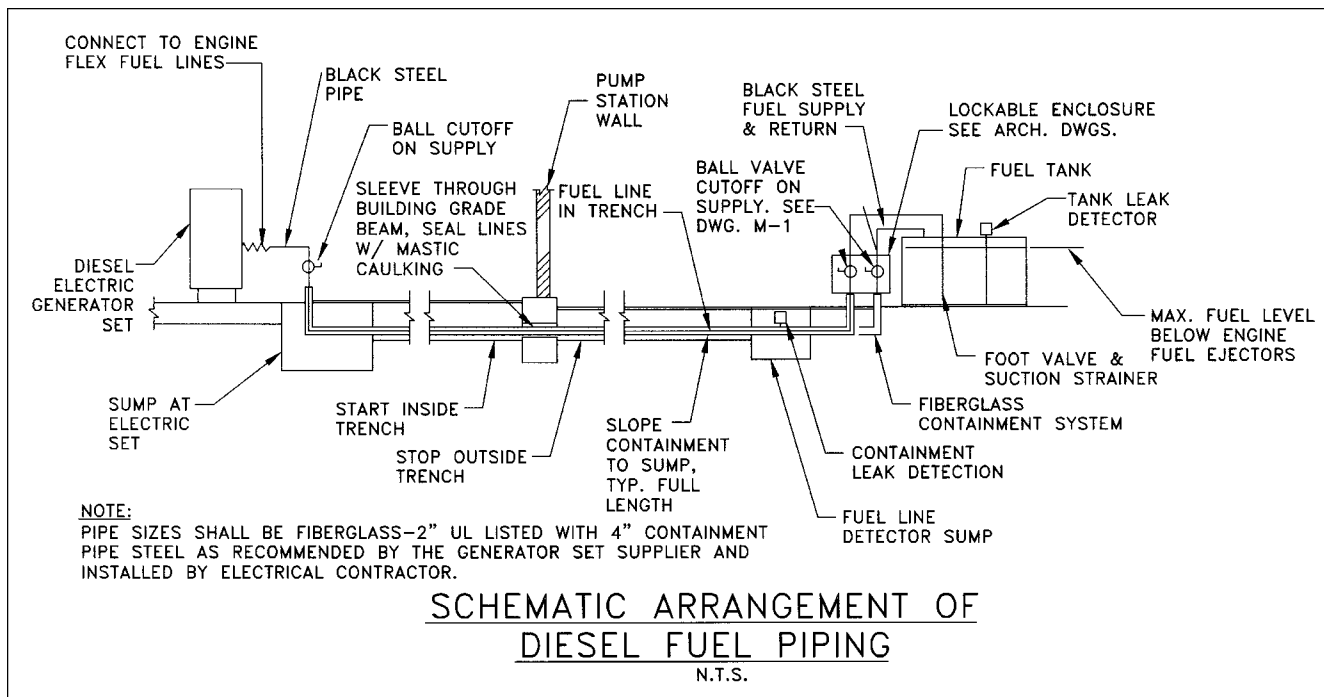
■ *Hydraulic surge (water hammer)* can occur when water flowing in a long pipeline suddenly stops. Surge can actually cause a water column separation at high points, or peaks, on the main. This, at first, exerts a tremendous negative pressure (or vacuum) on the main that can cause it to collapse. If the system does not collapse, the vacuum will be followed immediately by a pressure spike, which occurs as the separated water columns reunite, that also can rupture the pipe. To prevent this, vacuum-relief, or combination air release and vacuum-relief valves, are used at points on water mains where analysis indicates water column separation might occur. The vacuum-relief valve lets air into the pipeline during hydraulic surge to both relieve the vacuum and provide a volume of air that cushions the



**FIGURE 26.12** Aboveground storage tank and generator set. (Reprinted with permission of Core Engineered Solutions.)

<sup>1</sup> Golden Anderson Automatic Valve Reference Manual, Golden Anderson Industries, Inc. Pa.





**FIGURE 26.13** Schematic arrangement of diesel fuel piping.

pressure spike caused by the two water columns coming together again. Note: hydraulic surge analysis is a specialized field and must be done by qualified personnel.

### Pipes, Joints and Fittings

Typically, pipes used in waterworks systems are classified according to their design working pressures. The four most common classes of working pressures are 100, 150, 200, and 250 psi. Other parameters for identifying pipe strength are the bursting strength rating, which identifies the strength against internal forces caused by hydrostatic and hydrodynamic pressures, such as pressure surges and water hammer, and the crushing strength, which deals with the external forces related to soil, vehicles, and impact loadings.

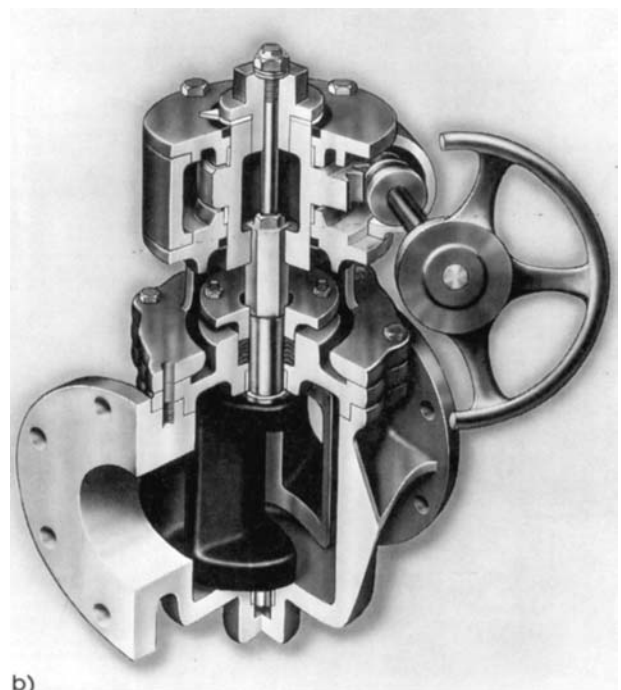
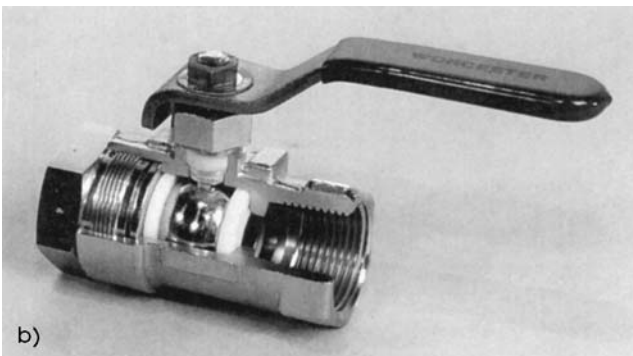
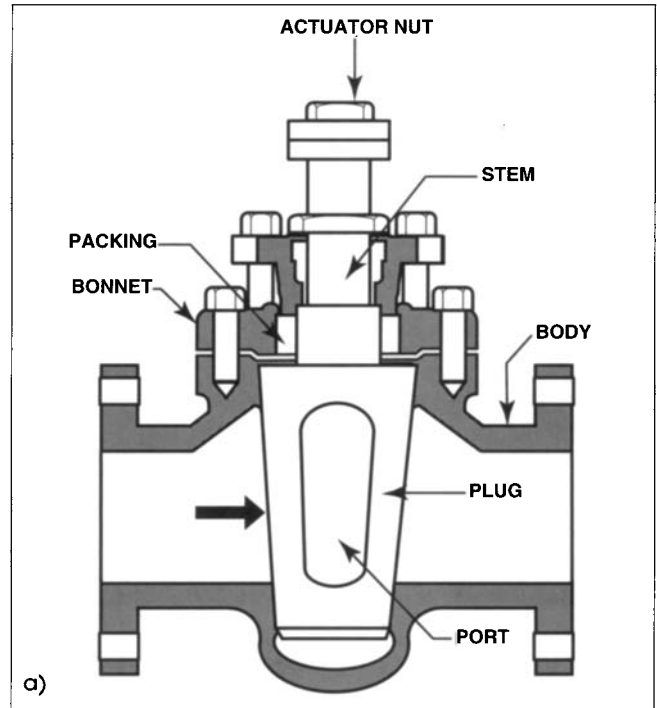
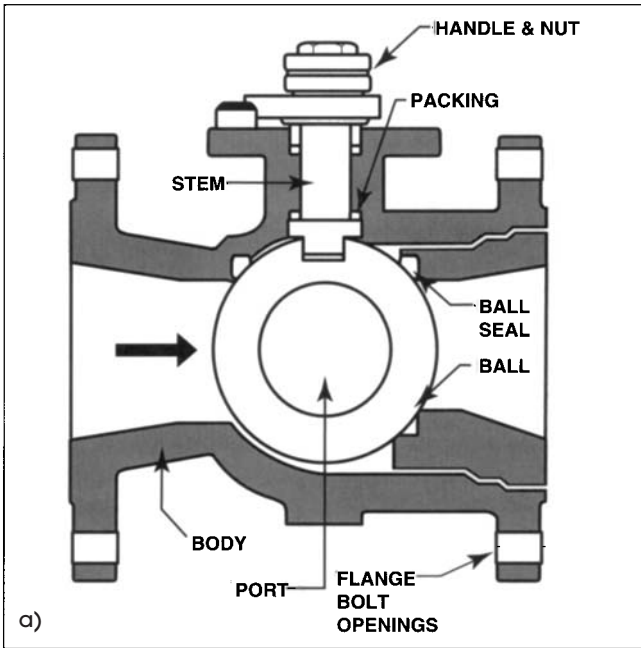
Typical standard pipe diameters used in distribution systems range from 6 in. through 20 in. in 2-in. increments. Then, beginning with 24 in., diameters increase in 6-in. increments. For residential plumbing pipe, diameters begin at 1/2 in. and increase in 1/4-in. increments. Since manufacturers and pipe material dictate the available pipe diameters, diameters other than those mentioned here might be available. The supplier should be contacted to verify what pipe diameters are available and if they are in stock.

Although ductile iron pipe is the most popular cast iron pipe, other common pipe materials for water-distribution lines include grey cast iron, steel, plastic and polyethylene. Note that this same type of piping is used for water-service lines larger than 2 in. Copper or plastic tubing is commonly used for service lines smaller than 2 in.

**Plastic Pipe.** Distribution lines for small water systems are normally constructed of PVC pipe because of its ease of construction and economy. Plastic pipe used for drinking applications should be certified by an acceptable testing laboratory (such as the National Sanitation Foundation) as not producing objectionable taste and as being non-toxic. Standards for PVC pipe (up to 12-in.) for underground service are ASTM D1785, ASTM D2241, and ANSI/AWWA C900. ASTM D1785 pipe is available in Schedule 40, 80, and 120 thickness. ASTM D2241 pipe has pressure ratings up to 250 psi, while AWWA C900 pipe is available in pressure ratings up to 200 psi. AWWA C900 pipe is manufactured to ductile-iron pipe outside dimensions, and both ASTM D2241 and ASTM D1785 are manufactured to steel pipe outside dimensions. Joining these pipes requires a transition gasket used with standard ductile-iron fittings.

**Ductile-Iron Pipe (DIP).** DIP is standardized in ANSI/AWWA C150/A21.50-96 and is available in sizes from 3- to 64 in. in various pressure classes. Push-on or mechanical joints are used on DIP for underground service. For water service, DIP is normally lined with cement-mortar. Soft water will leach the cement-mortar lining if not properly conditioned. Depending on soil conditions, DIP is often installed in polyethylene wrapping to aid in corrosion protection.

**Steel Pipe.** Steel pipe is normally used for large-diameter installations beyond the sizes of which DIP is available and is not often used for small water systems. However, it is available in various sizes and thickness classes. It is covered under the AWWA C200 standard. Note: buried steel pipe



**FIGURE 26.14** Ball Valve. Reprinted with permission of Worcester Controls, Marlboro, Mass.

**FIGURE 26.15** (a) Schematic diagram of a plug valve; (b) eccentric plug valve. Reprinted with permission of DeZurik, Sartell, Minn.

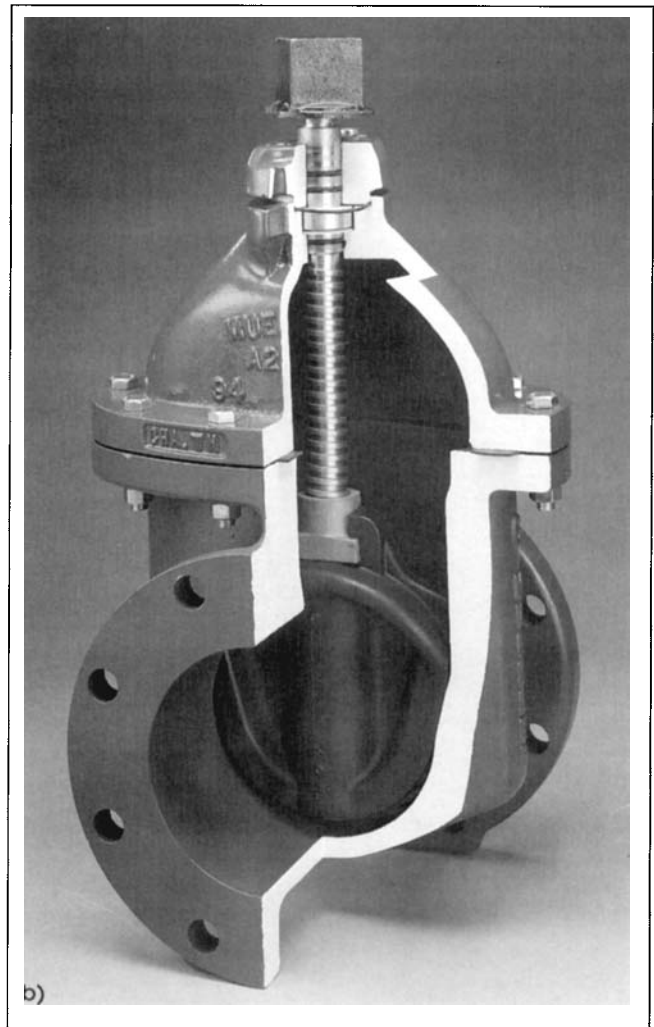
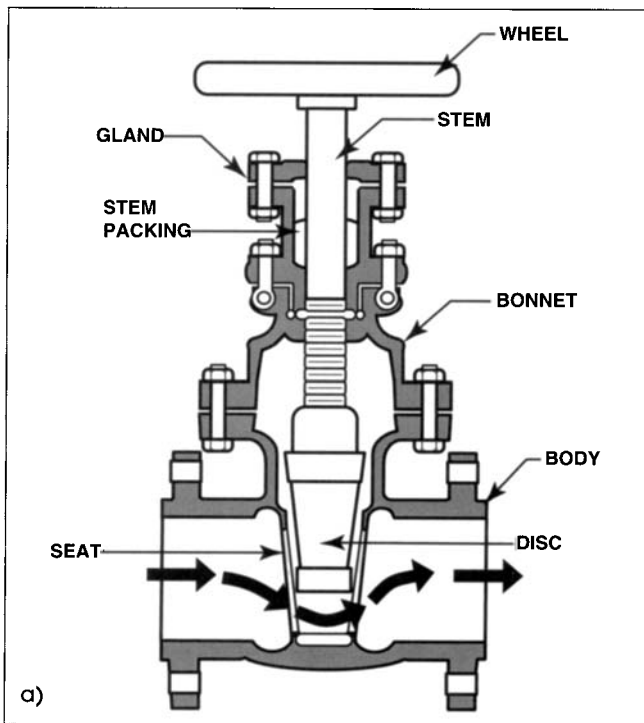
is more susceptible to corrosion than DIP and must be adequately protected. It is usually installed underground with a tape-wrap coating as a minimum, and the pipe interior is normally specified to be provided with a cement-mortar lining for drinking water service.

**Polyethylene Pipe (PE).** PE is becoming popular for small water service, mainly because it is economical to purchase and install. PE is available in sizes larger than 4 in., and the standard for PE is AWWA C906.

**Joints.** Distribution pipes are connected by either of the several different types of joints shown in Figure 26.21.

- The *push-on joint* is a compression joint that creates a pressure-tight seal when a gasket, inside the bell end, is compressed by the plain end of the pipe. This type of joint allows for fast and easy installation and is low in cost, which probably accounts for its popularity. However, this type of joint does not restrain the indi-

vidual lengths of piping from separating under pressure, so concrete thrust blocks are cast at elbows and other turns to restrain the pipe. Soil friction on the pipe also acts to prevent joint separation. The push-on joint is not suitable for above-ground use in pressure piping.



**FIGURE 26.16** Gate valve: (a) schematic diagram; (b) resilient seat gate valve. Reprinted with permission of Mueller Co., Decatur, Illinois.

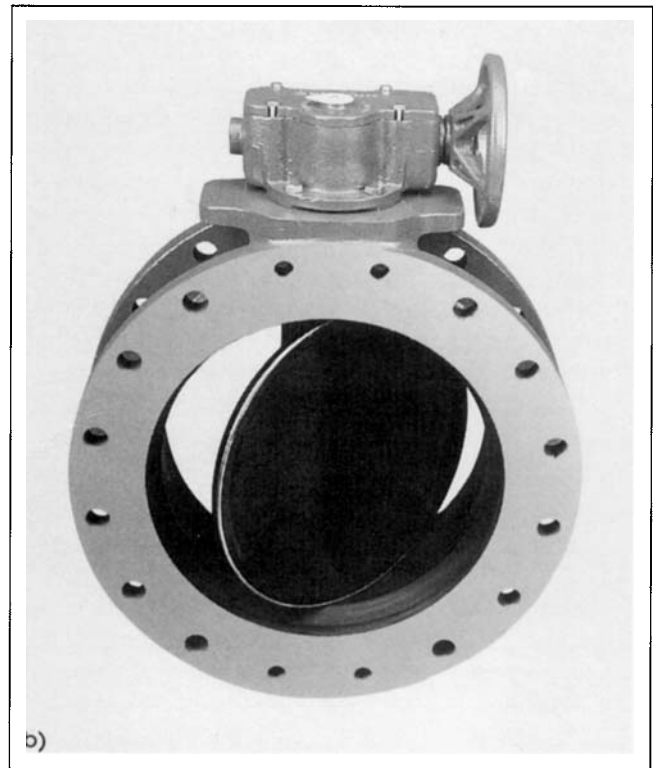
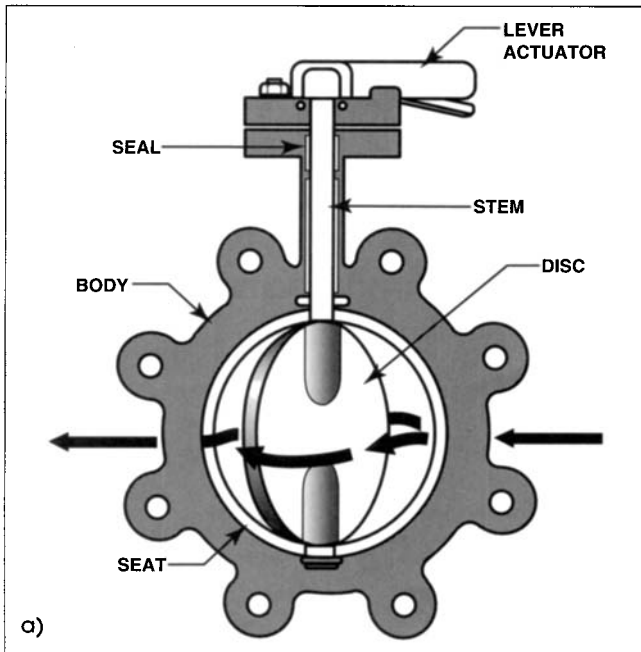
- For the *mechanical joint*, the pipes fit together as a plain end inside a bell-shaped end. A pressure-tight seal is created by a gasket tightened between the flange on the bell end and the gland, a flange-like collar that slips over the plain end. The flange and gland are held together with bolts. The mechanical joint is considered a “semi-restrained” joint in that friction, in the highly-compressed gasket, provides some restraint against pipe separation under pressure. Where joint restraint is not provided by thrust blocking, as for the push-on joint, a restraining collar with locking bolts is used to hold the joint together. The mechanical joint is easy to assemble and is intermediate in cost (i.e., between the push-on joint and the more costly ball joint, flanged joint or threaded joint).

- The *ball joint* is used for crossing waterways, as a restrained joint, and where free-turning deflections are needed. The gasket is inserted into the bell and compressed by the entering ball. The joint is secured by locking the bayonet-type retainer over the lugs on the

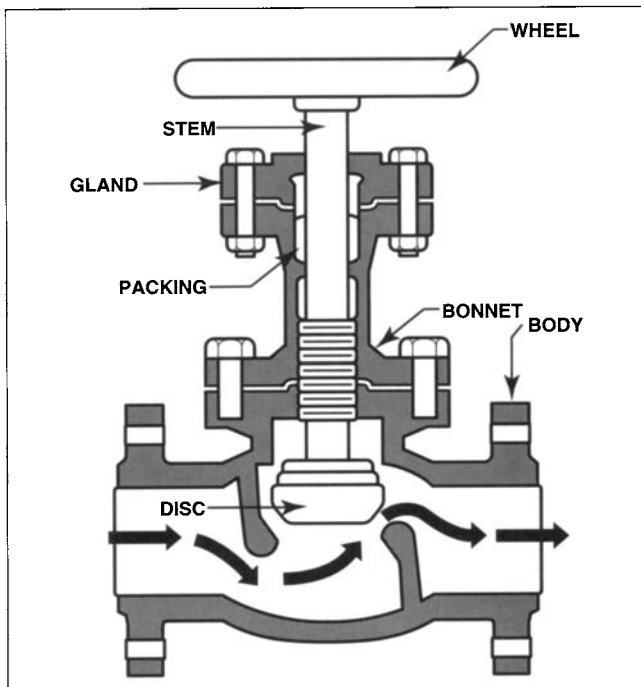
bell. A retainer lock inserted between the lugs prevents the retainer from rotating.

- The *flanged joint* is used for the larger sizes of above-ground distribution piping, such as in pump stations. This joint is seldomly used for underground distribution piping because of its cost and difficulty of installation. The flanged joint has a flat-faced or flat-faced with gasket-groove bolt collar or flange on each end of the pipe. A suitable gasket is placed between the flange faces and compressed by the bolts to form a pressure-tight joint. The flange joint is the strongest of the joints. The chief disadvantage of the flange joint is that usually some type of length-compensating joint must be included in the pipe run, because as the flange bolts are drawn up, the overall length of the pipe run is shortened by the compression of the gaskets.

- The *threaded or screwed joint* is formed by cutting threads into the end of the pipe on a tapered diameter



**FIGURE 26.17** (a) Butterfly valve; (b) AWWA butterfly valve. Reprinted with permission of DeZurik, Sartell, Minn.



**FIGURE 26.18** Schematic diagram of a globe valve.

(i.e., “pipe threads”). When the tapered threads are screwed together, they create an interference fit, which with the aid of a mastic pipe sealing compound on the threads makes a pressure tight joint. The threaded joint is not commonly used for distribution piping because it is difficult to make in larger pipe sizes and because the loss of metal in the pipe wall at the threads makes it weaker than the flanged joint.

- The ball, flanged, and threaded joints are considered “restrained” joints in that the joints will hold the individual lengths of pipe together under system-operating pressure; as such, these joints are suitable for above-ground use.

**Fittings.** Fittings are used to connect same or different-size pipes, change the flow direction, split the flow into several directions and stop the flow. Examples of fittings commonly used in the distribution network are crosses, tees, wyes, and bends. Standard bends are 90° or full bends, 45° or 1/2° bends, 22½° or 1/4° bends and 11¼° or 1/8° bends. Fittings are readily available in standard sizes. Non-standard sizes are special-order items from a manufacturer. Because the special-order items cost significantly more and may delay construction while on order, their use and necessity in design should be avoided except in the most extreme cases. Typically, the AWWA, ANSI, and ASTM designate acceptable standards and specifications for various pipes and fixtures for water-supply systems. Examples of various fittings are shown in Figure 26.22.

**TABLE 26.9 Valve Summary (Water Distribution Operator Training HB, AWWA, 1976)**

<b>VALVE TYPE</b>	<b>COMMON SIZE RANGE (in.)</b>	<b>PARTICULARLY ADAPTED TO</b>	<b>MAIN ADVANTAGES</b>	<b>MAIN DISADVANTAGES</b>
Gate	1/2–24	Isolation services in distribution grids	Low cost in small sizes Low friction Good service life Ease of installation	High cost in large sizes Large sizes are quite heavy Poor for throttling; should not be used where frequent operation is necessary
Butterfly	3 & up	Isolation and automatic control	Low cost in larger sizes for normal service pressures Some types have very short lengths Ease of operation	Higher friction loss than gate valve Difficult to open in lines where differential pressures exists May cause problems when relining pipe Leaks because of seat damage
Globe	1/2–24	Isolation in smaller sizes Flow control in larger sizes Pressure control	Simple construction Dependable, can be used for throttling Good for pressure control Sediment or material unlikely to prevent complete closing	High friction loss Very heavy and expensive in large sizes
Ball and Plug	1/2–3 1/2–12	Isolation and throttling	Dependable Very low friction loss Slow shut-off characteristic minimizes closing surges Ease of operation Resistant to erosion Long life	High cost in large sizes Very heavy

Reprinted from "American Water Distribution Operator Training Handbook," by permission. Copyright © 1976, American Waterworks Association.

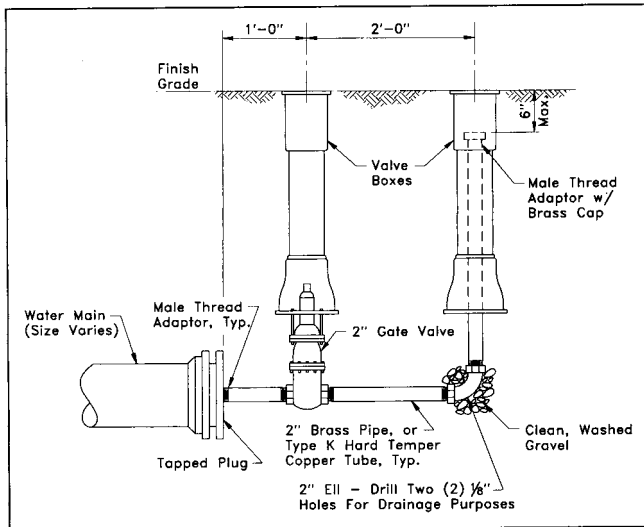


FIGURE 26.19 Two-inch blow-off valve detail.

**Fire Hydrants**

A fire hydrant consists of the above-ground barrel that extends below grade to the water supply line. The supply line connecting the barrel to the water main is a minimum six-in diameter pipe. Typically, a gate valve located in an underground valve box allows for shut-off and repair. Maximum recommended friction losses in the hydrant are 2.5 psi, and five-psi in the water supply line, from the main to the hydrant, for flows of 600 gpm. Hydrants have various nozzle arrangements, but the most common hydrant has two 2.5-in. hose connections and one 4.5-in. pumper connection. The bend directly below the barrel connects the barrel to the supply line. A thrust block prevents movement from lateral forces and a concrete block underneath the bend prevents excessive settling. After use of the hydrant, the barrel is drained by way of a drain near the base of the barrel. A gravel bed surrounds the thrust block or bend to allow for draining of the barrel.

**Meters**

Meters are commonly used to measure water for setting cost, collecting payments from customers, and determining fair distribution of water delivery costs. Other uses for meters include flow measurements to and from treatment plants and reservoirs, blending of water and chemicals, and measuring water sold to other jurisdictions.

Meters used to measure usage for residential dwellings and commercial use are located in areas easily accessible to the water utility company. They should not interfere with public safety nor act as a hazardous obstacle. Most meters are located below ground surface levels for safety and to reduce damage due to weather and tampering. Occasionally meters will be located inside the building with a remote recorder mounted on the outside.

Meters are available in the same sizes as pipe diameters. Usually the meter is sized one diameter size less than the pipe for reasons associated with accuracy, head loss, and cost. Unlike pipe, where the cost differential between successive pipe sizes may be on the order of 10% to 25%, the cost differential between successive meter sizes can be significantly more. Downsizing the meter also downsizes succeeding appurtenances and fixtures thereby reducing costs. The increase in accuracy in using the smaller size meter is counterbalanced by an increase in the head loss. Usually the cost savings and the accuracy offset the drawback incurred from the increased head loss.

**Thrust Restraint**

Two basic forces associated with flowing water under pressure are the hydrostatic forces and hydrodynamic forces. Simply explained, hydrostatic forces are due to the pressure exerted by a depth of fluid as shown in Figure 26.23. This pressure force acts perpendicular to a surface. Thus, for a pipe with full flow, the pressure acts radially outward and also along the longitudinal axis of the pipe.

Consider a pipe connected to a tank and plugged at the other end as shown in Figure 26.23. From basic hydrostatics, the pressure on the plugged end of the pipe is proportional to the depth of water,  $H$ , as shown by the hydrostatic pressure distribution diagram. The pressure,  $p$ , is equal to  $\gamma H$ , where  $\gamma$  is the specific weight of the fluid. The magnitude of the force acting at the end of the pipe is equal to the product of the pressure and the cross-sectional area of the pipe,  $F_p = \gamma H A$ , where  $A$  is the cross-sectional area of the pipe.

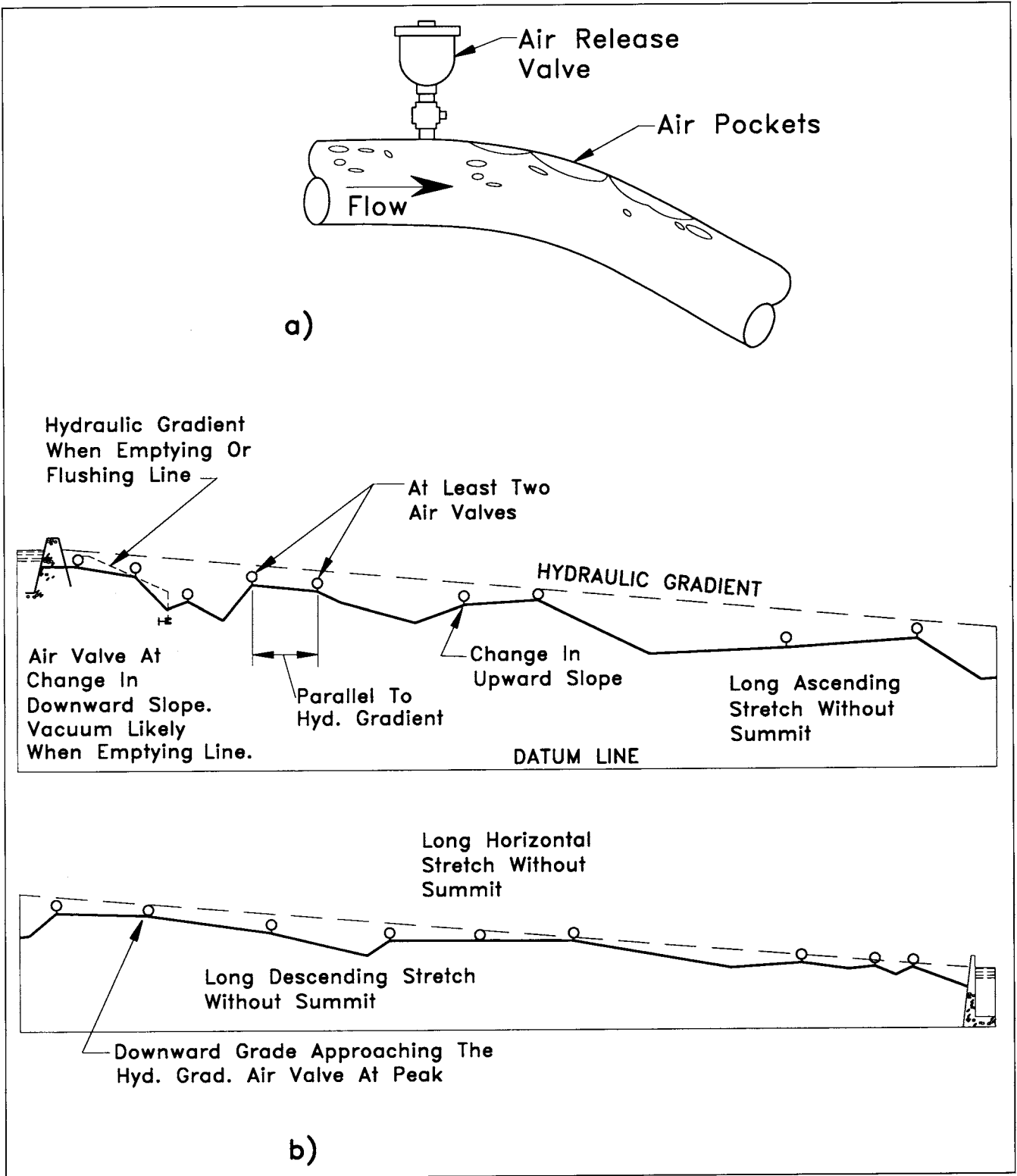
Hydrodynamic forces are the result of changes in momentum of the moving fluid. Any change in direction or magnitude of flow velocity results in a change in momentum of fluid. Fittings such as bends, tees, and wyes change the direction of flow; nozzles, valves, and reducers change the cross-sectional area of the flow path. Each of these fixtures has force acting on them due to hydrostatics and hydrodynamics.

The impulse-momentum equation (assuming incompressible flow) determines the resultant hydrodynamic force of the water on the fitting. The impulse-momentum equation, in vector component form, is

$$\begin{aligned} \sum F_x &= \rho Q(\Delta V_x) \\ \sum F_y &= \rho Q(\Delta V_y) \end{aligned} \tag{26.5}$$

where  $Q$  is the discharge,  $\rho$  is the density, and  $V_x$  and  $V_y$  represent the  $x$  and  $y$  components of velocity.

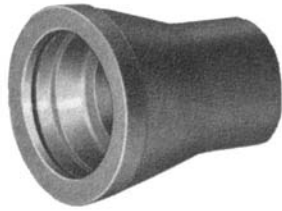
Consider the horizontal bend of Figure 26.24-a. The force of the water on the bend is the result of the pressure forces,  $pA$ , acting on the water within the bend, as illustrated in Figure 26.24-b. To counter these forces, an equal and opposite force is produced by the longitudinal stresses in the pipe wall (Figure 26.24). Since common pipe joints



**FIGURE 26.20** (a) Typical bubble behavior; (b) suggested air release valve locations. Reprinted with permission of GA Industries, Inc., Cranberry Township, PA.

### SUPER BELL-TITE PUSH-ON JOINT

The Super Bell-Tite Push-On joint is a rubber ring gasket, compression type joint whose only accessory is a molded rubber gasket. Fitted into the gasket recess of the bell, the gasket forms a pressure-tight seal with ample deflection when compressed in position by the entering plain end of pipe.



*Super Bell-Tite Push-On Joint*

### MECHANICAL JOINT

Mechanical joint is used extensively for underground water, sewage, and gas lines and for conveying liquids and semi-solids, both outside and inside buildings. The joint consists of an integral bell with flange, and is made up with a rubber ring gasket, follower glands, nuts and bolts. When assembled the joint provides for normal expansion, and contraction and permits adequate deflection in all diameters



*Mechanical Joint*

### THREADED I.P.S. JOINT

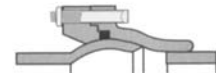
Cast iron I.P.S. (Iron Pipe Size) is cast to the same outside diameter as steel pipe, and is used for plumbing and drainage services, such as downspouts, waste and vent lines. The pipe is used with regular screwed, drainage and soil fittings, and can be cut, threaded and fitted on the job with the ordinary tools of the piping trade.



*Threaded I.P.S. Joint*

### BALL JOINT

Ball joint iron pipe is designed especially for submarine and submerged piping and for pipe lines installed across hilly terrain. Easy joint assembly and long lengths make this pipe ideal for such purposes. The joint provides a maximum 15° free turning deflection with no reduction in the internal opening area at any deflection.



*Ball Joint*

### FLANGED JOINT

Flanged joint cast iron pipe is ideally used for exposed piping inside and outside of buildings for conveying water, sewage, sludge and other fluids corrosive to steel but not to cast iron. It also is used for low pressure air, steam, oil, gas and other services.



*Flanged Joint*

**FIGURE 26.21** Joint connections for cast iron pipe. Reprinted with permission of McWane Cast Iron Pipe Company, Birmingham, Alabama.



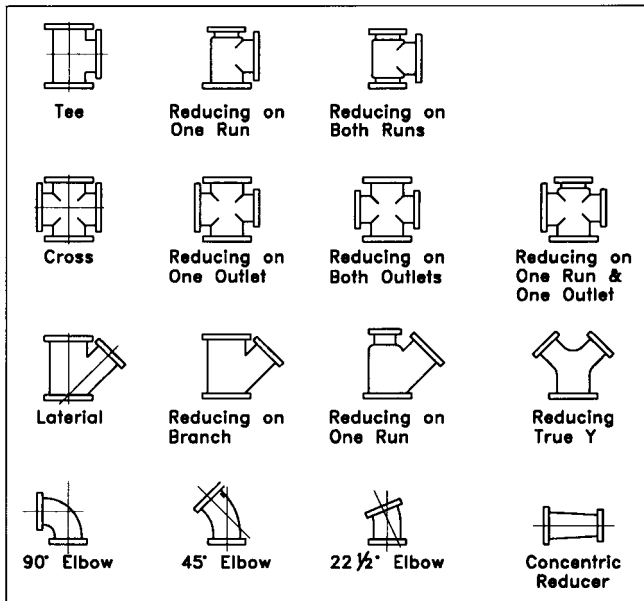


FIGURE 26.22 Various types of fittings.

cannot resist these longitudinal forces, the developed forces must be reduced or eliminated by additional anchors, either in the form of thrust blocks or with supplemental clamps and collars at the joints.

As an example of the magnitude of such forces, consider a typical water distribution pipe system with operating pressure ranges from 40 to 150 psi and velocities approximately 3 to 10 fps. Within these ranges, hydrostatic forces are far greater than hydrodynamic forces and for the purposes of design can, in most cases, be ignored. To illustrate this point, consider a 12-in.-diameter, 45° horizontal bend. Assume line pressure of 50 psi and flow velocity of 5 fps. The right side of the momentum equation (Equation 26.5) is the force due to the change in momentum of the fluid. The left side represents all forces acting on the fluid, such as gravity, pressure forces, and shear forces. For the conditions

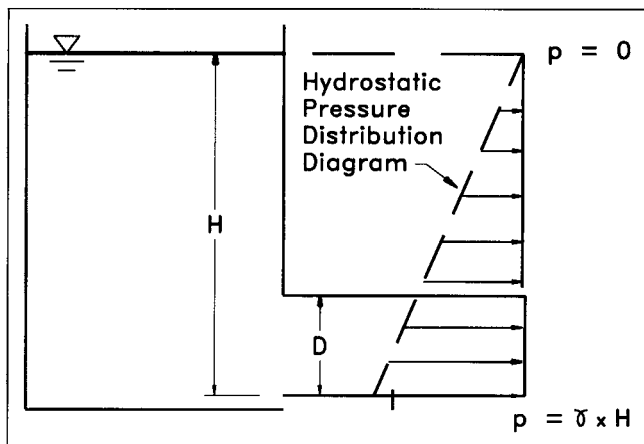


FIGURE 26.23 Hydrostatic pressure in a pipe.

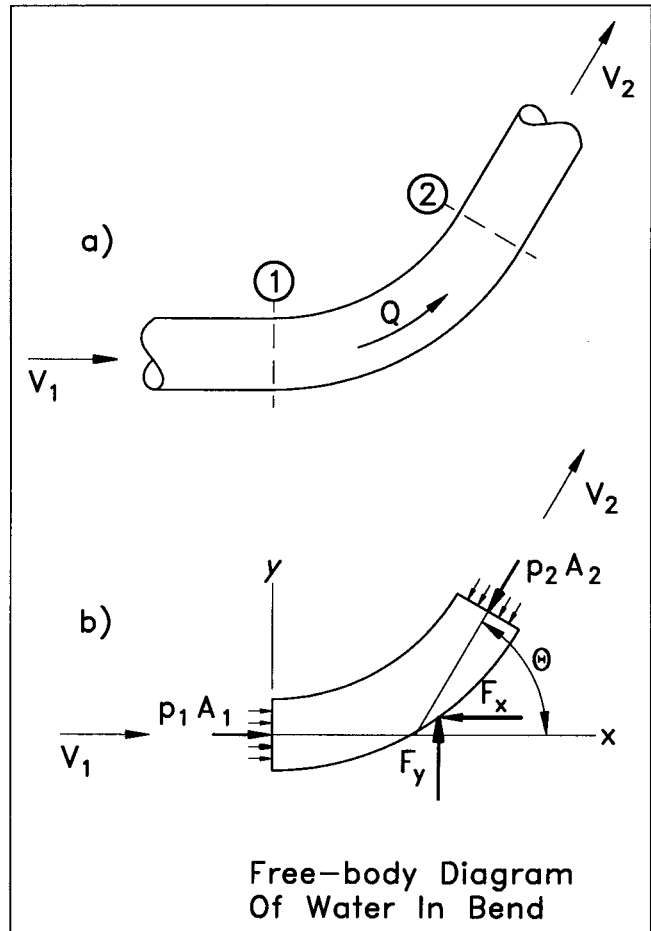


FIGURE 26.24 Forces for flow through a horizontal bend.

given, let  $F_x$  and  $F_y$  represent the forces of the bend on the fluid. Expanding Equation 26.5 gives

$$\begin{aligned} \rho_1 A_1 - F_x - \rho_2 A_2 \cos 45^\circ & \quad (26.6) \\ &= \rho Q (V_2 \cos 45^\circ - V_1) \\ & F_y - \rho_2 A_2 \sin 45^\circ \\ &= \rho Q V_2 \sin 45^\circ \end{aligned}$$

Rearranging terms and substituting the appropriate values

$$\begin{aligned} & \left( 50 \text{ psi} * 144 \frac{\text{in.}^2}{\text{psi}} \right) (0.79 \text{ ft}^2) & (26.7) \\ & - \left( 50 \text{ psi} * 144 \frac{\text{in.}^2}{\text{psi}} \right) (0.79 \text{ ft}^2) \cos 45^\circ \\ & - 1.94 \text{ slugs} * 3.9 \frac{\text{ft}^3}{\text{sec}} \\ & \left( 5 \frac{\text{ft}}{\text{sec}} \cos 45^\circ - 5 \frac{\text{ft}}{\text{sec}} \right) = F_x \end{aligned}$$

or

$$1666 \text{ lbs} - (-11 \text{ lbs}) = 1677 \text{ lbs} = F_x \quad (26.8)$$

Similarly the y-component of the force is

$$\left( 50 \frac{\text{lbs}}{\text{in}^2} * 144 \frac{\text{in}^2}{\text{ft}^2} * 0.79 \text{ ft}^2 \sin 45^\circ \right) + (19.4 \text{ slugs}) * \left( 3.9 \frac{\text{ft}^3}{\text{sec}} \sin 45^\circ \right) = F_y \quad (26.9)$$

or

$$5688 \text{ lbs} + 5.4 \text{ lbs} = 5693 \text{ lbs} = F_y \quad (26.10)$$

The preceding computations show that for conditions typical of water distribution systems, the components of the momentum force (i.e., 11 lb and 5.4 lb) are substantially less than the x and y components of the pressure forces, namely 1666 lbs and 5688 lbs, respectively.

The types of joints commonly used to connect ductile iron pipe are push-on or mechanical joints. Although leak-proof and quite effective for normal radial stresses, these joints are not capable of resisting the hydrostatic and hydrodynamic forces at the components in the longitudinal direction. Added measures are necessary to reinforce longitudinal separation at the joints. For instance, thrust blocks are used to transfer the fluid pressure forces to undisturbed soil. Other measures include collars and clamps around the joints to provide additional restraint.

The design concept of thrust blocks is similar to foundation design. The intent is to transfer internal hydrostatic forces to the undisturbed soil mass through a mass of concrete. For a given hydraulic design, the size of the thrust block depends on the bearing capacity of the soil and the hydrostatic forces within the waterline component. Figure 26.25 shows the magnitude and line of action of hydrostatic forces for various types of waterline components. The location of the thrust block depends on the type of pipe fittings. Parameters considered in the design of thrust blocks include pipe size, hydrostatic pressures, type of component, and soil properties.

The Ductile Iron Pipe Research Association (DIRPA) recommends the following for thrust block design (with reference to Figure 26.26).

- Bearing surface of the block should be placed against undisturbed soil. Alternatively, when this is not possible, the fill soil between the block and the undisturbed earth must be compacted to minimum 90% Standard Proctor density. The required bearing block area is

$$A_b = hb = \frac{TS_r}{S_B} \quad (26.11)$$

where:

$A_b$  = the bearing surface area of the block

$h$  = height of the block

$b$  = width of the block

$T$  = the resultant hydrostatic thrust force on the component. The hydrostatic forces (PA terms) corresponding to  $T$  are found from Figure 26.25.

$S_f$  = factor of safety (usually 1.5 for thrust block design)

$S_B$  = bearing strength of the soil (psf)

- The block height ( $h$ ) should be equal to or less than  $\frac{1}{2}$  the total depth to the bottom of the block,  $H_T$ , but not less than the pipe diameter,  $D$ .

- Block height ( $h$ ) should be chosen such that the calculated block width ( $b$ ) varies between one and two times the height.

Table 26.10 provides bearing strengths for generalized soil, to be used whenever actual bearing strengths are not available. The engineer is responsible for proper selection of the bearing strength based on proper soil identification. The DIPRA suggests that in lieu of the values for soil bearing strength shown in Table 26.10, a designer may choose to use calculated Rankine passive pressure or other determination of soil bearing strength based on actual soil properties.

On horizontal bends, the placement of the thrust block is against the outer edge of the bend. On vertical down bends, similar placement would put the thrust block on top of the pipe. Besides the danger of crushing the pipe, the fill dirt on top of the block provides marginal bearing support. Instead, the vertical down bend is stabilized with a gravity thrust block attached to the underside of the bend (see Figure 26.21-b). The vertical component ( $T_y = PA \sin \theta$ ) of the thrust force is countered by the weight of the block itself. The required volume of the block is

$$V_G = \frac{S_f PA (\sin \theta)}{W_m} \quad (26.12)$$

where  $V_G$  = the volume of the block and  $W_M$  = density of the block material.

The horizontal component of the thrust force,  $T_x = PA(1 - \cos \theta)$ , is transferred to the soil by the right side bearing area of the block.

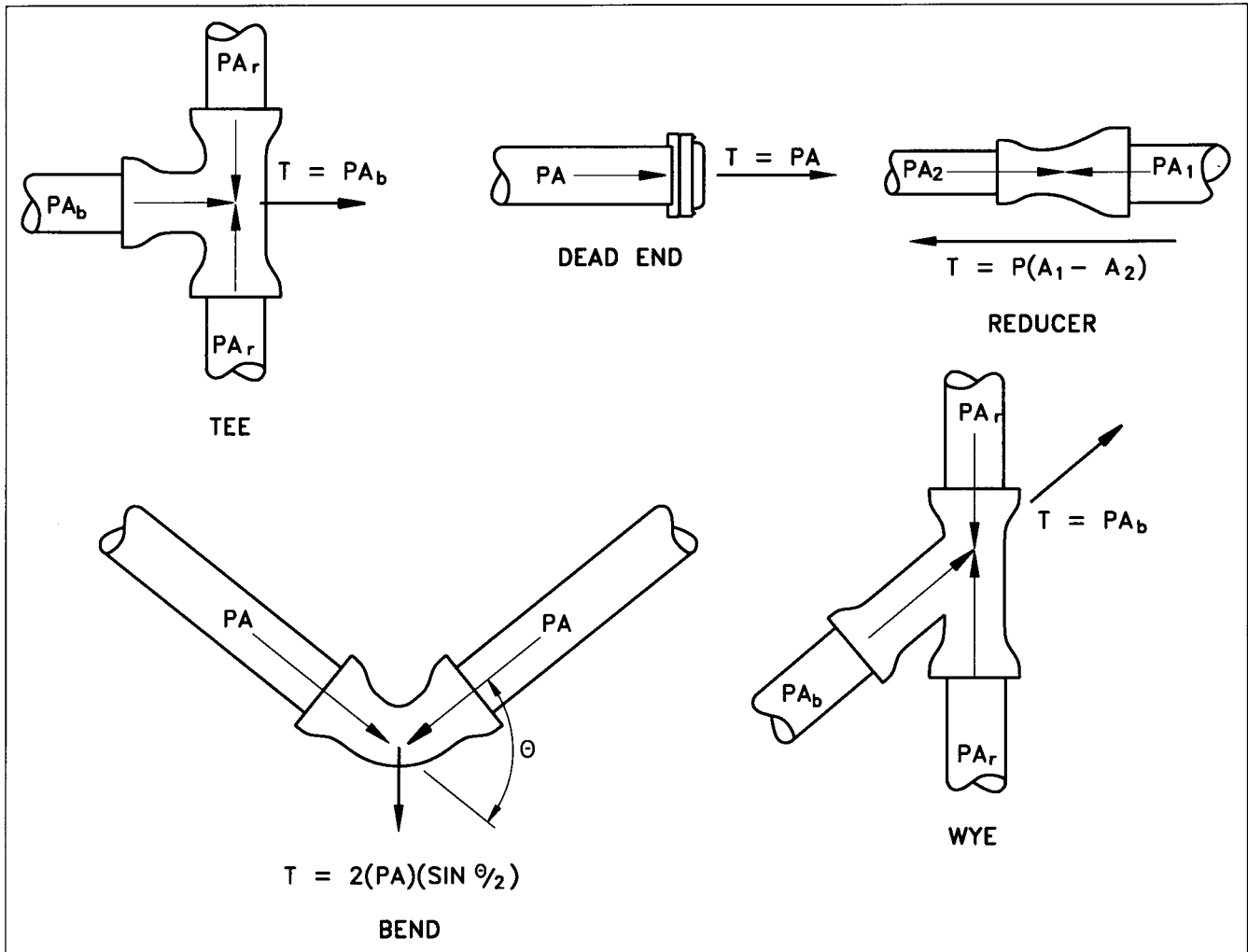
In situations where thrust blocks cannot be used, another common method for thrust resistance is the use of restrained joints. A restrained joint is a specially designed joint that provides longitudinal restraint. The thrust force at the component is transferred to the surrounding soil through frictional resistance and bearing from a predetermined length of pipe.

Other types of thrust resistance include steel straps attached to boulders and bedrock (not very common), and tie rods connecting collars on either side of a joint.

## SYSTEM LAYOUT CONSIDERATIONS

### Pipe Network

In residential areas, distribution lines typically follow the street network and are usually located within the public right-of-way. Some localities require the location of the wa-



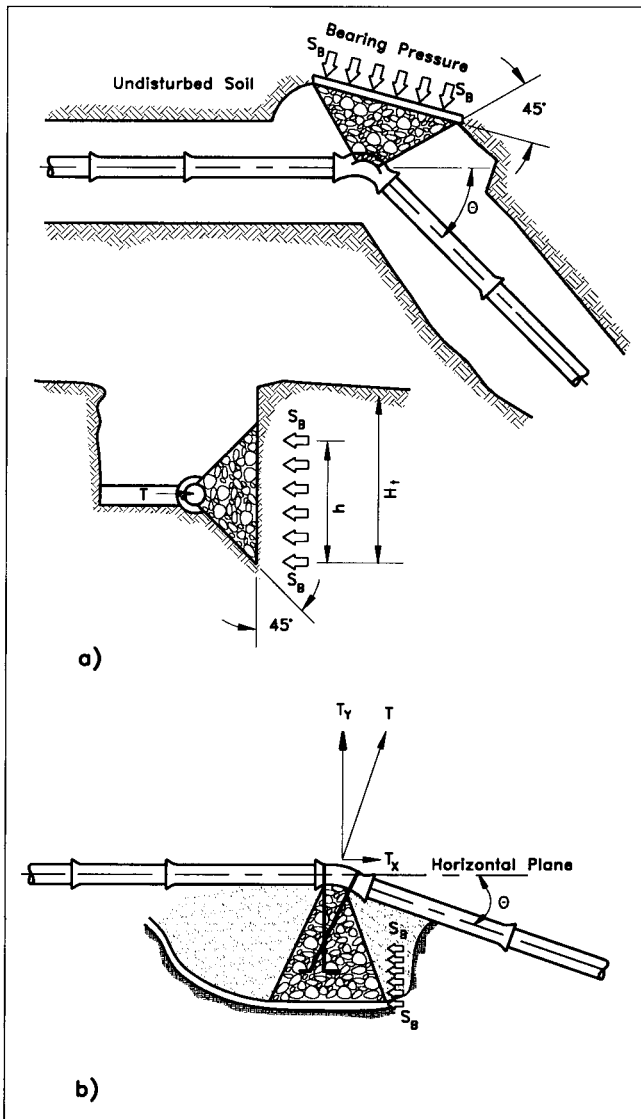
**FIGURE 26.25** Thrust forces and line of action for selected fittings. Reprinted with permission of Ductile Iron Pipe Research Association, Birmingham, Alabama.

ter line to be a fixed distance from the centerline, curb line, or right-of-way line and on the same side of the street for uniformity. For instance, in some parts of the United States, the water line is laid on the north and east side of the centerline. Whether the water main is located under the pavement or behind the curb line is a matter of preference set according to local standards.

Although the grid layout is preferable to the branching layout, dead-end lines cannot always be avoided. Therefore, most distribution systems are a combination of loops and branches, with the branch lines kept to a minimum. Many branch lines are temporary and are built for the convenience of future connections needed when new developments are added and streets are extended. Cul-de-sac streets, common in residential developments, make looping of distribution lines difficult since easements are required to extend the water line through private property. The water supply company, in its desire to minimize dead-end water lines, may require the developer to provide easements through the project for inter-connecting the water line. The

easements should preferably run through out-parcels and common land rather than individual lots. However, if the water line must pass through a lot, the preferred location is along lot lines, with the lot lines coincident with the edge of the easement. Note: this method of locating the easement must comply with other subdivision and zoning constraints. For example, the water line must meet the criteria for minimum distance to dwellings and other structures.

Location of water mains in commercial areas and high-density residential areas (e.g. townhouses and condominiums) is dictated, in part, by other utilities, their structures, location of fire prevention and control devices for the internal parts of the building (e.g., fire hose connections, sprinkler systems), location of fire hydrants and type of metering system. Generally, these sites contain a more complex maze of underground utilities within the travelways that present obstacles in the layout. To illustrate: the travelways are narrower than public street right-of-ways; the sites contain higher impervious areas, and this translates to more storm sewer pipe and inlets; and the higher density



**FIGURE 26.26** Schematic for (a) bearing block; (b) gravity block. Reprinted with permission of Ductile Iron Pipe Research Association, Birmingham, Alabama.

usually means more sanitary connections. Since water lines are normally non-gravity-dependent, the water line can be adapted to weave over, under and around the other gravity-dependent utilities.

Although water mains are usually no smaller than 6 in in diameter, since this size is common for stub lines serving fire hydrants and lines that only serve a few residential units, 4-in. lines may be acceptable for special situations. Typical pipe sizes for water mains are 4, 6, 8, 12, 16, 18, and 24-in. diameters. The size varies according to the specifications of the jurisdiction and the availability of pipe materials. Since the velocity increases as the diameter decreases, smaller diameter pipes result in higher head-losses for the same discharge. Velocities of 3 fps to 6 fps at normal working conditions are preferred, although higher velocities in short lengths of pipe for short periods are tol-

**TABLE 26.10** Bearing Strength of Generalized Soils

SOIL	STRENGTH ( $S_B$ ) (lb/ft <sup>2</sup> )
Muck	0
Soft clay	1,000
Silt	1,500
Sandy silt	3,000
Sand	4,000
Sandy clay	6,000
Hard clay	9,000

\*Although the above bearing strength values have been used successfully in the design of thrust blocks and are considered to be conservative, their accuracy is totally dependent on accurate soil identification and evaluation. The ultimate responsibility for selecting the proper bearing strength of a particular soil type must rest with the design engineer. Reprinted with permission of Ductile Iron Pipe Research Association, Birmingham, Alabama.

erable. Note: sustained high-discharge velocities scour the interior of pipes.

For health and safety reasons, the horizontal distance between a water line and sanitary sewer line or manhole should be a minimum 10 ft. Pipes and pipe joints may develop leaks due to deterioration, improper construction, and excessive pressures. Contamination of the water supply from the sanitary wastewater system presents a potential for creating a major health problem as well as a major cost in locating the point of contamination and repairing it. When the water and sanitary sewer line are essentially parallel and the horizontal distance is less than 10 ft, special provisions are done to ensure no contamination. When the water main is in the vicinity of a sanitary sewer, the water main should be above the sanitary line, with a minimum distance of 18 in between the invert of the water line and the crown of the sanitary pipe. In addition, pressure pipe should be used for the sanitary sewer, and water-tight joints should be used to provide additional protection from contamination. Sanitary sewer manholes in the vicinity of water mains must also be of water-tight construction. As an alternative, design standards may require concrete encasement of the water line when it falls in close proximity to sanitary lines. In the extreme cases, when the sanitary line must be above the water main, the sanitary sewer should have adequate support (e.g., concrete cradle) to prevent excessive deflection, settling on, and breaking of the water line. If the sanitary sewer crosses the water main, not only should upgraded materials be used, but the water-main section of pipe should be centered at the point of crossing to put the water main's pipe joints as far from the sanitary pipe as possible.

Water lines are set at a depth below the frost line. The engineer profiles the water line after the horizontal layout is complete to show how the water line weaves over and under other utilities in the development. Minimum clearance between water main and storm sewer is approximately one ft. When the water main passes under a storm sewer pipe, additional structural support to the storm pipe should be considered to prevent crushing of the water main. When water mains pass over utilities at less than minimum depth, consideration for freezing is necessary.

### Valve Locations

Isolation valves are placed on line for shutting down a section of the system for repairs and maintenance. Recommended valve spacing ranges from 500 to 1200-ft intervals. Since the purpose of isolation valves is to shut down a section for repairs, the objective in placing the valves is to balance the economics of using as few valves as possible to inconvenience the smallest number of customers when a section is shut down.

Most water main systems follow a community's public road. In moderate to high-density residential and commercial areas (i.e.,  $>3$  DU/AC), the frequency of street intersections is typically less than 1000 ft and in many developments 500 ft and less. At these intersections, tees and crosses divide the water line into other feeder lines. Included with these tees and crosses are valves that allow for various ways to segment small service areas. Generally, valves are placed on two sides of a tee and three sides of a cross. At tees, one valve is placed on the minor line. Likewise with crosses, valves are placed at each side of the minor connection (see Figure 26.27-a). The location of the remaining valves for the cross and tee depends on the location of the valves at the next junction upstream and downstream from the point under consideration. Referring to Figure 26.27-a, recognize that two valves at each tee can accomplish the same results as three valves. The water-main segments of B-D, D-F, and C-E can be isolated without the need for a valve at P.

The exception to the above-noted rule occurs on tees where the stub side connects to either a reasonably large commercial building or a fire hydrant. In these cases, valves are placed on all three sides of the tee as shown in Figure 26.27-b. The two valves along the distribution line allow flows to the building or hydrant, even though part of the nearby distribution line may be shut down. The third valve on the stub side allows the building or hydrant to be isolated without disrupting flows along the water main.

Valves are also commonly located where the distribution line has been downsized. As the number of service connections decrease on the line, for instance at a terminating street, the distribution line's diameter can be reduced because of reduced demand. A reducer is a component that allows for the connection of a large pipe to a smaller pipe. Typically, as an economic consideration, valves are placed after the reducer on the side with the smaller diameter.

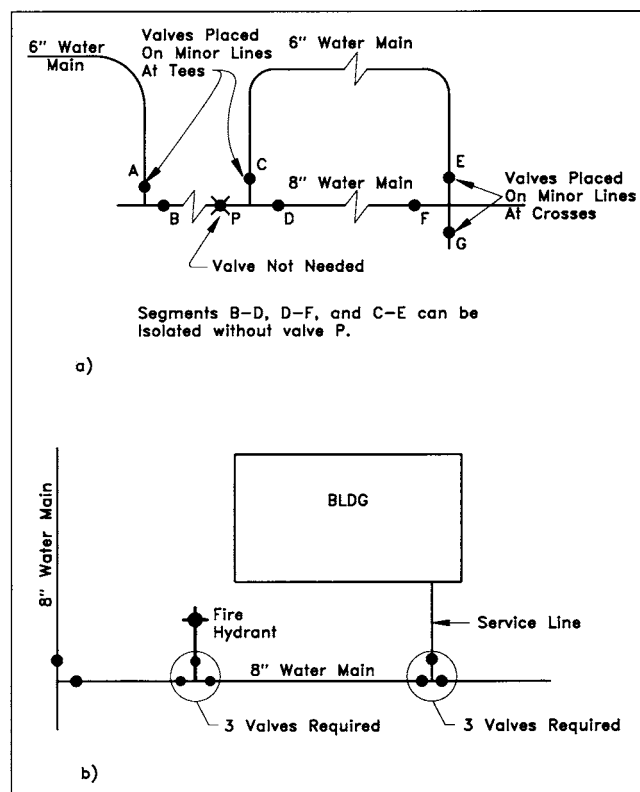


FIGURE 26.27 Valve placement.

The number and location of valves in a development project is a function of the density. In low and moderate-density areas, the placement of valves every 500-ft is inherent due to the street layout. In areas of higher densities (e.g., 10 DU/AC, or apartment complexes), the placement of valves at 500-ft intervals would mean inconveniencing numerous households when a section is shut down. In such cases, the location of valves will depend on the number of service lines that can comfortably be shut down. A reasonable range is 15 to 25 service lines between isolation valves. To do this requires intermediate valves placed on line to limit the number of households to a reasonable level.

Valves are installed in cast iron or PVC valve boxes—a sleeve placed over the valve that prevents backfilling over the valve and allows access from the ground level. The valve box and its lid present a nuisance when located in lawns. When possible, the valve should be placed in accessible areas, but not where the valve box would be covered with soil and overgrown vegetation (e.g. wooded areas). Although locating valves in such areas should be avoided, this is not always possible. In such instances, the valve is enclosed within a vault. The vault may be concrete or another permanent enclosure that protrudes above the ground surface at least six in. A vent stack may also be connected to the vault. The protruding vault and vent stack allows for locating the valve much easier.

Placement of valves for isolation purposes can be very subjective. Occasionally, the water supply company dictates

the location of valves and other controlling components. The water supply company has immediate access to information not readily available to the project engineer. The project engineer can then only make the best guess using the foregoing concepts and personal experience for showing water-line and component location on the plan sheets.

### Fire Hydrant Location

Although aesthetic value of a development has a high priority, there is very little compromise with safety and fire protection. Fire hydrants need to be conspicuously located and highly accessible. Fire fighters in the throes of fighting a blaze need to locate a hydrant by quick glances in obvious directions. Typically, fire hydrants are located 2 to 3 ft behind the curb or edge of pavement. The hydrant is oriented so that the pumper connection faces the street or travelway and is set far enough behind the face of curb to avoid damage from vehicles. Hydrants are placed to be accessible for hose connections and the pumper truck and, therefore, should not be placed high on embankments, in depressions or ditches, or near trees, poles, walls, or where there is insufficient clearance for hose lines (Figure 26.28).

In high-density residential areas (e.g., townhouse and condominium sites) fire hydrants are frequently placed on parking islands near buildings. Consideration is given to the minimum distance between the fire hydrant and closest building it is intended to serve. Local codes prescribe the safe distance for which the fire fighters can access a hydrant without sustaining injury from the burning building and this is approximately 50 to 75 ft. Therefore, several hydrants may be needed throughout high-density and commercial areas to protect the building. Another consideration when locating hydrants is the placement of fire hoses. For example, BOCA fire codes do not recommend placement of unprotected fire hoses across travel ways. Placement of fire hydrants on both sides of the street to protect the building opposite from the fire hydrant will not suffice because of the BOCA restrictions on placement of fire hoses.

On commercial sites, hydrant spacing is governed by the estimated coverage as determined by fire flow requirements, the course of the unrolled fire hose, and the location of fire hose connections on the building (in addition to any other local requirements). Since fire hoses are unrolled along the travelways and grassy areas, the distance between hydrants is not measured along a straight line connecting the hydrants. Subsequently, for streets and travel ways with many short radius curves and obstructions (e.g., fences and walls), more fire hydrants will be needed. Commercial buildings may be required to have an exterior fire hose manifold (e.g., standpipe, siamese connections). The location of this manifold on the building must be near (probably within 100 ft) a fire hydrant.

The ISO recommends hydrant distribution according to the coverage presented in Table 26.11. Rule-of-thumb hydrant spacing ranges from 300 ft to 1,000-ft intervals. Actual spacing considers the type and size of the dwelling,

land use, risk (e.g., high-value district in terms of property cost and public safety) and local controlling ordinances.

To the extent possible, placement of hydrants should not detract from the streetscape. The fire hydrant needs to be visible and accessible to the fire fighters and yet not be obtrusive. For example, in single-family detached subdivisions, a hydrant placed behind the curb at the side property line extended would serve both purposes. A fire hydrant in the middle of the front property line may be unattractive—especially if the house sits close to the front property line.

In placing fire hydrants in a residential subdivision, a systematic method would be to start on the cul-de-sac streets. Determine the fire flow coverage or the maximum distance from a fire hydrant to the farthest buildings as prescribed by local codes. Using this distance, place a fire hydrant as far away from the houses within the cul-de-sac as this distance allows. Proceed along the cul-de-sac street toward the through street, placing fire hydrants at the appropriate distances, considering hydrant spacing and prescribed maximum distances to dwellings. After all cul-de-sac streets have the appropriate number of hydrants, the hydrants along through streets are located to provide fire protection not covered by the cul-de-sac fire hydrants or any existing fire hydrants.

A map, such as the one of Figure 26.29, showing hydrant and valve locations might be available from the water utility company or Department of Public Works. A copy of a map such as this for the project area should be obtained prior to beginning the design. This provides information for assessing water availability and hydrant location during the site-analysis phase.

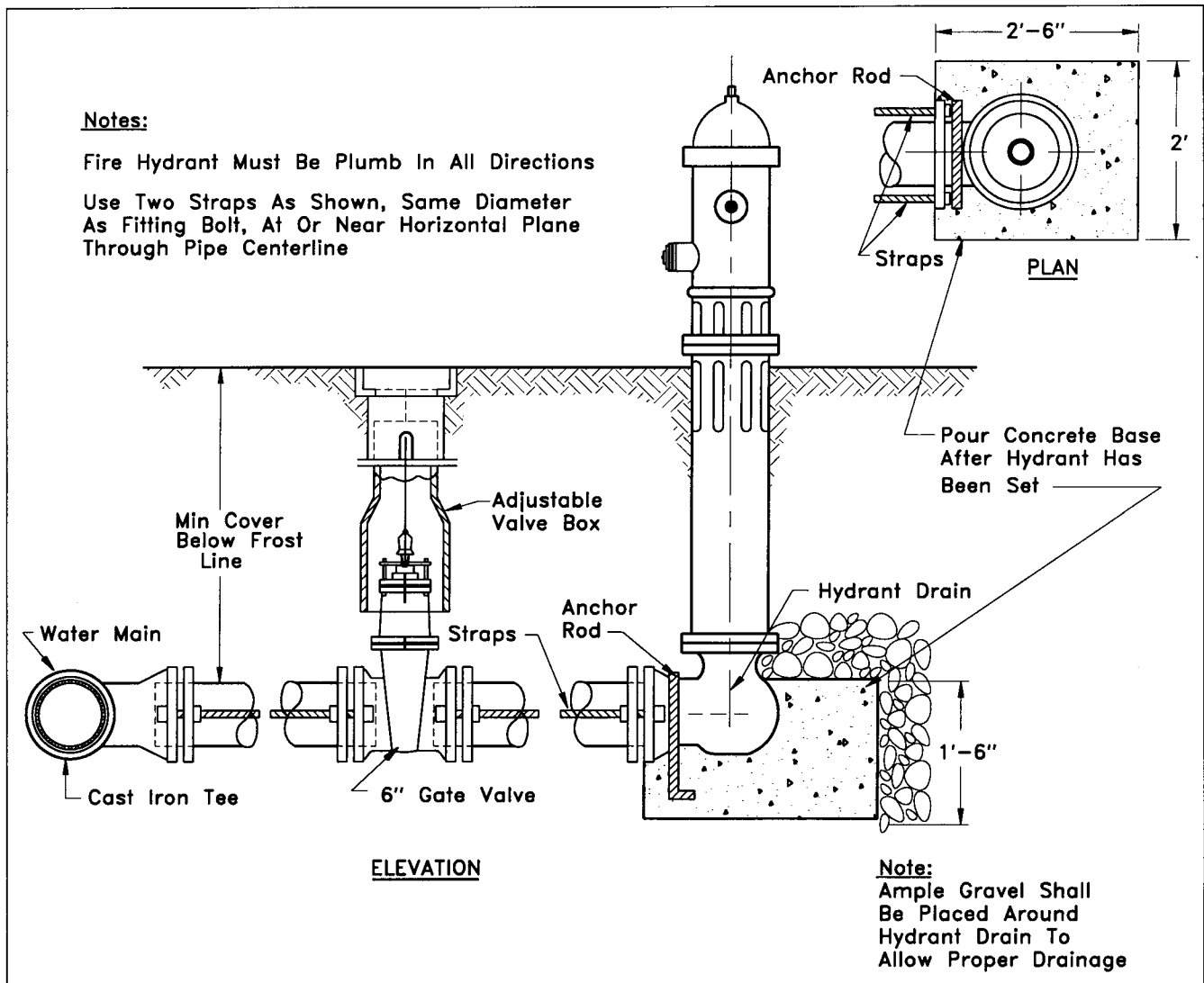
### Pipe Curvature

Gradual change of direction with pipe is accomplished by using bends or simulating curvature through succeeding deflection of pipe joints. The approximate radius of curvature depends on the type of pipe joint, length of pipe, and pipe diameter. Figure 26.30 shows details for joint deflections for mechanical and push-on type pipe joints.

### Connections

During construction, building service connections are extended from the water main to the right of way line. Whereas the water main may be 6- to 12-in.-diameter pipe, the service connections for residential dwellings are typically  $\frac{3}{4}$  to 1-in.-diameter. A corporation stop assembly (see Figure 26.31) connects the service line to the water main at a point above the horizontal center of the water main. This location does not draw off any sediment from the bottom of the water main, is an easily accessible location for repairs, and allows trapped air to escape from the water main and be expelled through water fixtures. However, the specific location of the connection depends on the policy of the utility company.

The service connection ends at the meter box or curb box. The meter box contains the shut-off service valve, a



**FIGURE 26.28** Typical hydrant settings.

yoke, and the water meter. The yoke is a special fitting to hold the service pipe stubs at the proper alignment for connecting the meter. The service box contains only the service valve. In some areas, the building contractor is responsible for installation of the meter and the service line leading to the house, and in others the water utility company manages the installation of the water main and service lines. The stubbed service lines are installed during construction of the water main. This allows the house connections to be made as the houses are built without cutting into new pavement.

### Connecting to an Existing Water Line

Special types of construction methods are used to connect the proposed water main to an existing water main, when the existing service cannot be shut down—even for the short time needed to make the connection. The type of

method selected depends on the specific site conditions and the locality.

One common method uses a tapping sleeve and valve. As illustrated in Figure 26.32, the tapping sleeve and valve consists of two half-sections of pipe that are strapped over the existing line. Special equipment designed to fit over a valve cuts a hole in the existing water main through the opened valve fixture. After the hole is cut, the equipment is withdrawn and the valve is closed. The water main is extended and the valve is opened when the extended water main has been approved for service. The advantage of the tapping sleeve and valve is that it does not require existing service to be shut down while the connection is made. However, this type of connection, sometimes referred to as a wet-tap, can be expensive.

If isolation valves are located in close proximity to the proposed connection, they can be closed while the connec-

**TABLE 26.11 Standard Hydrant Distribution**

<b>FIRE FLOW REQUIRED (gpm)</b>	<b>AVERAGE AREA PER HYDRANT (ft<sup>2</sup>)</b>
1000 or less	160,000
1500	150,000
2000	140,000
2500	130,000
3000	120,000
3500	110,000
4000	100,000
4500	95,000
5000	90,000
5500	85,000
6000	80,000
6500	75,000
7000	70,000
7500	65,000
8000	60,000
8500	57,500
9000	55,000
10,000	50,000
11,000	45,000
12,000	40,000

(Reprinted with permission of Insurance Services Office, Inc. Copyright © 1974).

tion is made. While the valves are closed, the existing pipe can be cut. The cut section is removed, and a tee and sleeve is inserted in its place. Note: this method requires inconveniencing the customers while the construction is performed.

Another method consists of inserting valves onto the existing line in the area where the connection is to be made. The valves are installed similar to the wet-tap method. The valves are closed and the existing water line is cut and replaced by the fixture. However, like the wet-tap, this method can be expensive.

Many connections of appurtenances require additional stability due to pressure forces. Besides using thrust blocks, stability is attained by strapping the appurtenances to other stable parts. Strapping consists of connecting one fixture to another fixture or pipe by using several rods attached at the flanges.

### Cross Connections

A cross connection is a link through which contaminated water enters the potable water supply. The most obvious situation in which this occurs is when the potable water supply is directly connected to a contaminated source, either through an error during construction or poor hydraulic design. Although the system of checks and balances that exists during the design and approval process minimizes the chances of such a connection occurring, the repercussions of even one occurrence is costly both in human safety and monetary damages. Even if not physically connected to the water-supply system, cross connections can occur when back siphonage and back flows of contaminated water enter the potable-water system. Back siphonage, a type of back flow, results from negative pressures within the distributing pipes of the potable water supply, which draw water from the surrounding area into the system. Back siphonage can occur from inadequate pressure at the suction side of a booster pump, a water main break causing significantly reduced pressure in the system, and reduced pressure in the system caused during repairs.

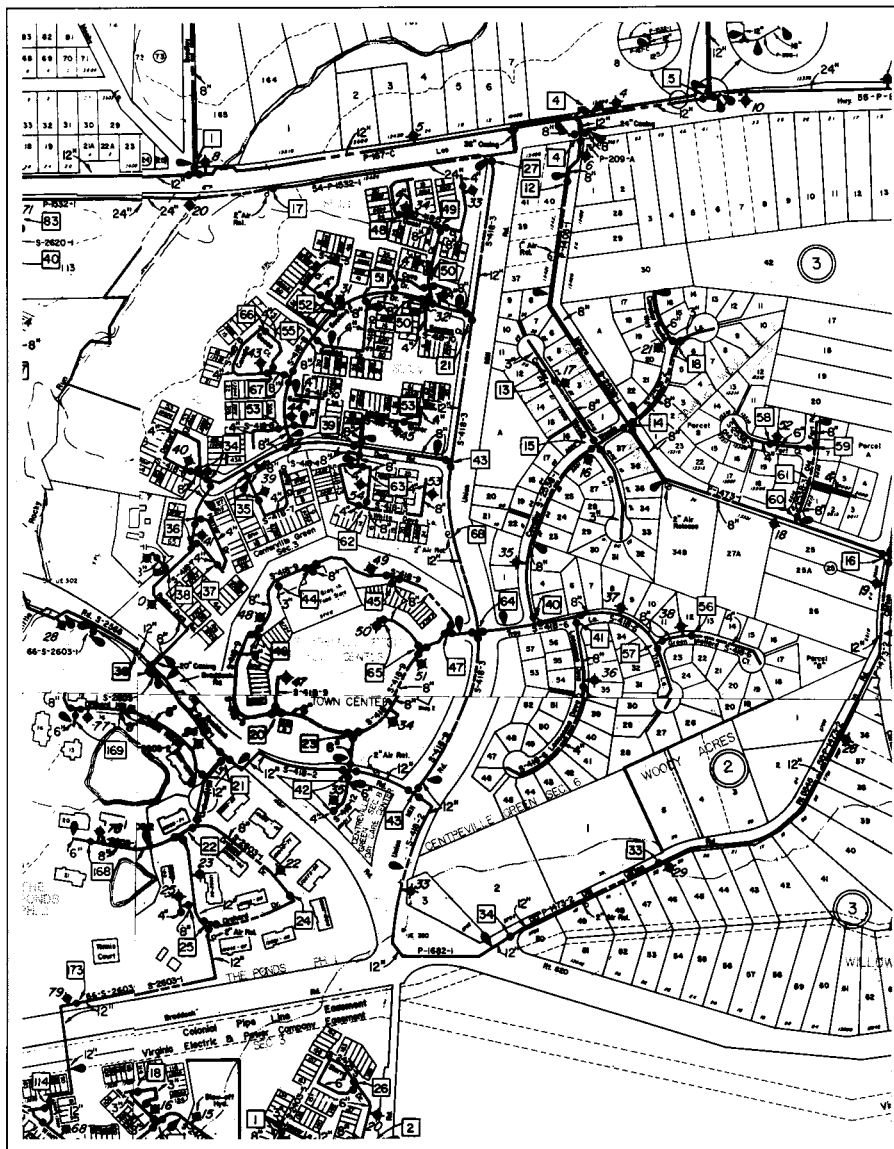
An actual situation where contamination of the water supply resulted from back flow occurred in 1971. As reported by the EPA:

A contractor using a tank truck with a rig designed to pump and spray a mixture of water, fertilizer, grass seed and wood pulp was working on the grounds of a subdivision. The contractor was using a direct connection to a fire hydrant to fill the tank with water, which was then mixed with the fertilizer, etc. A high pressure pump then sprayed the mixture onto the ground. As the wood pulp circulated through the tank piping system, it plugged one of the lines while the pump continued to run creating a very high pressure in the tank. This pressure was higher than the water-supply system pressure and it forced the solution of fertilizer into the water system. Several people in the subdivision became ill after drinking the water, but the contamination was discovered and quick action in flushing and disinfecting the lines eliminated the danger.<sup>2</sup>

Although certain precautionary measures can be used to reduce the potential for cross connection during the design, as long as the human element is present in design and construction, the potential for cross connection will always exist. The most direct and easiest method for back flow prevention is the inclusion of an air space between the free

<sup>2</sup> Cross Connection Control Manual, USEPA 1975





**FIGURE 26.29** Municipal map showing waterline, valve and hydrant location in residential area.

flowing discharge end of the pipe and the flood level rim of the fixture. Other mechanical methods, used mainly in the smaller distribution and service lines, include vacuum breakers, double check valves, and reduced-pressure-principle devices.

### SYSTEM ANALYSIS

Analysis and design of a water-distribution system is a complex process. Due to the availability of computers and the myriad software programs available to analyze complex systems, detailed long-hand computations have become virtually obsolete. This does not de-emphasize the importance of the fundamental knowledge of hydraulics. Very often, the computer software is used as a “black box” to generate answers from the input data. Although it is not imperative that the program users fully understand the program and

the numerical methods used within the program, it is imperative that the users understand the limitations of the program and the reliability of the input data. To intuitively get a feel for the correctness of the output, the engineer still needs to understand the fundamental methods typically used in the software program. The results from a computer software program are only as good as the input data and the numerical methods used within the program.

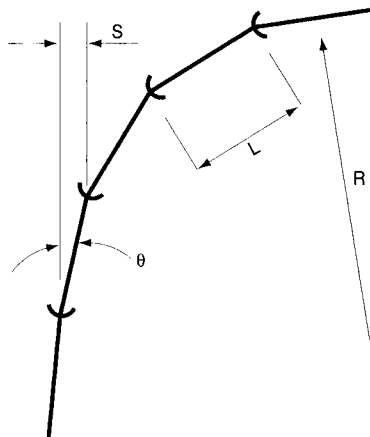
### Pipe Flow

In general, flow through a pipe conduit is affected by its hydraulic and geometric properties. Length and shape are the significant geometric properties; hydraulic properties include pipe material (for roughness), type and number of components (e.g., valves, reducers, bends, tees, etc.) and pressure. Analysis of fluid flow is a matter of determining

**Pipeline Curve Geometry**

θ = deflection angle  
 S = joint deflection offset  
 L = laying length  
 R = radius of curve

$$R = \frac{L}{2 \tan \frac{\theta}{2}}$$



**Maximum Joint Deflection Full-Length Pipe—Mechanical-Joint Pipe**

Nominal Pipe Size <i>in.</i>	Deflection Angle-θ <i>deg</i>	Maximum Offset—S <i>in. (m)</i>		Approx. Radius of Curve—R Produced by Succession of Joints <i>ft. (m)</i>					
				L = 18 ft (5.5 m)	L = 20 ft (6.1 m)	L = 18 ft (5.5 m)	L = 20 ft (6.1 m)		
		3	8-18	31	(0.79)	35	(0.89)	125	(38)
4	8-18	31	(0.79)	35	(0.89)	125	(38)	140	(43)
6	7-07	27	(0.69)	30	(0.76)	145	(44)	160	(49)
8	5-21	20	(0.51)	22	(0.56)	195	(59)	220	(67)
10	5-21	20	(0.51)	22	(0.56)	195	(59)	220	(67)
12	5-21	20	(0.51)	22	(0.56)	195	(59)	220	(67)
14	3-35	13.5	(0.34)	15	(0.38)	285	(87)	320	(98)
16	3-35	13.5	(0.34)	15	(0.38)	285	(87)	320	(98)
18	3-00	11	(0.28)	12	(0.30)	340	(104)	380	(116)
20	3-00	11	(0.28)	12	(0.30)	340	(104)	380	(116)
24	2-23	9	(0.23)	10	(0.25)	450	(137)	500	(152)
30	2-23	9	(0.23)	10	(0.25)	450	(137)	500	(152)
36	2-05	8	(0.20)	9	(0.23)	500	(152)	550	(167)
42	2-00	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)
48	2-00	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)

**Maximum Joint Deflection\* Full-Length Pipe—Push-On Type Joint Pipe**

Nominal Pipe Size <i>in.</i>	Deflection Angle-θ <i>deg</i>	Maximum Offset—S <i>in. (m)</i>		Approx. Radius of Curve—R Produced by Succession of Joints <i>ft. (m)</i>					
				L = 18 ft (5.5 m)	L = 20 ft (6.1 m)	L = 18 ft (5.5 m)	L = 20 ft (6.1 m)		
		3	5	19	(0.48)	21	(0.53)	205	(62)
4	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
6	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
8	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
10	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
12	5	19	(0.48)	21	(0.53)	205	(62)	230	(70)
14	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
16	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
18	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
20	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
24	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
30	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
36	3*	11	(0.28)	12	(0.30)	340	(104)	380	(116)
42	2*	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)
48	2*	7.5	(0.19)	8	(0.20)	510	(155)	570	(174)
54	1 1/2*	5.5	(0.14)	6	(0.15)	680	(207)	760	(232)

\* For 14-in. and larger push-on joints, maximum deflection angle may be larger than shown above. Consult the manufacturer.

**FIGURE 26.30** Joint deflections for mechanical and push-on joints. Reprinted from American Water Works Association Standard for Ductile-Iron Water Mains and Their Appurtenances, by permission. Copyright © 1993, American Water Works Association.

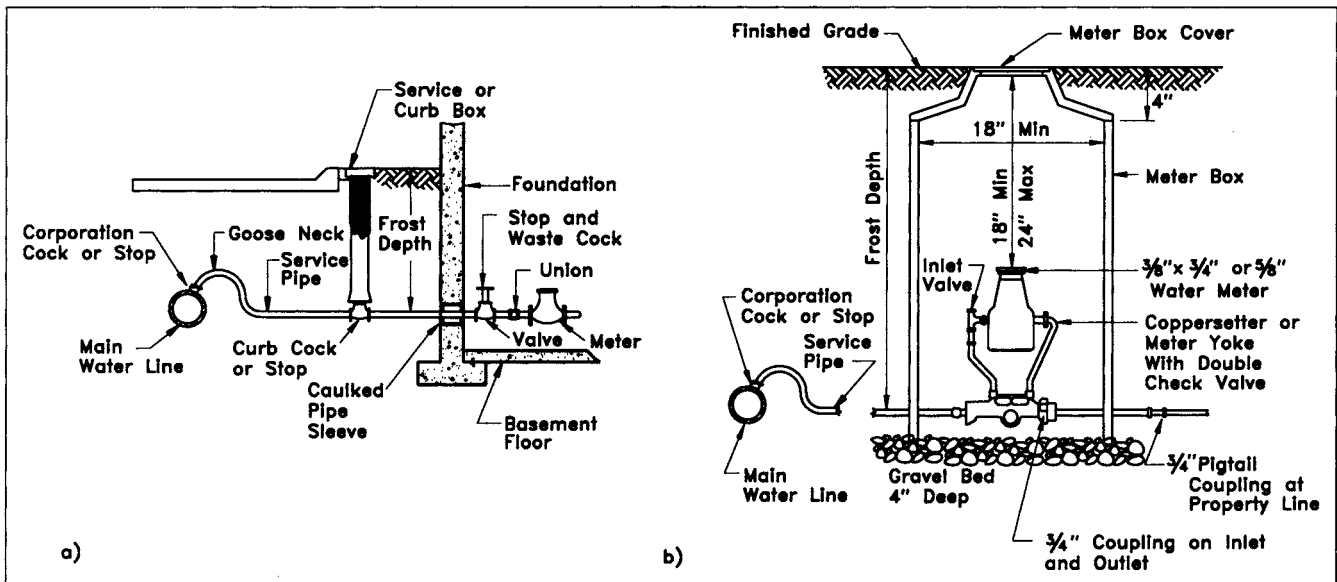


FIGURE 26.31 Typical service connections.

the total hydraulic energy of the fluid. To do so requires the conversion of the fluid's known energy-related parameters—pressure, velocity, and relative elevation—to units of energy.

Most water-distribution systems function as pressure flow. Mathematically, system pressure is converted to energy by changing the units of pressure to an equivalent height of fluid. This is done by dividing the pressure by the specific weight of the fluid. Although specific weight is a function of temperature, the value of 62.4 lb/ft<sup>3</sup> is ordinarily assumed constant for the conditions encountered in water distribution systems. Equation 26.13 converts pressure to pressure head—the depth of fluid that creates a pressure identical to the pressure in the pipe.

$$\frac{p}{\gamma} = h \tag{26.13}$$

where  $p$  is the pipe pressure in psf,  $\gamma$  is the specific weight of the fluid (lb/ft<sup>3</sup>) and  $h$  is a depth in ft.

This is also referred to as the flow energy of the fluid. The kinetic energy of a moving fluid, typically referred to as the velocity head, is

$$h_v = \frac{v^2}{2g} \tag{26.14}$$

where  $h_v$  = velocity head,  $v$  = average velocity in the cross section of flow, and  $g$  = gravitational acceleration.

Fluid within the distribution system also has potential energy. This is the energy associated with a fluid's height relative to a reference datum. The datum is arbitrarily selected for the convenience of analyzing the system or identifying other parameters associated with the system. The

variable  $z$  is used in hydraulic nomenclature to reference the potential energy of the system.

Analysis of a system, or components within specific confines of the system, usually requires the comparison of the total energy of the fluid at two locations. From the conservation-of-energy principle, equating the energy at one location to the fluid's energy at the second location produces:

$$\frac{p_1}{\gamma} + \frac{v_1^2}{2g} + z_1 + \sum h_i = \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + z_2 + \sum h_L \tag{26.15}$$

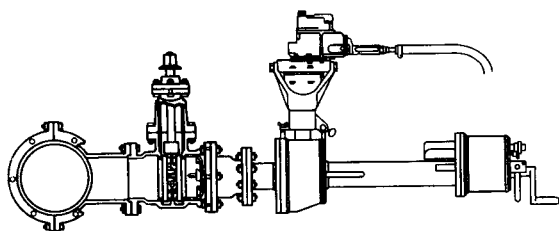
where the  $\sum h_L$  terms represent energy lost by the fluid in going from location one to location two.

Energy lost or taken from the system is attributed to pipe friction, components such as valves and reducers, and sudden enlargements and contractions.

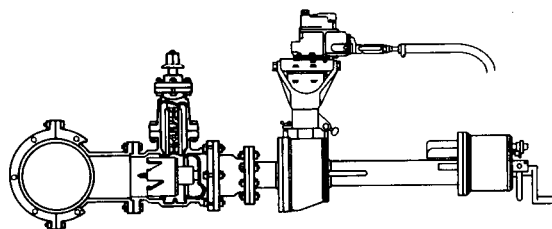
In the energy equation (Equation 26.15), the  $p/\gamma + z$  terms represent the hydraulic grade line (HGL). This is the locus of values above the centerline of the pipe that designate the height to which the water level would rise due to the pressure head if it were not confined within the pipe. Along straight sections of pipe, energy losses in the system cause the HGL to drop in the direction of flow. However, increases in pipe velocity in the downstream direction will cause the HGL to rise. Adding the  $v^2/2g$  term to the HGL represents the energy grade line. This represents the fluid's available energy, and similar to the HGL, decreases in the downstream direction when no external energy is input into the system.

Discharge through pipes is represented by the continuity equation,  $Q = VA$ . Unless there is a leak or the fluid is highly compressible, the continuity equation shows that the amount of water flowing in a length of pipe is constant;

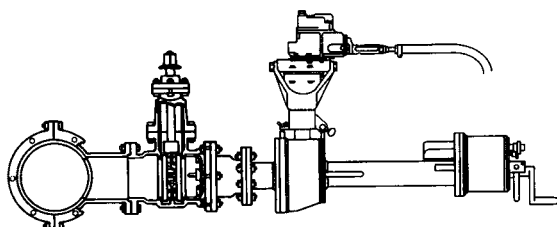
The MUELLER method for making a lateral connection using the MUELLER CL-12 Drilling Machine



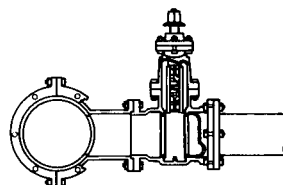
**1** The tapping sleeve and valve are first attached to the main and tested. Then the drilling machine, with a shell cutter attached to the boring bar, is attached to the tapping valve using an adapter. The assembly should be pressure tested prior to making the cut.



**2** With the tapping valve open, the shell cutter and boring bar are advanced to cut the main.



**3** The boring bar is retracted and the tapping valve closed to contain the water pressure.



**4** With the machine removed, the lateral is connected and the tapping valve opened to pressurize the lateral and place it in service.

Large drilling machine selection guide

Machine	Tapping valve size range	Maximum pressure	Operation	Recommended uses
CL-12	2"-12"	250 psig (1724 kPa)	Hand or power	For making lateral connections or inserting valves under pressure
C1-36-99002	2"-24"	500 psig (3447 kPa)	Hand or power	
CC-25	2"-12"	500 psig (3447 kPa)	Hand	



FIGURE 26.32 Tapping sleeve and valve. Reprinted with permission of Mueller Co., Decatur Illinois.

velocity adjusts proportionally to any change in cross-sectional area to keep the discharge constant. Inherent in pipe flow is the energy loss from the effects of the roughness of the pipe material. The pipe's surface roughness affects the velocity across the cross-sectional flow area, with a net result of retarding the flow. Therefore, the equations for determining the discharge in pipes must account for this head (i.e., energy) loss.

The Darcy-Weisbach equation is one of the most popular methods for determining the head loss in pipe flow. This equation,

$$h_L = f \frac{L}{D} \frac{V^2}{2g} \quad (26.16)$$

where

- $h_L$  = head loss,
- $f$  = dimensionless friction factor,
- $L$  = length of pipe,
- $D$  = diameter of the pipe,
- $V$  = average velocity in the pipe, and
- $g$  = gravitational constant)

and experimental verification show the following to be true:

- The head loss varies directly with the length of pipe and inversely with the diameter.
- The head loss varies with the square of the velocity.
- The head loss depends on the roughness of the interior pipe wall.
- The head loss depends on fluid properties of density and viscosity and is independent of pressure.

The last two items are evident from the significance of the parameter  $f$ . The value of  $f$  is dependent on the Reynolds number, the ratio of the inertia forces to the viscous forces of the fluid and the relative roughness of the pipe. These are the only two significant forces of the fluid in a completely filled conduit. For a circular conduit, the Reynolds number ( $R$ ) is

$$R = \frac{\rho DV}{\mu} \quad (26.17)$$

For laminar flows (i.e.,  $R < 2000$ )  $f$  is,

$$f = \frac{64}{R} \quad (26.18)$$

where  $\rho$  is the density and  $\mu$  is the absolute viscosity of the fluid.

For any consistent system of units,  $R$  is a dimensionless number. For turbulent flows,  $f$  has been found through experimentation to be related to the ratio of  $\epsilon/D$  where  $\epsilon$  is the absolute roughness, a parameter that measures the size of the irregularities of the pipe material. This ratio depends

on the pipe material and the pipe diameter. Figure 26.28 shows this ratio for various commercial pipes. The Moody diagram (Figure 26.34) shows the relationship between  $f$ ,  $\epsilon$ , and  $R$ .

The energy loss,  $h_L$ , for a known discharge and pipe can be found by:

1. Determining the Reynolds number, Equation 26.17.
2. Determining  $\epsilon$  from Figure 26.34
3. Determining the  $\epsilon/D$  ratio and enter  $R$  and  $\epsilon/D$  on the Moody diagram (Figure 26.34) and reading the corresponding value of  $f$
4. Using the Darcy-Weisbach Equation 26.16 to find  $h_L$
5. Using the Bernoulli equation and/or the continuity equation to determine other design parameters

In lieu of the Moody diagram, Equation 26.19 can be used to approximate  $f$  for the given constraints.

$$f = \frac{1.325}{[\ln(\epsilon/3.7D + 5.74/R^{0.9})]^2} \quad (26.19)$$

for

$$10^{-6} \leq \frac{\epsilon}{D} \leq 10^{-2}$$

$$5000 \leq R \leq 10^8$$

Pipe analysis problems with incompressible flow require the determination of the variables  $Q$ ,  $L$ ,  $D$ ,  $h_f$  and  $V$ . Recognize that  $V$ ,  $D$  and  $f$  are inter-related. Hence, the solution to solving problems where these parameters are unknown (and therefore  $R$  is unknown) requires a trial-and-error procedure. Since the value of  $f$  changes very slowly at the higher  $R$  values, begin the trial-and-error procedure by assuming a value for either  $f$  or  $R$  and obtain a final solution for  $Q$  or  $V$ . Compute an  $f$  (or  $R$ ) using the final solution, and use the Moody diagram to find  $R$  (or  $f$ ) and compare this to the assumed value. The procedure ends when the computed value is within an acceptable tolerance of the assumed value. For example, if  $D$  is known, the trial-and-error procedure begins by assuming  $f$  and using the Darcy-Weisbach equation to find the velocity. Use this velocity to compute  $R$  and find the corresponding  $f$  value from the Moody diagram. If the two values are within an acceptable tolerance, the procedure ends and the design parameters are known; otherwise, assume another value for  $f$  and repeat the procedure.

Another popular method used to find pipe discharges and other design parameters is the Hazen-Williams equation, shown for use with English units as

$$V = 1.318 CR^{0.63} S^{0.54} \quad (26.20)$$

where

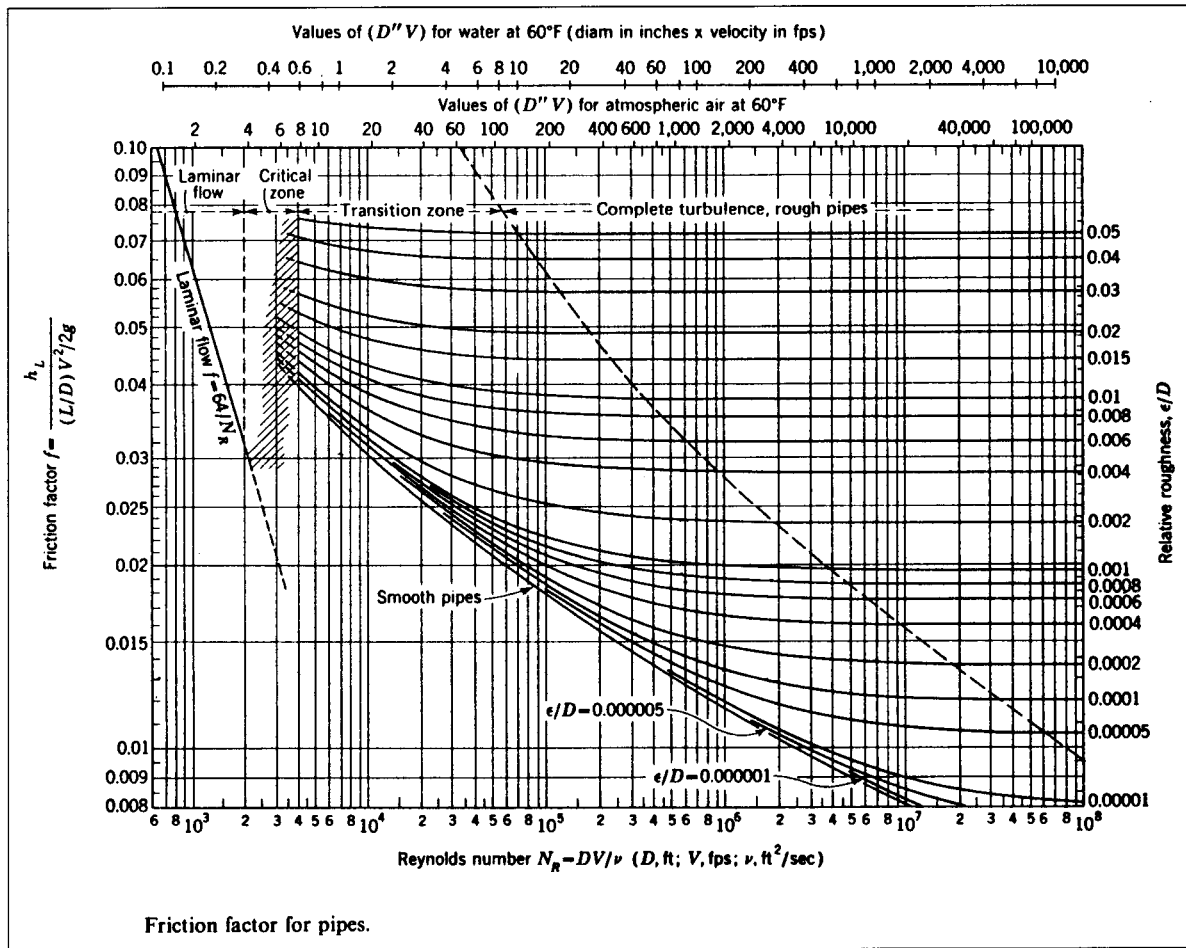


FIGURE 26.33 Moody diagram. (ASME Transactions, vol. 66, 1944. Reprinted by permission of American Society of Mechanical Engineers.)

- $V$  = velocity,
- $C$  = coefficient associated with pipe roughness,
- $R$  = hydraulic radius (=  $D/4$  for circular pipes flowing full), and
- $S$  = energy gradient ( $h_L/L$ ).

Although not dimensionally correct, as many empirical equations tend to be, the Hazen-Williams equation provides generally acceptable discharge estimates. In most problems for design purposes,  $C$  is taken to be 100, but for aged pipe,  $C$  can be as low as 70. Table 26.12 provides a range of  $C$  values for different pipe materials. The advantage in using this equation over the Darcy-Weisbach equation, is the independence of  $C$  from the Reynolds number. Because this equation does not include any terms relating to the physical properties of the fluid, the Hazen-Williams equation is used for water discharges for turbulent conditions only. Additionally, this non-dependence on the physical properties of the water makes the Hazen-Williams equation the preferred equation for pipe flow analysis. Figure 26.35 is the nomograph for the Hazen-Williams equation. Given the value of two of the variables in the equation, the nomograph provides the unknown values of the remaining two variables.

### Minor Losses

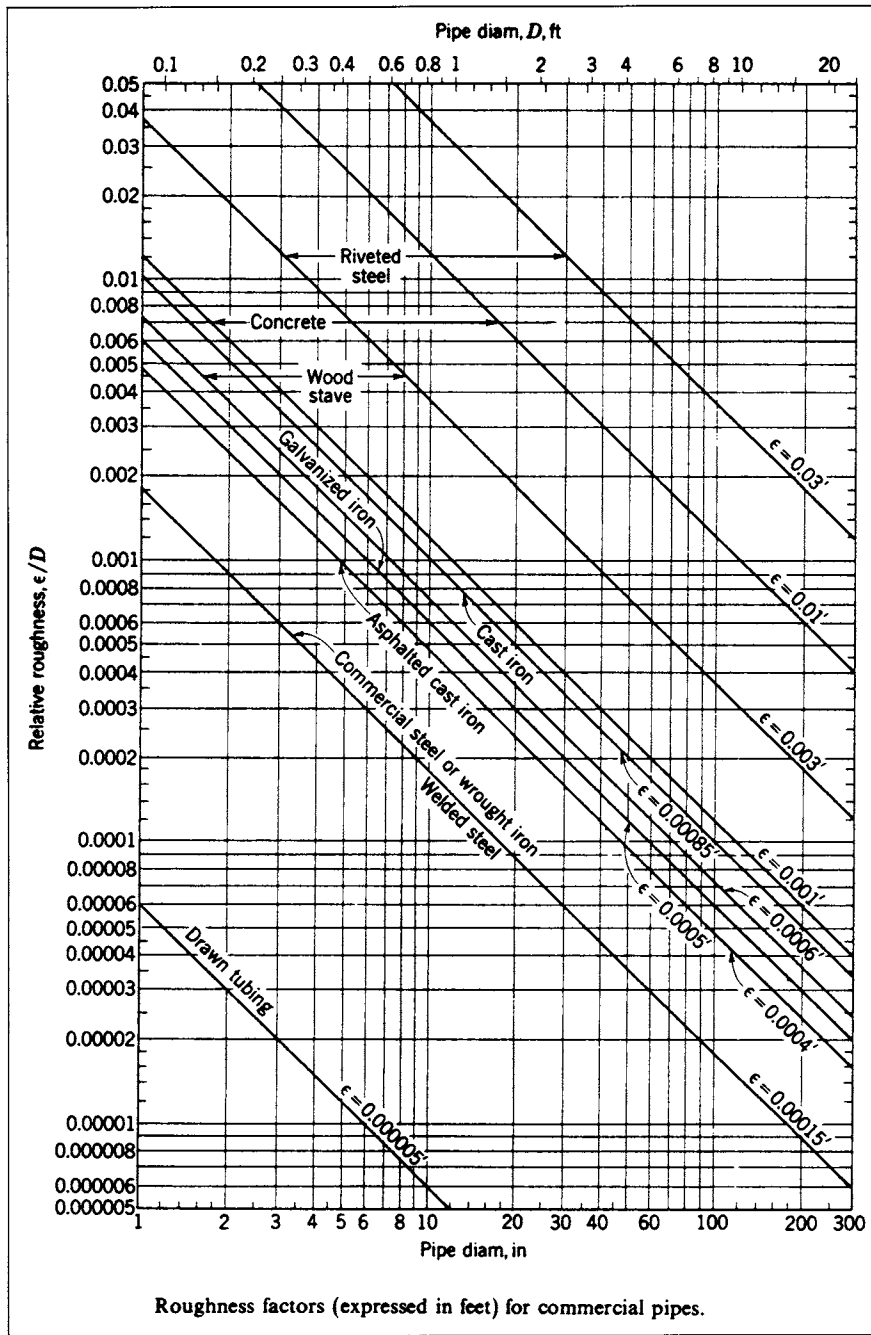
Components in the pipe network contribute to the system's head losses. These are considered as minor losses and, depending on the section of the system under analysis, may or may not need to be incorporated into the design computations. Energy lost through friction is significantly larger than the combined minor losses when pipe lengths exceed  $1000 \pm$  diameters. In these situations, the minor energy losses can be ignored without appreciable effect on the results (due to the uncertainty in the  $C$  coefficient).

The loss coefficient method uses a constant of proportionality based on the configuration of the component. This constant is applied to the velocity head of the water entering or exiting the component. This method computes the minor losses using the form

$$h_v = k \frac{v^2}{2g} \tag{26.21}$$

where  $k$  is the constant of proportionality.

Table 26.13 shows  $k$  factors for various fittings and other water line appurtenances. For additional information on head losses through fittings as well as other components, the manufacturer's specifications should be consulted.



**FIGURE 26.34** Roughness factors for commercial pipes. (ASME Transactions, vol. 66, 1944. Reprinted by permission of American Society of Mechanical Engineers.)

**Equivalent Pipe Lengths**

A pipe network typically consists of loops and branches, with fittings and pipes of various diameters. In order to simplify pipe network computations, it may be advantageous to convert the fittings and various pipe lengths into equivalent lengths of a uniform diameter. The equivalent pipe length represents a length of pipe of known diameter that has either the same head loss as a length of pipe of

different diameter with the same discharge, or the same head loss as a fitting for the given discharge. Using the Darcy-Weisbach equation, an equivalent pipe length for a fitting with a given head loss is

$$f \left( \frac{L_e}{D} \right) \frac{V^2}{2g} = K \left( \frac{V^2}{2g} \right) \quad \text{or} \quad L_e = K \frac{D}{f} \quad (26.22)$$

**TABLE 26.12 Relative Roughness and Hazen-Williams Constants for Various Pipe Materials**

TYPE OF PIPE OR SURFACE	ε(ft)		C		
	RANGE	DESIGN	RANGE	CLEAN	DESIGN
<b>STEEL</b>					
welded and seamless	0.0001–0.0003	0.0002	150–80	140	100
interior riveted, no projecting rivets				139	100
projecting girth rivets				130	100
projecting girth and horizontal rivets				115	100
vitriified, spiral-riveted, flow with lap				110	100
vitriified, spiral-riveted, flow against lap				100	90
corrugated				60	60
<b>MINERAL</b>					
concrete	0.001–0.01	0.004	152–85	120	100
cement-asbestos			160–140	150	140
vitriified clays					110
brick sewer					100
<b>IRON</b>					
cast, plain	0.0004–0.002	0.0008	150–80	130	100
cast, tar (asphalt) coated	0.0002–0.0006	0.0004	145–50	130	100
cast, cement-lined	0.000008	0.000008		150	140
cast, bituminous-lined	0.000008	0.000008	160–130	148	140
cast, centrifugally spun	0.00001	0.00001			
galvanized, plain	0.0002–0.0008	0.0005			
wrought, plain	0.0001–0.0003	0.0002	150–80	130	100
<b>MISCELLANEOUS</b>					
fiber				150	140
copper and brass	0.000005	0.000005	150–120	140	130
wood stave	0.0006–0.003	0.002	145–110	120	110
transite	0.000008	0.000008			
lead, tin, glass		0.000005	150–120	140	130
plastic (PVC and ABS)		0.000005	150–120	140	130

Taken from *Civil Engineering Reference Manual*, 4th edition, by Michael Linderbers, with permission from Professional Publications, Inc., Belmont CA. Copyright © 1989.

where the terms are as defined earlier and  $L_e$  represents the length of pipe of diameter  $D$  that corresponds to the head loss through the component.

The nomograph of Figure 26.38 can be used in lieu of the equations to convert various fixtures to equivalent lengths.

To convert a pipe of one diameter to pipe of different diameter, for a given discharge, either the Darcy-Weisbach equation or the Hazen-Williams equation can be used. After rearranging terms of the Darcy-Weisbach equation, converting one pipe of known diameter  $D$  to another diameter  $D_e$ , of equivalent length  $L_e$  is found by

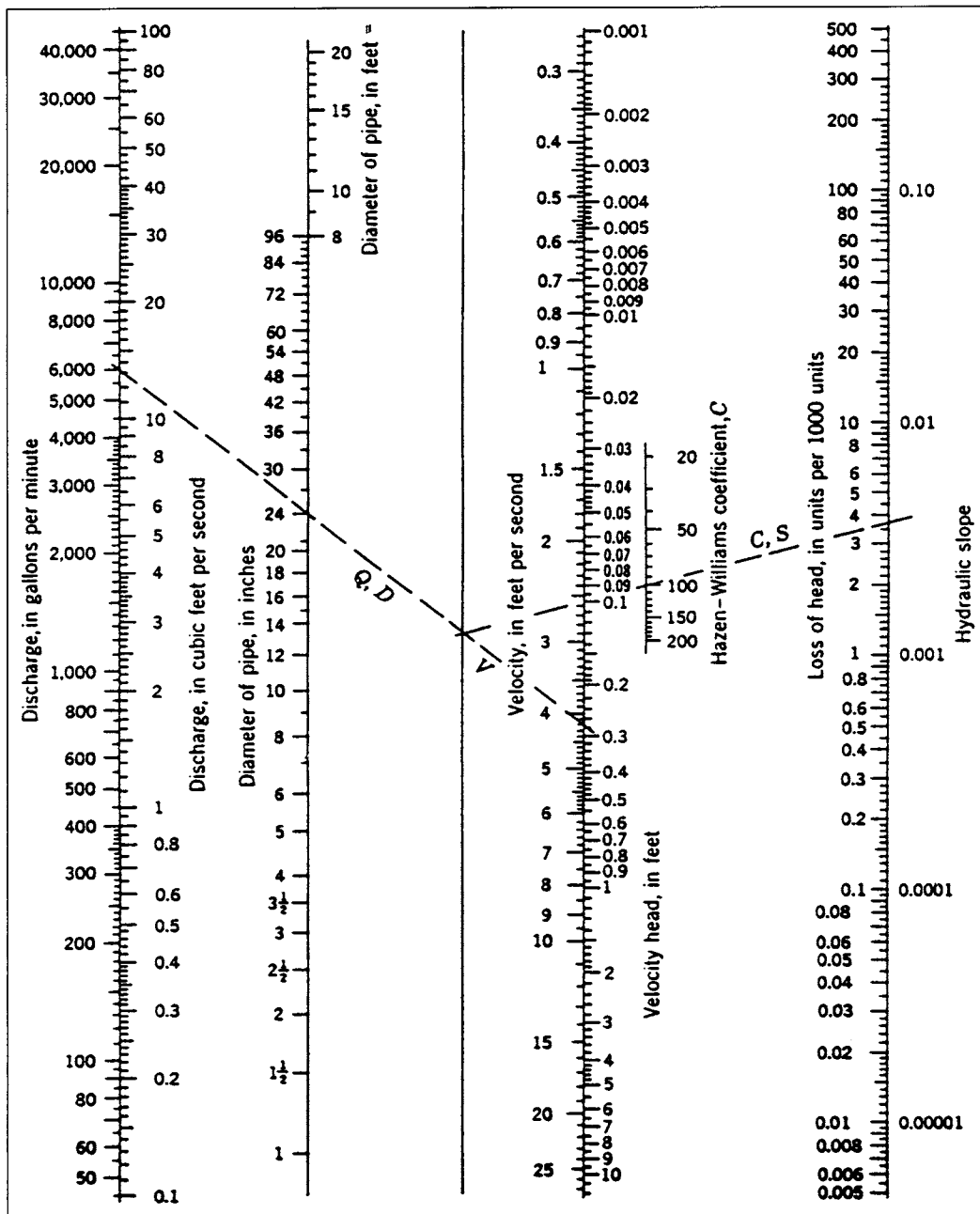
$$L_e = L \left( \frac{f}{f_e} \right) \left( \frac{D_e}{D} \right)^5 \tag{26.23}$$

where  $f$  and  $f_e$  are the friction factors for the given pipe and equivalent length pipe respectively.

Similarly, the Hazen-Williams equation is rearranged to find an equivalent length pipe using the diameters and the coefficients of pipe roughness for the equivalent length pipe,  $C_e$ , and the coefficient of roughness for the given pipe,  $C$ .

$$L_e = L \left( \frac{D_e}{D} \right)^{4.87} \left( \frac{C}{C_e} \right)^{1.85} \tag{26.24}$$





**FIGURE 26.35** Hazen-William nomograph. Design and Construction of Sanitary and Storm Sewers (1969). American Society of Civil Engineers and the Water Pollution Control Federation. Reprinted with permission of ASCE, New York, New York.

As an example, find the equivalent length of 12-in diameter ductile iron pipe ( $C = 100$ ) for 500 ft of 10-in-diameter pipe ( $C = 100$ ) attached with a globe valve at one end ( $Q = 5$  cfs).

The velocity in the 10-in. pipe is

$$V_{10} = \left( \frac{Q}{A_{10}} \right) = \frac{5}{\frac{\pi}{4} \left( \frac{10}{12} \right)^2} = 9.2 \text{ fps} \quad (26.25)$$


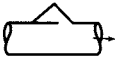



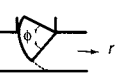

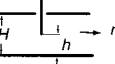

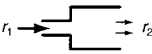

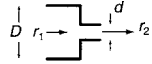

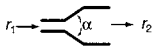

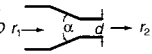

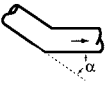
Using the same equation, the velocity in the 12-in.-diameter pipe is 6.4 fps.

From Table 26.13, the head loss coefficient for a globe valve is 10 and the corresponding head loss through the globe valve is

$$h_l = 10 \left( \frac{9.2^2}{2g} \right) = 13.2 \text{ ft} \quad (26.26)$$


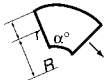


**TABLE 26.13 Table of Local Loss Coefficients**

Use the equation  $h_v = kv^2/2g$  unless otherwise indicated. Energy loss  $E_L$  equals  $h_v$  head loss in feet.

<p><b>Perpendicular square entrance:</b></p>  <p><math>k = 0.50</math> if edge is sharp</p>	<p><b>Check valves:</b></p>  <p>Swing type <math>k = 2.5</math> when fully open                      Ball type <math>k = 70.0</math>                      Lift type <math>k = 12.0</math></p>																																								
<p><b>Perpendicular rounded entrance:</b></p>  <table border="1" data-bbox="279 541 667 619"> <tr> <td><math>R/d =</math></td> <td>0.05</td> <td>0.1</td> <td>0.2</td> <td>0.3</td> <td>0.4</td> </tr> <tr> <td><math>k =</math></td> <td>0.25</td> <td>0.17</td> <td>0.08</td> <td>0.05</td> <td>0.04</td> </tr> </table>	$R/d =$	0.05	0.1	0.2	0.3	0.4	$k =$	0.25	0.17	0.08	0.05	0.04	<p><b>Angle valve:</b></p>  <p><math>k = 5.0</math> if fully open</p>																												
$R/d =$	0.05	0.1	0.2	0.3	0.4																																				
$k =$	0.25	0.17	0.08	0.05	0.04																																				
<p><b>Additional loss due to skewed entrance:</b></p>  <p><math>k = 0.505 + 0.303 \sin \alpha + 0.226 \sin^2 \alpha</math></p>	<p><b>Segment gate in rectangular conduit:</b></p>  <p><math>k = 0.3 + 1.3 [(1/n)]^2</math>                      where <math>n = \phi/\phi_0 =</math> the rate of opening with respect to the central angle</p>																																								
<p><b>Strainer bucket:</b></p>  <p><math>k = 10</math> with foot valve  <math>k = 5.5</math> without foot valve</p>	<p><b>Sluice gate in rectangular conduit:</b></p>  <p><math>k = 0.3 + 1.9 [(1/n) - n]^2</math>                      where <math>n = h/H</math>.</p>																																								
<p><b>Standard tee, entrance to minor line:</b></p>  <p><math>k = 1.8</math></p>	<p><b>Sudden expansion:</b></p>  <p><math>E_L = (1 - \frac{v_2}{v_1})^2 \frac{v_1^2}{2g}</math> or <math>E_L = (\frac{v_1}{v_2} - 1)^2 \frac{v_2^2}{2g}</math></p>																																								
<p><b>Confusor outlet:</b></p>  <table border="1" data-bbox="279 1197 566 1270"> <tr> <td><math>d/D =</math></td> <td>0.5</td> <td>0.6</td> <td>0.8</td> <td>0.9</td> </tr> <tr> <td><math>k =</math></td> <td>5.5</td> <td>4</td> <td>2.55</td> <td>1.1</td> </tr> </table>	$d/D =$	0.5	0.6	0.8	0.9	$k =$	5.5	4	2.55	1.1	<p><b>Sudden contraction:</b></p>  <table border="1" data-bbox="997 1197 1442 1270"> <tr> <td><math>(d/D)^2 =</math></td> <td>0.01</td> <td>0.1</td> <td>0.2</td> <td>0.4</td> <td>0.6</td> <td>0.8</td> </tr> <tr> <td><math>k =</math></td> <td>0.5</td> <td>0.5</td> <td>0.42</td> <td>0.33</td> <td>0.25</td> <td>0.15</td> </tr> </table> <p>use <math>v_2</math> in equation (13.13)</p>	$(d/D)^2 =$	0.01	0.1	0.2	0.4	0.6	0.8	$k =$	0.5	0.5	0.42	0.33	0.25	0.15																
$d/D =$	0.5	0.6	0.8	0.9																																					
$k =$	5.5	4	2.55	1.1																																					
$(d/D)^2 =$	0.01	0.1	0.2	0.4	0.6	0.8																																			
$k =$	0.5	0.5	0.42	0.33	0.25	0.15																																			
<p><b>Exit from pipe into reservoir:</b></p>  <p><math>k = 1.0</math></p>	<p><b>Diffusor:</b></p>  <p><math>E_L = k(v_1^2 - v_2^2)/2g</math></p> <table border="1" data-bbox="997 1449 1324 1522"> <tr> <td><math>\alpha^\circ =</math></td> <td>20</td> <td>40</td> <td>60</td> <td>80</td> </tr> <tr> <td><math>k =</math></td> <td>0.20</td> <td>0.028</td> <td>0.32</td> <td>0.35</td> </tr> </table>	$\alpha^\circ =$	20	40	60	80	$k =$	0.20	0.028	0.32	0.35																														
$\alpha^\circ =$	20	40	60	80																																					
$k =$	0.20	0.028	0.32	0.35																																					
<p><b>Diffusor outlet for <math>D/d &gt; 2</math>:</b></p>  <table border="1" data-bbox="279 1617 566 1690"> <tr> <td><math>\alpha^\circ =</math></td> <td>8</td> <td>15</td> <td>30</td> <td>45</td> </tr> <tr> <td><math>k =</math></td> <td>0.05</td> <td>0.18</td> <td>0.5</td> <td>0.6</td> </tr> </table>	$\alpha^\circ =$	8	15	30	45	$k =$	0.05	0.18	0.5	0.6	<p><b>Confusor:</b></p>  <p><math>E_L = k(v_1^2 - v_2^2)/2g</math></p> <table border="1" data-bbox="997 1690 1442 1764"> <tr> <td><math>\alpha^\circ =</math></td> <td>6</td> <td>10</td> <td>20</td> <td>40</td> <td>60</td> <td>80</td> <td>100</td> <td>120</td> <td>140</td> </tr> <tr> <td><math>D = 3d</math></td> <td>0.12</td> <td>0.16</td> <td>0.39</td> <td>0.80</td> <td>1.0</td> <td>1.06</td> <td>1.04</td> <td>1.04</td> <td>1.04</td> </tr> <tr> <td><math>D = 1.5d</math></td> <td>0.12</td> <td>0.16</td> <td>0.39</td> <td>0.96</td> <td>1.22</td> <td>1.16</td> <td>1.10</td> <td>1.06</td> <td>1.04</td> </tr> </table>	$\alpha^\circ =$	6	10	20	40	60	80	100	120	140	$D = 3d$	0.12	0.16	0.39	0.80	1.0	1.06	1.04	1.04	1.04	$D = 1.5d$	0.12	0.16	0.39	0.96	1.22	1.16	1.10	1.06	1.04
$\alpha^\circ =$	8	15	30	45																																					
$k =$	0.05	0.18	0.5	0.6																																					
$\alpha^\circ =$	6	10	20	40	60	80	100	120	140																																
$D = 3d$	0.12	0.16	0.39	0.80	1.0	1.06	1.04	1.04	1.04																																
$D = 1.5d$	0.12	0.16	0.39	0.96	1.22	1.16	1.10	1.06	1.04																																
<p><b>Gate valve:</b></p>  <table border="1" data-bbox="279 1858 782 1932"> <tr> <td><math>e/D =</math></td> <td>0</td> <td>1/4</td> <td>3/8</td> <td>1/2</td> <td>5/8</td> <td>3/4</td> <td>7/8</td> </tr> <tr> <td><math>k =</math></td> <td>0.15</td> <td>0.26</td> <td>0.81</td> <td>2.06</td> <td>5.52</td> <td>17.0</td> <td>97.8</td> </tr> </table>	$e/D =$	0	1/4	3/8	1/2	5/8	3/4	7/8	$k =$	0.15	0.26	0.81	2.06	5.52	17.0	97.8	<p><b>Sharp elbow:</b></p>  <p><math>k = 67.6 \times 10^{-6}(\alpha^\circ)^{2.17}</math></p>																								
$e/D =$	0	1/4	3/8	1/2	5/8	3/4	7/8																																		
$k =$	0.15	0.26	0.81	2.06	5.52	17.0	97.8																																		

Courtesy of Simon, Andrew L. Hydraulics 1986. Adopted by permission of Prentice Hall, Inc. Englewood Cliffs, NJ.

**TABLE 26.13 Table of Local Loss Coefficients (Continued)**

<b>Globe valve:</b>		<b>Bends:</b>																					
	$k = 10$ when fully open		$k = (0.13 + 1.85(r/R)^{3.5})\sqrt{\alpha/180^\circ}$																				
<b>Rotary valve:</b>		<b>Close return bend:</b>																					
	<table border="1"> <tr> <td><math>\alpha^\circ =</math></td> <td>5</td> <td>10</td> <td>20</td> <td>30</td> <td>40</td> <td>50</td> <td>60</td> <td>70</td> <td>80</td> </tr> <tr> <td><math>k =</math></td> <td>0.05</td> <td>0.29</td> <td>1.56</td> <td>5.47</td> <td>17.3</td> <td>52.6</td> <td>206</td> <td>485</td> <td><math>\infty</math></td> </tr> </table>	$\alpha^\circ =$	5	10	20	30	40	50	60	70	80	$k =$	0.05	0.29	1.56	5.47	17.3	52.6	206	485	$\infty$		$k = 2.2$
$\alpha^\circ =$	5	10	20	30	40	50	60	70	80														
$k =$	0.05	0.29	1.56	5.47	17.3	52.6	206	485	$\infty$														

Using the Hazen-Williams equation, the equivalent length of 12-in.-diameter pipe for the globe valve is

$$L_{12} = \left( \frac{1.318CR^{0.63}h_L^{0.54}}{V} \right)^{1.85} \tag{26.27}$$

$$= \frac{1.318(100) \left( \frac{1}{4} \right)^{0.63} 13.1^{0.54}}{6.4} = 700 \text{ ft}$$

From Equation 13.16, the equivalent length of 12-in.-diameter pipe for 500 ft of 10-in.-diameter pipe is

$$L_{12} = 500 \left( \frac{1}{\left( \frac{10}{12} \right)} \right)^{4.87} \left( \frac{100}{100} \right)^{1.85} = 1,215 \text{ ft} \tag{26.28}$$

The total equivalent length of 12-in.-diameter pipe is  $700 + 1215 = 1915$  ft.

**Fixture Unit Method**

Occasionally, the project engineer may be required to size the water service line to a building. This normally occurs on commercial or other high rise/high-density projects. For the most part, however, the analysis and sizing of the internal distribution system is typically done by mechanical/plumbing engineers. The size of the water-supply line to the building depends on the demand within the building, and this may be related to the number and type of plumbing fixtures in the building, or perhaps the building's use. Since there are several methods available to estimate the demand, the one used depends on the building codes of that jurisdiction.

The simplest method used to estimate demand is one that relates the demand to the building's use. For example, water demand for an office building could be based on the square feet of floor area or the number of employees, while for apartments the demand may be based on the number of units in the building. If this type of method is used, the

demand table for various building uses will be prescribed in the applicable building code.

Another method estimates the demand based on the number and type of plumbing fixtures in the building. However, the size of the water-supply line is based on maximum probable demand rather than the sum of the maximum flows of all of the fixtures. It is highly unlikely that every plumbing fixture would be operating at maximum discharge simultaneously. Under this pretense, the demand is related to the number and type of fixtures, the rates required by these fixtures and the probable simultaneous operation of the fixtures. The unit of measurement used to correlate these flow characteristics is the water-supply fixture unit (WSFU) or "fixture unit."

The fixture unit is a measure of the hydraulic demand of a fixture. It is the average discharge, during use, of an arbitrarily selected plumbing fixture. It takes into account the anticipated discharge, the average duration of flow when the fixture is in use, and the frequency with which the fixture is likely to be used. The WSFU is a factor chosen so that the load-producing effects of different kinds of plumbing fixtures, as well as their conditions of service, can be expressed as multiples of that factor. For example, if a bathtub with a discharge of 7.5 gpm is selected as the arbitrary fixture, a fixture with a discharge of 22 gpm is assigned a fixture unit of 3.

Typical values of fixture units are given in Table 26.14. By knowing the type and number of water supply fixtures, the total WSFU for a building can be determined and then converted to flow rates using Figure 26.31<sup>3</sup>.

After estimating the peak demand, the service line is sized using head loss and velocity parameters for the pipe material selected. Consideration is given to the available pressure in the water main to determine if the pressure is adequate for the calculated losses through the building.

<sup>3</sup> This curve, frequently referred to as the Hunter curve, was first proposed by Roy B. Hunter of the National Bureau of Standards in 1923. Since then it has been modified based on other research data.

TABLE 26.14 Demand Load of Fixtures

FIXTURE	OCCUPANCY	TYPE OF SUPPLY CONTROL	LOAD VALUES ASSIGNED, WATER SUPPLY FIXTURE UNITS		
			COLD	HOT	TOTAL
Water closet	Public	Flush valve	10		10
Water closet	Public	Flush tank	5		5
Urinal	Public	1" (25.4 mm) flush valve	10		10
Urinal	Public	¾" (19 mm) flush valve	5		5
Urinal	Public	Flush tank	3		3
Lavatory	Public	Faucet	1.5	1.5	2
Bathtub	Public	Faucet	3	3	4
Showerhead	Public	Mixing valve	3	3	4
Service sink	Offices, etc.	Faucet	2.25	2.25	3
Kitchen sink	Hotel, restaurant	Faucet	3	3	4
Drinking fountain	Offices, etc.	⅜" (9.52 mm) valve	0.25		0.25
Water closet	Private	Flush valve	6		6
Water closet	Private	Flush tank	3		3
Lavatory	Private	Faucet	0.75	0.75	1
Bathtub	Private	Faucet	1.5	1.5	2
Shower stall	Private	Mixing valve	1.5	1.5	2
Kitchen sink	Private	Faucet	1.5	1.5	2
Laundry trays (1 to 3)	Private	Faucet	2.25	2.25	3
Combination fixture	Private	Faucet	2.25	2.25	3
Dishwashing machine	Private	Automatic		1	1
Laundry machine [8 lb (3.6 kg)]	Private	Automatic	1.5	1.5	2
Laundry machine [8 lb (3.6 kg)]	Public or general	Automatic	2.25	2.25	3
Laundry machine [16 lb (7.3 kg)]	Public or general	Automatic	3	3	4

Note: For fixtures not listed, loads should be assumed by comparing the fixture with one listed using water in similar quantities and at similar rates. The assigned loads for fixtures with both hot and cold water supplies are given for separate hot and cold water loads and for total load, the separate hot and cold water loads being three-fourths of the total load for the fixture in each case.

From Nielson, L., 1981. *Standard Plumbing Engineerings Design*, 2nd Ed. New York: McGraw-Hill.

Booster pumps may be required when pressures are inadequate at the upper level floors.

**EXAMPLE 2:** Size the water-supply line for a three-story apartment building having five apartment units on each level. Each apartment has one bath-

room and a kitchen with a dishwasher. There are four (16-lb) laundry machines in the basement. The elevation of the 8-in water main located in the street is 165.5 ft. The water meter, where the service line connects to the building, is 160 ft away from the water main and is at elevation 175.5 ft. The service line has

two 90° elbows and one angle valve. Minimum pressure in the water main is 45 psi.

**SOLUTION:**

1. Tabulate the fixture units. Using Table 26.14 to get the total fixture units (see Table 26.15)
2. Using Figure 26.37, the total demand for 151 (WSFU) is 60 gpm.
3. Using Figure 26.38, the demand discharge of 60 gpm and a design velocity of 6 fps indicates that a 2-in. pipe is required. Note that the corresponding pressure drop (i.e., head loss) is 2.8 psi per 100 ft.
4. Convert the elbows and angle valve to equivalent lengths of pipe. From Figure 26.36, the angle valve is 28 ft of 2-in pipe and an elbow is 5 ft of 2-in. pipe.
5. The pressure at the inlet side of the water meter is the minimum pressure in the water main less the losses in the valves and piping and the elevation difference (i.e., static lift) between the water main and the outlet connection, or  
Pressure @ water meter

$$\begin{aligned}
 &= 45 \text{ psi} \\
 &\quad - \left[ \frac{2.8 \text{ psi}}{100 \text{ ft}} (160 + 28 + 2(5)) - (174.5 - 164.5) \right] \\
 &= 29.5 \text{ psi}
 \end{aligned}
 \tag{26.29}$$

**Hardy-Cross**

Analysis of a water-distribution system can be extremely complex due to the varying pipe sizes, pump-storage facilities, fluctuations in demand, and fire flow requirements

**TABLE 26.15 Fixture Unit Count for Example Problem**

FIXTURE	TOTAL UNITS	WSFU	TOTAL WSFU
Kitchen sink	15	2	30
.....			
Dishwasher	15	1	15
.....			
Lavatory (sink)	15	1	15
.....			
Water closet (flush tank)	15	3	45
.....			
Bathtub	15	2	30
.....			
Laundry machine	4	4	16
			Total = 151

that must be considered. Many methods used for analysis are iterative processes that incrementally adjust the flows or head losses until convergence. Several iterative techniques include the linear theory method, the Newton-Raphson method, and the ever popular Hardy-Cross method. One of the most widely used and accepted computer models, KYPIPE, is based on the linear theory method.

The simplicity of the Hardy-Cross method, a form of the Newton-Raphson method, lends itself to hand calculations. Although the personal computer has made laborious hand calculations nearly obsolete, an understanding of even the simplest method helps in assessing the validity of the final values of the analysis. The Hardy-Cross method uses an equation of the form

$$h_f = KQ^n \tag{26.30}$$

where  $h_f$  is the friction head loss in a section of pipe,  $K$  is a constant, and  $Q$  is the discharge.

The value of the exponent  $n$  depends on the governing equation selected for solving the problem. As an example, the Hazen-Williams equation (Equation 26.20) can be written as

$$Q = 1.318C \left(\frac{D}{4}\right)^{0.63} S^{0.54} \left(\frac{\pi D^2}{4}\right) \tag{26.31}$$

For a given pipe of known material and diameter flowing full, the equation simplifies to

$$Q = KS^{0.54} \tag{26.32}$$

where the constant  $K$  incorporates the coefficient 1.318, the diameter, the  $C$ -factor,  $p$ , and other constants.

Substituting for  $S$ , the equation is

$$h_f = KQ^{1.85} \tag{26.33}$$

There are two main underlying assumptions in the Hardy-Cross method. The first is that continuity must be preserved (the total flow into a junction equals the total flow exiting the junction). The second is that pressure at any junction is single-valued (the summation of the head loss around a loop is zero).

With reference to Figure 26.39, if continuity is preserved, the flow at point B is equal to the flow at point C. For the pressure at point C to be single-valued, the head loss in the upper loop must equal the head loss in the lower loop. In all likelihood the pipe lengths and diameters and possibly even the pipe material in each loop are different. The discharge will divide at point B in such a way that the head loss through the two sections of loop is equal. As a result, if a clockwise direction of flow is assumed to be positive, the summation of the head loss from  $BC_{up}$  around  $CB_{low}$  will be zero.

The solution is determined through a series of iterations, with each iteration producing a loop correction factor. Some value of flow for  $Q_{up}$  and  $Q_{low}$  is assumed. For these as-

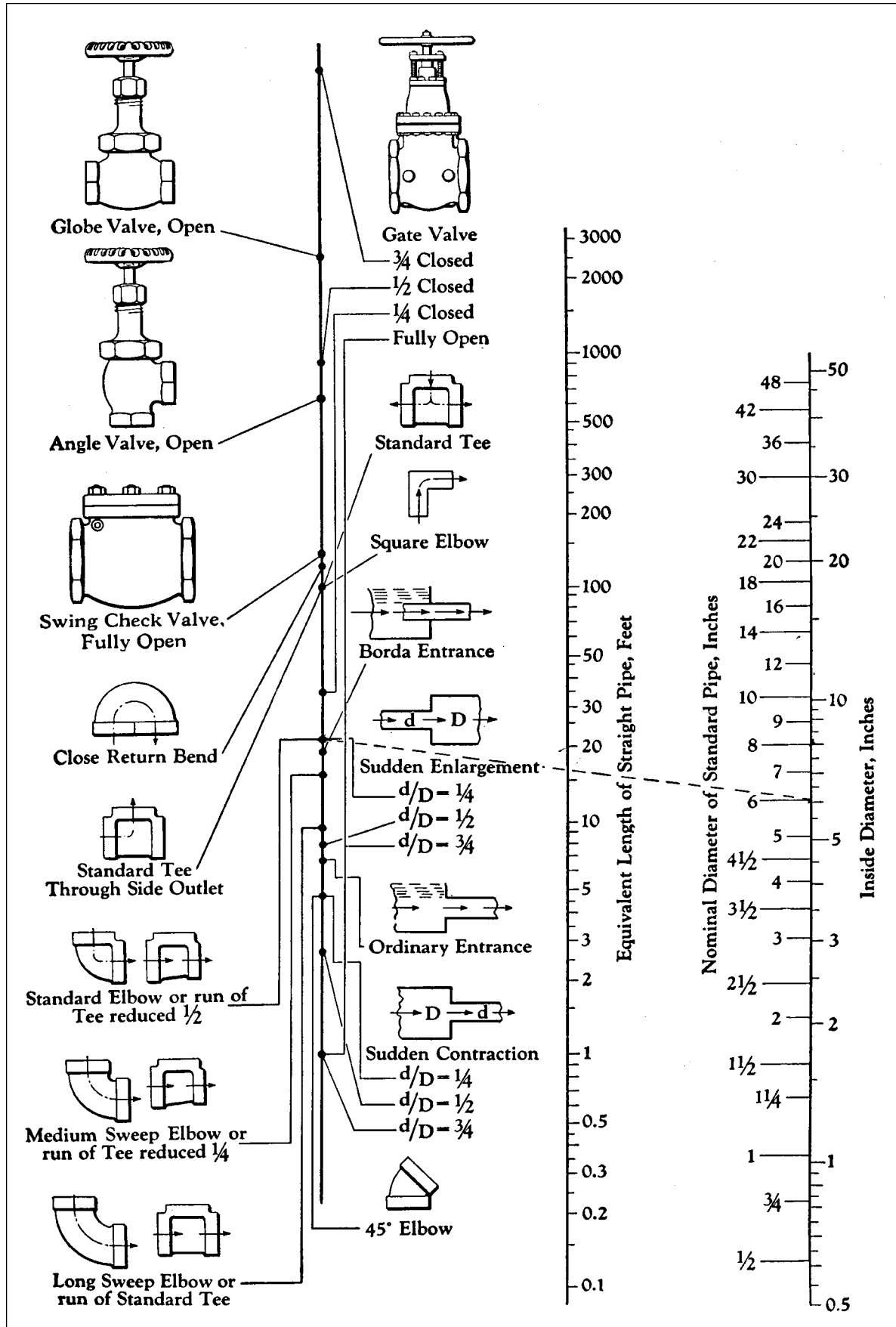


FIGURE 26.36 Nomograph for conversion of fixture losses to equivalent pipe lengths.

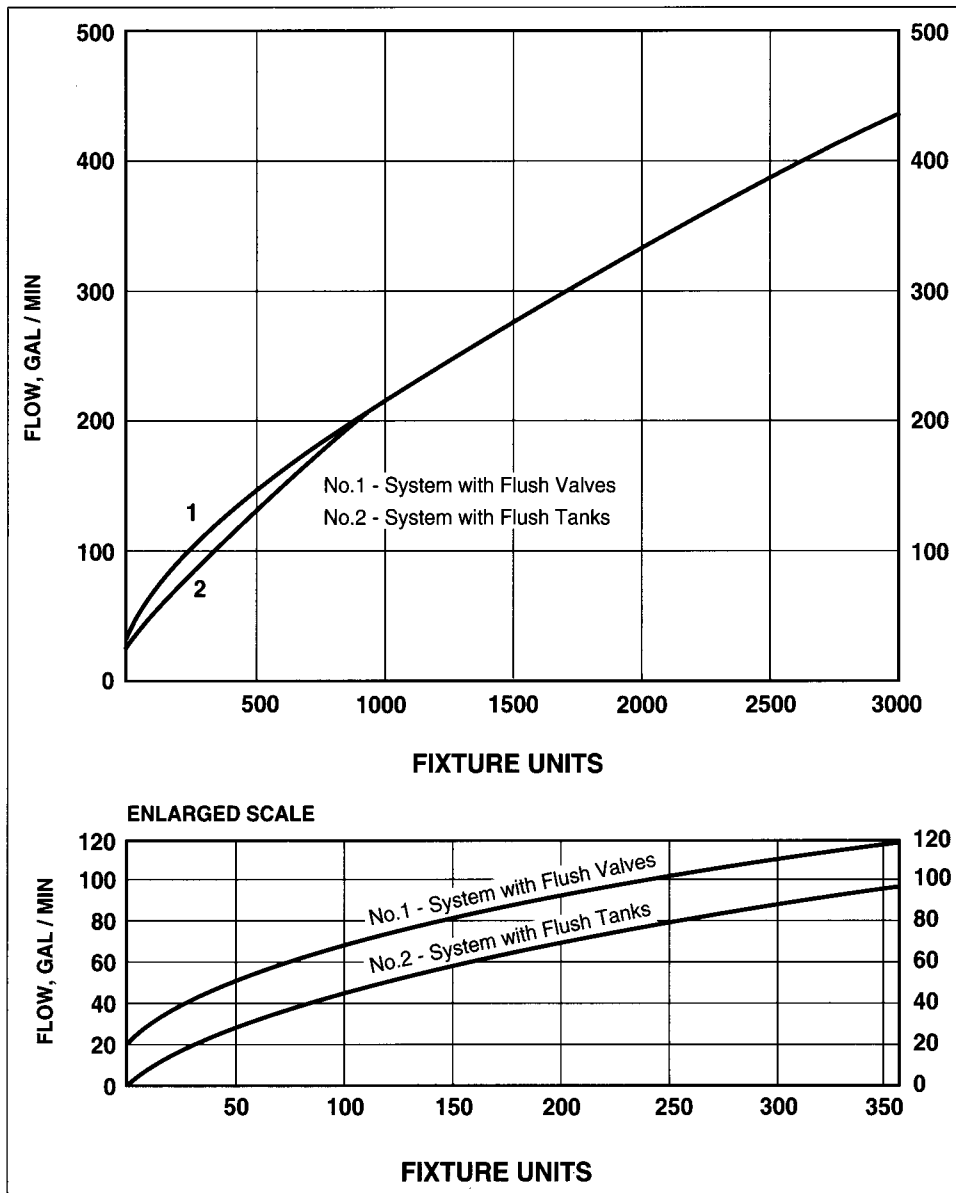


FIGURE 26.37 Curves for estimating demand load.

summed flows, if  $h_{f(BC-up)}$  is greater than  $h_{f(BC-low)}$ , then the discharge in the upper loop must be reduced and the discharge in the lower loop must be increased. That is,  $Q'_{up} = Q_{up} + \Delta Q$  and  $Q'_{low} = Q_{low} - \Delta Q$ , where  $\Delta Q$  is the loop correction factor. Eventually the discharges in each loop are adjusted, such that the summation of the head loss around the loop converges to zero. The flow correction factor for the loop is

$$\Delta Q = \frac{h_{f(BD-up)} - h_{f(BC-low)}}{1.85 \left( \frac{h_{f(BC-up)}}{Q_{up}} + \frac{h_{f(BC-low)}}{Q_{low}} \right)} \quad (26.34)$$

For a more complex multi-loop problem, the initial task is to set up the following parameters:

- Pipe system identifying pipe sizes
- Lengths and materials
- Identify all location of flows entering and leaving the system
- Assume a sign convention for direction of flow.

In the initial assumption of pipe flows, it is imperative for the sum of the flows at any junction to be zero. Using these initial conditions, the head loss for each pipe is computed as

$$\Delta Q = - \left( \sum_{i=1}^n (H_{\bar{d}})_i \right) \times \sum_{i=1}^n \left( \left| \frac{(H_{\bar{d}})_i}{Q_i} \right| \right) \quad (26.35)$$

where  $(H_f)_i$  represents the head loss in the  $i$ -th pipe of the

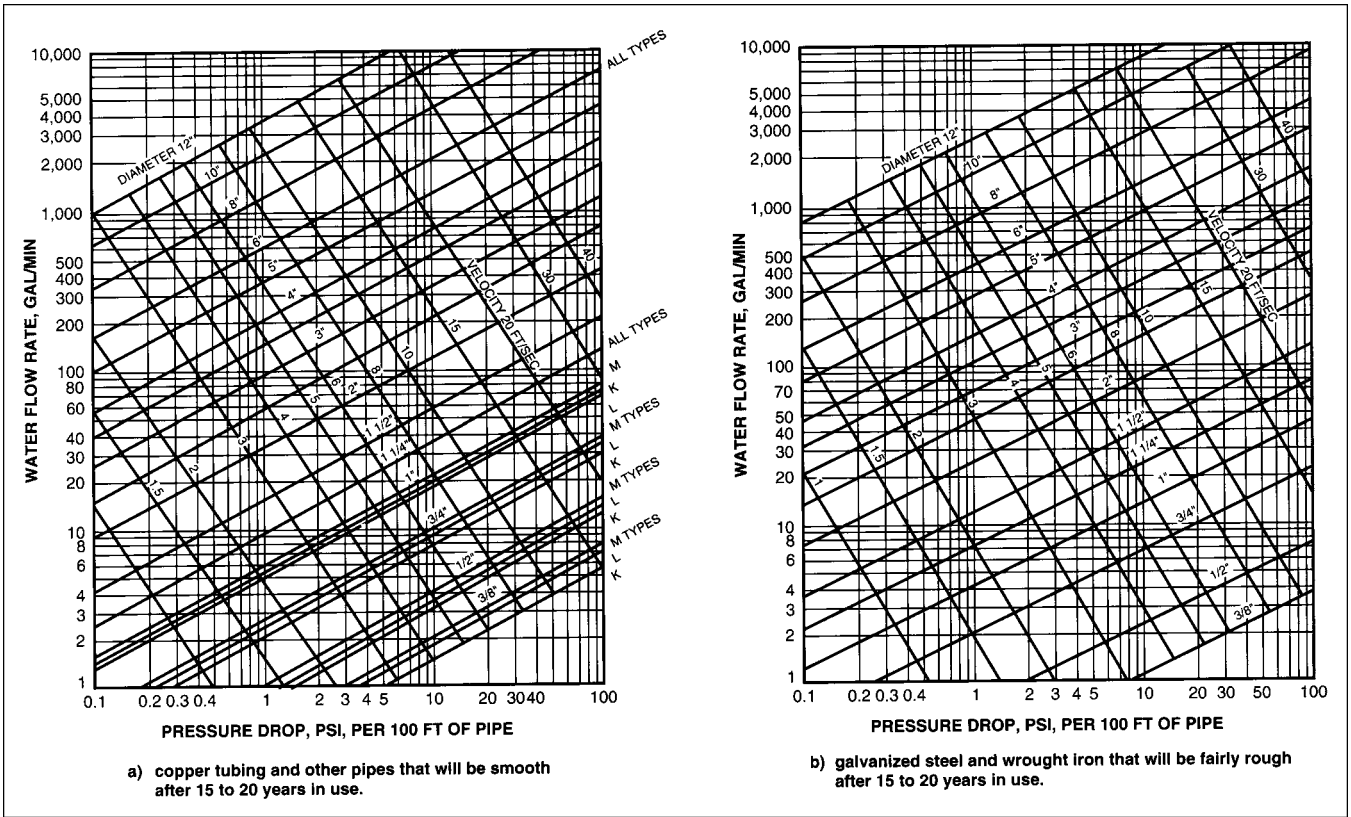


FIGURE 26.38 Chart for determination of flow in pipes (GPM versus velocity versus pressure drop).

loop and  $Q_i$  is the discharge in that pipe, which was used to calculate  $H_f$ .

The sign convention applies to the  $H_f$  term—all flows in the negative direction will have negative  $H_f$  terms.

Figure 26.40 shows a simple three-loop system with pipe information and assumed flows. Table 26.16 summarizes the information shown on Figure 26.40. Note the algebraic sign attached to the discharge: negative if the flow is counterclockwise; positive for clockwise direction. At each junction the  $\Sigma Q = 0$ . In most systems there are valves, pumps, storage tanks, and other components that affect the calculations. Additionally, systems are analyzed for maximum fire flow demands at several locations to ensure adequate pressures everywhere. This simple system serves only to illustrate the fundamental procedure for Hardy-Cross calculations.

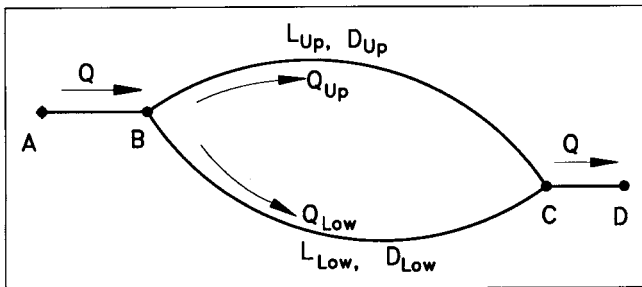


FIGURE 26.39 Simple loop for Hardy-Cross solution.

Tables 26.17 and 26.18 show how loop flows are adjusted by a  $\Delta Q$  correction. Note how the adjustments are determined for pipes common to more than one loop. For example, pipe 4 is common to loops one and two. In loop 1, the first iteration  $\Delta Q$  is +0.082, while the  $\Delta Q$  for loop 2 is 0.113. The new flow in pipe 4 is then  $0.20 + 0.82 - 0.113 = 0.17$  cfs. The initial flow of 0.2 is adjusted si-

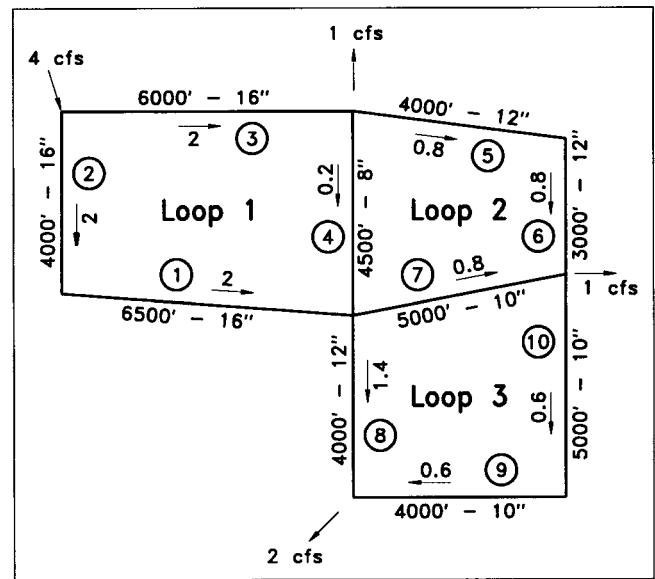


FIGURE 26.40 Schematic diagram for Hardy-Cross example.



**TABLE 26.16 Initial Assumptions for Flows and Directions**

INPUT VALUES				
LOOP No.	PIPE No.	DIAM. (in.)	LENGTH (ft)	Q (cfs)
1	1	16	6500	-2.00
	2	16	4000	-2.00
	3	16	6000	2.00
	4	8	4500	.20
.....				
2	4	8	4500	-.20
	5	12	4000	.80
	6	12	3000	.80
	7	10	5000	-.80
.....				
3	7	10	5000	.80
	8	12	4000	-1.40
	9	10	4000	.60
	10	10	5000	.60

multaneously by the correction of the first loop and the negative of the correction of the second loop. The iterative process continues until the sum of the absolute values of the  $\Delta Q$ s is less than a prescribed minimum tolerance. The acceptable tolerance depends on the precision of the known values and the degree of accuracy required.

**DISTRIBUTION SYSTEM CONSTRUCTION PLANS**

**Preliminary Layout and System Modeling**

Water-line analysis and design are done using computers and commercial software, such as KYPIPE, CYBERNET, WaterCAD and others. To model the system accurately, pipe geometry, system demand, flow, and pressure data for the existing water services, at the points where the proposed water line connects to the existing water line, are needed. Pressure and flows, at these connection points, serve as the boundary conditions for the proposed water line. Additionally, data may be needed on storage tank capacities, water-surface elevations in the storage tank at the time of the flow tests, as well as pumping station and well pump curves.

If the data is not available directly from the utility company, flow tests will have to be conducted at nearby hydrants to obtain static and residual pressures for several different flow conditions. If the municipality or water utility company has the computational abilities and resources, they may dictate the location and size of the water-supply

**TABLE 26.17 First Iteration Values**

LOOP No.	PIPE No.	DIAMETER (in.)	LENGTH (ft)	Q (cfs)	H <sub>L</sub>	1.85*H <sub>L</sub> /Q	Q' (cfs)
1	1	16	6500	-2.00	-5.47	5.06	-1.92
	2	16	4000	-2.00	-3.37	3.11	-1.92
	3	16	6000	-2.00	5.05	4.67	2.08
	4	8	4500	.20	<u>1.56</u>	<u>14.43</u>	.17
					-2.23	27.27	
					$\Delta Q = -(-2.23/27.27) = .082$		
.....							
2	4	8	4500	-.20	-1.56	14.43	-.17
	5	12	4000	.80	2.51	5.80	.91
	6	12	3000	.80	1.88	4.34	.91
	7	10	5000	-.80	<u>-7.61</u>	<u>17.60</u>	-.52
					-4.78	42.17	
					$\Delta Q = -(-4.78/42.17) = 0.113$		
.....							
3	7	10	5000	.80	7.61	17.60	.52
	8	12	4000	-1.40	-7.06	9.33	-1.57
	9	10	4000	.60	3.57	11.00	.43
	10	10	5000	.60	<u>4.47</u>	<u>51.71</u>	.43
					8.59	51.73	
					$\Delta Q = -(8.59/51.73) = -0.166$		

TABLE 26.18 Second Iteration Values

LOOP No.	PIPE No.	DIAMETER (in.)	LENGTH (ft)	Q (cfs)	H <sub>L</sub>	1.85*H <sub>L</sub> /Q	Q' (cfs)
1	1	16	6500	-1.92	-5.07	4.89	-1.85
	2	16	4000	-1.92	-3.12	3.01	-1.85
	3	16	6000	2.08	5.44	4.84	2.15
	4	8	4500	.17	<u>1.13</u>	<u>12.30</u>	.26
					-1.62	25.04	
					$\Delta Q = -(-1.62/25.04) = 0.064$		
2	4	8	4500	-.17	-1.13	12.30	-.26
	5	12	4000	.91	3.20	6.51	.88
	6	12	3000	.91	2.40	4.88	.88
	7	10	5000	-.52	<u>-3.43</u>	<u>12.20</u>	-.57
					-1.04	35.89	
					$\Delta Q = -(1.04/35.89) = 0.029$		
3	7	10	5000	.52	3.43	12.20	.57
	8	12	4000	-1.57	-8.68	10.23	-1.55
	9	10	4000	.43	1.96	8.43	.45
	10	10	5000	.43	<u>2.45</u>	<u>10.54</u>	.45
					-0.84	41.40	
					$\Delta Q = -(-.84/41.40) = -0.20$		

system components. Some larger utilities now use system modeling software that is integrated with a geographic information system (GIS) for the management of their water infrastructure. For small systems, the consulting engineer will normally do the hydraulic analysis and design of the additional distribution system needed for the new development project.

The following criteria are considered when designing the water-supply system:

- In most cases the flows in the water-distribution system are designed to supply the amount of water needed to meet the maximum daily potable demand plus fire flows, or the peak hour flow, whichever is greater (the AWWA defines the maximum daily demand as the maximum amount of water used during one 24-hr period occurring during the latest 3-yr period).
- The minimum residual pressure in the system at the average daily demand should be around 40 to 50 psi. According to the AWWA, the average daily demand is the average amount of water used each day during a 1-yr period for the entire system. If the water pressure in the main is 60 psi or greater, the engineer should consider incorporating pressure-reducing valves on the domestic service lines.

- The minimum residual system pressure during the peak hour demand on the maximum day is typically required to be 30 psi or greater. Additionally, the pressure in the water main should be greater than 20 psi during the maximum daily demand plus fire flow.

The preceding considerations are only guidelines for designing the water distribution system. The regulating authorities should be consulted for specific line pressures, demand flows and other design criteria.

After estimating the demand and determining the peaking factors, layout the pipe network, including fire hydrants, valves, tees, and crosses on a plan view of the project. The layout also should include any stubouts needed to provide water service to future developments. In laying out the waterline, the engineer follows the local criteria for setting the waterline in public right-of-ways based on the number of dwelling units being served, the engineer estimates the pipe size and labels them on the plan view. In most moderately-sized developments, pipe diameters typically range from 6 in. on short cul-de-sacs streets to 12 in. for the main feeder line. Typically, the size requirement results from fire flow demand and not because of the domestic demand. In all likelihood, the size of the distribution lines through the proposed development will have similar sizes as the existing lines in surrounding similar developments.



Input this data into the model and examine the output for high velocities, excessive head losses, and unacceptable pressures. Adjust the pipe sizes to meet demand and local standards criteria. When the model analysis is complete, finish the plan and profile of the waterline for the construction drawings.

### The Water Main Plan and Profile

The plan and profile of the water line identifies the size and type of pipe and the location of the fittings. To the contractor, it shows the material needed, the volume of excavating and potential construction problems. To the review agencies, the plan and profile shows that the waterline is in conformance with the applicable building and health codes.

The water-line location is accurately drawn on the plans as per local criteria. For stakeout and construction purposes, the location of a water line is identified by stationing. Since most waterlines in a development project are located in the street and typically run near parallel with the centerline, the street stationing can be used for stationing the water line. When the course of a water line cannot be tied to the street, an arbitrary stationing system on the water line itself serves the same purpose. Although stationing of the pipe system is done as a matter of convenience and should reasonably reflect the length of the pipe used, it does not need to start with 0+00. In the figures showing the plan and profile of a typical water line, the water-line stationing coincides with the street stationing.

Although some localities allow pipes to be laid with curves, other localities require that pipe direction changes be made with elbow fittings. On commercial sites, the designer needs to know where the fire hose connections at the building are located. In addition, some locales have minimum spacing for fire hydrants in commercial areas. Both of these items affect the placement of the hydrants. In residential areas, placement of fire hydrants may be based on minimum spacing or based on the area of coverage for the required fire flow. The engineer should ensure that the location of the hydrants shown on the worksheet complies with appropriate design criteria. After the water-line location and stationing are established in the plan view, the water-line profile is drawn using the same stationing system. In the profile view, pipe characteristics (e.g., diameter, material, bearing strength, etc.) are labeled. The profile view identifies the station of fitting, hydrants, and other pertinent components. The profiled water line shows how it will weave around other utilities. As discussed previously, special construction techniques may be necessary when the waterline is placed either near a sanitary sewer or at less than minimum depth. The engineer should ensure that all utility crossings are shown in the profile view and that minimum clearance (as per applicable standards) is maintained.

At the point where the proposed water line connects to the existing water main, the plans should identify the method for location of existing line and type of connection

(e.g., wet-tap with tapping sleeve and valve). Although the water utility company probably has records to show where the existing line should be, test holes are dug to find the exact location. In many instances, where the water line is terminated from a prior development, a blow-off valve is set to easily connect a future water line (see Figure 30.41).

### REFERENCES

- American Water Works Association, *Design and Construction of Small Water Systems*, American Water Works Association, Denver, Colo., 1999.
- American Water Works Association, *Water Distribution Operator Training Handbook*, American Water Works Association, Denver, Colo., 1976.
- Ductile Iron Pipe Research Association, *Thrust Restraint Design for Ductile Iron Pipe*, Ductile Iron Pipe Research Association, Birmingham, Al., 1984.
- Engineers Automatic Valve Reference*, Golden Anderson Industries Inc., Pittsburgh, Pa.
- Fire Suppression Rating Schedule*, Insurance Services Office, New York, 1980.
- Grading Schedule for Municipal Fire Protection*, Insurance Service Office, New York, 1974.
- Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers, *Recommended Standards for Water Works*, Health Education Services, Albany, N.Y., 1992.
- Guide For Determination of Required Fire Flow*, Insurance Services Office, New York, 1974.
- Lehr, Jay, et. al., *Design and Construction of Water Wells*, Van Nostrand Reinhold, New York, 1988.
- Merrick, Ronald, C., *Valve Selection and Specification Guide*, Van Nostrand Reinhold, New York, 1991.
- National Association of Home Builders Research Foundation, *Water Distribution Systems Land Development Standards*, National Association of Home Builders Research Foundation, Rockville, Md., 1974.
- National Fire Protection Association, *National Fire Codes Vol. 10; Recommended Practice for Fire Flow Testing and Marking of Hydrants*, National Fire Protection Association, Quincy, Mass., 1988.
- National Plumbing Code, *Building Officials and Construction Administrators*, National Plumbing Code, Country Club Hills, Il., 1990.
- Nielsen, Louis S., P.E., *Standard Plumbing Engineering Design*, 2nd ed., McGraw-Hill, New York, 1982.
- Ponce, Victor Miguel, *Engineering Hydrology*, Prentice Hall, Englewood Cliffs, N.J., 1989.
- U.S. Department of the Interior, USGS, *Estimated Use of Water in the United States in 1985*, Circular 1004, U.S. Government Printing Office, Washington, D.C., 1985.
- U.S. Environmental Protection Agency, *Cross-Connection Control Manual*, U.S. Government Printing Office, Washington, D.C., 1973.
- Viessman, Warren, Jr., and Mark J. Hammer, *Water Supply and Pollution Control*, 4th ed., Harper & Row, Cambridge, Mass., 1985.
- Virginia Department of Health, *Waterworks Regulations, Commonwealth of Virginia*, Bureau of Water Supply, Department of Health, Richmond, Va., 1982.



## CHAPTER 27

# WASTEWATER TREATMENT

*Ernest Jennelle, Ph.D., P.E.*

### ON-SITE SYSTEMS

#### General

The most desirable means of managing the sewage flow from a home or community is through a central system where the sewage is conveyed to a community sewage treatment plant for treatment and disposal. However, many homes will not be located where a central system is available. Also, when the lot size becomes greater than about  $\frac{3}{4}$  acre, the per-unit cost of installing a collecting sewer system may become prohibitive, limiting options for wastewater management to on-site systems. Until very recently, either a cesspool or a septic tank/drain field system were the only satisfactory on-site alternatives available to the design engineer or homeowner. The regulations for on-site drain fields, which have become more rigorous in recent years, now clearly define the types of soils and the depths to the water table that are satisfactory for the design and placement of septic tank drain fields. Thus, local soil conditions may make many locations unsuitable for this method of wastewater management. In instances where the soil is found to be unsuitable or the water table is too high for a septic tank/drain field system, the design engineer has other options available that may make it possible to locate a reliable on-site system on a lot. Design procedures for several alternative systems are presented in this chapter.

The need to install an on-site wastewater management system need not detract from the potential to develop a particular site. An on-site facility may mean a larger lot size and other considerations related to the site plan. However, many homes and businesses have been satisfactorily served by on-site systems for many years. More than 25% of the homes in the U.S. rely on on-site systems and about 0.5

million new homes are being added to this number each year. It has been estimated that about one third of the land area in the United States is suitable for on-site systems employing absorption fields.

Pressure for development often occurs for land areas that are not the most suitable for absorption systems. Land availability, together with higher land costs, means that lot size continues to decrease, placing greater restrictions on available areas for on-site absorption fields. Denser housing, environmental awareness about ground and surface-water pollution and a better understanding of design requirements have caused, and will continue to cause, public health officials to impose additional conditions on the design and placement of absorption fields.

The high failure rate for older on-site systems that were located in unsuitable soil has contributed to a low opinion of the technology by some perspective homeowners. Septic tank failures occur because of poor design, construction and maintenance. A properly designed, constructed, and maintained system offers an affordable, reliable and environmentally sound means of providing on-site wastewater management. The absence of central sewerage need not always reduce the potential of a proposed site for development.

The performance of an on-site system depends on several local geographic and geologic conditions, including the average annual temperature, the depth to ground water, the distance to bedrock, the type and degree of weathering of the bedrock, the type and depth of soil and the annual rainfall. The material presented herein is only a guide to sound design practice. This is a field of rapidly developing technology. The alternatives included herein represent the principal technologies in use. The engineer should have a

broad knowledge of the acceptable alternatives for the local geographic and political area. The "Small Flow Clearing-house" (see Table 27.7) is a source of current data covering new technology.

Regulations for on-site systems are controlled at the state and local government level and may vary significantly between two adjacent political boundaries. A builder or developer should have an engineer conduct an investigation to determine what on-site technology is appropriate for the specific land area before proceeding with plans leading to construction. The selected alternative may affect the development plan in the placement of lots, building locations, or the size of the lots.

### Septic Tank

The septic tank and associated absorption field has been the most widely used on-site wastewater management system in the United States for many years. A septic tank is a buried, water-tight receptacle designed to receive wastewater flow. The tank has no moving parts but is sized and constructed to remove and retain the settleable and floatable material in the incoming flow. The solids removed in the septic tank are mostly organic matter that will consolidate and partly decompose. While the effluent is largely devoid of settleable and floating solids, it is septic, carries a significant amount of suspended and dissolved organics and requires further treatment. Septic tanks for single-family homes are generally purchased from a local precast concrete firm that will deliver and set the tank on a prepared base. The design provided will normally comply with state and local codes. Precast tanks are preferred because they are normally well constructed and watertight. In areas where precast tanks are not available, concrete or masonry tanks are constructed in place. Metal tanks that have been properly coated to retard corrosion are also generally acceptable. Plastic and fiberglass tanks are also available and offer long service life.

**Design.** The design of a septic tank should provide, as a minimum, the following:

- 24-hr flow retention plus maximum sludge and scum storage volume. This size has normally been based on the number of bedrooms to be served. While the normal per person contribution of wastewater flow is about 45 gpd, this is generally adjusted to 75 gpd as a safety factor. It is further assumed in the design process that there are two people per bedroom in a dwelling. Thus, the daily design flow for a two-bedroom dwelling is 300 gpd. When allowance is made for sludge and scum storage, a 750-gallon tank is needed for a two-bedroom home. The Uniform Plumbing Code recommends the tank volumes shown in Table 27.1. The recommended volumes shown in Table 27.1 are equal to or greater than required under the regulations of the Federal Housing Authority and the U.S. Public Health Service. Larger tanks may be needed for

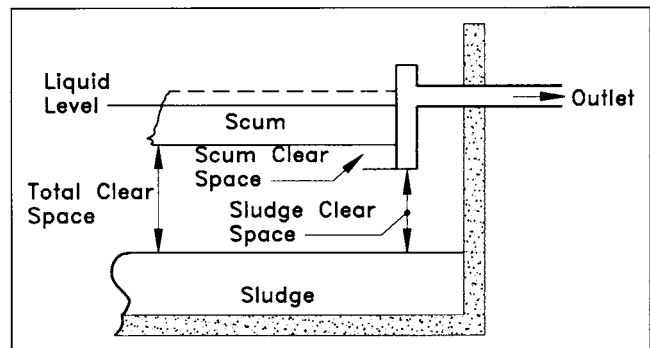
**TABLE 27.1** Septic Tank Volumes

NUMBER OF BEDROOMS	VOLUME (gal)
1-2	750-1000
3	750-1000
4	1000-2000
5	1200-2000
Each additional bedroom	150-250

industrial or commercial flows or for serving a cluster of homes. The volume of the septic tank for this type service should be at least one and a half times the average daily flow. A utility or an equivalent organization should maintain tanks serving more than one home.

- The inlet and outlet should be designed to still the incoming flow velocity, trap the scum layer and minimize scour of the accumulated sludge into the effluent. Some suppliers of precast tanks may provide tanks having baffles placed to the proper depth across the tank at both the inlet and outlet ends of the tank. Most tanks are supplied without baffles with the installing plumber using cast iron or PVC tees to provide the required venting of gases and trapping of solids and scum. Figure 27.1 shows the design of a typical tank outlet.

- The outlet tee is normally 4 in. in diameter, but the vertical stem of the tee may be increased to 6 in. to reduce effluent rising velocity and thus reduce the potential for solids to be scoured into the effluent. The tee should project 12% of the liquid depth above and 24% of the liquid depth below the liquid level. Some local jurisdictions recommend that these projections be increased 50% for tanks serving homes having garbage grinders. It is also recommended that the septic tank volume be increased 250 gallons where a garbage grinder is installed on the system. The normal liquid



**FIGURE 27.1** Septic tank outlet design. (U.S. EPA 1979)

depth in a tank is about 4 ft, with a 1-ft clear space above the liquid.

Septic tank systems serving sources other than domestic homes may require some type of pretreatment of the flow ahead of the tank. Flow from a restaurant should always pass through a grease trap ahead of the tank. Flow from some industrial establishments may require grit removal and/or screening ahead of the tank.

- Sufficient sludge storage. The accumulation of sludge depends primarily on temperature. If the influent to the tank has been screened to eliminate inert material, the organic solids should decompose into water and gases. In warm climates, where the tank contents are warm throughout the year, considerable decomposition will occur, with a corresponding decrease in solids accumulation. However, very little decomposition will occur at locations where the temperature of the tank's contents drops during the winter months. Normally, a septic tank should be checked every 2 years and should be cleaned when the sludge accumulation reaches to within 8 in. of the bottom of the outlet pipe. After accumulating a few years of record, the frequency of checking can be adjusted to the specific need. It is important to clean the tank before the sludge level reaches the outlet pipe entrance to prevent washout of solids from the tank. A manhole is located at both ends of the tank for inspection and cleaning.

- Venting. The tank should be constructed to permit venting back through the influent piping and house plumbing to the roof vent.

### Placement Considerations

- Locate tank in an area where vehicles will not cross or park on top of the tank.
- The tank should be placed such that the top is about 6 in. below grade to discourage entry by unauthorized persons and yet not be so deep as to present a problem at inspection and cleaning time.
- The tank should be located to allow gravity flow from the source into the tank if possible. At locations where this is not possible, a duplex grinder pump is used to lift the flow from the building sewer to the septic tank.
- Trees should not be planted near the tank because of the potential for root damage. Fast growing trees such as elms and willow have the greatest root mass.

### Operation and Maintenance Requirements

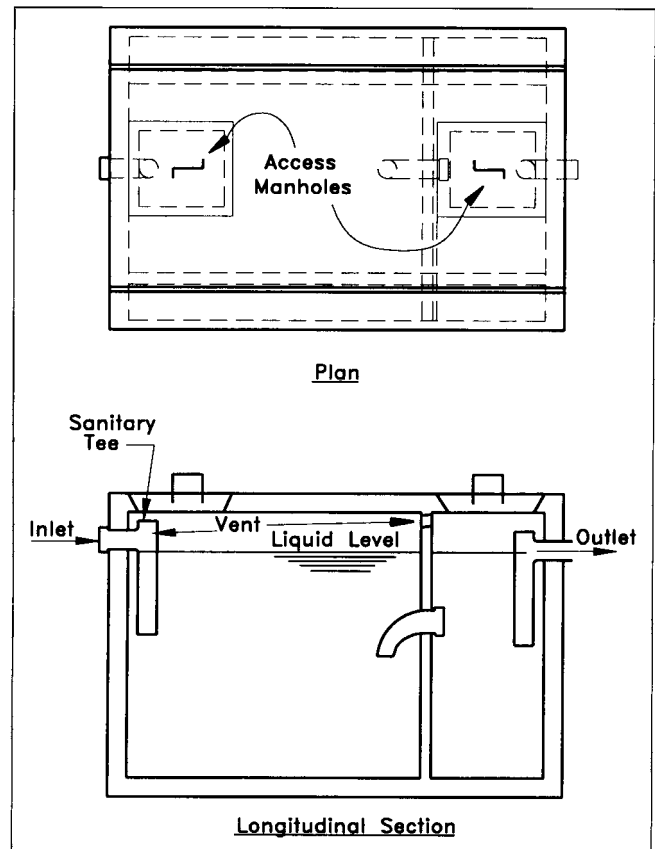
- A properly installed and maintained septic tank should last for many years.

- Have a qualified septage-hauling firm check the tank for solids level every 2 years until evidence shows that a less frequent schedule is satisfactory. Check with the local health official for a list of qualified firms. Qualified firms are licensed by most localities.

- If possible, pump the tank in late spring to allow stabilization to start during the warmer summer months. Some officials recommend that a small amount of solids be left in the tank to seed and buffer the new sludge. The tank should not be washed when it is cleaned. All that is needed is to pump the contents.

- There is no evidence that adding special seed or concoctions will help the performance of a septic tank. However, some of the grease-decomposing cultures have been shown to be effective in other locations in sewer systems and may be effective in reducing a problem with grease buildup in septic tank scum. The use of additives is not a substitute to the recommended periodic inspections.

Figure 27.2 shows a typical two-compartment tank in both plan and elevation. Most tanks have a single com-



**FIGURE 27.2** Typical two-compartment septic tanks. (U.S. EPA 1979)



partment. The single and two compartment tanks are alike except for the intermediate baffle wall.

### Disposal Methods

**General.** The septic tank, if properly designed and maintained, removes the settleable and floatable material from the wastewater flow. Much of the material initially removed by settling from the flow and the non-settleable organics are carried from the tank in the effluent as either dissolved or finely divided solids. The tank effluent is septic, high in solids and reduced chemical forms such as ammonia and sulfides. The biochemical oxygen demand of the effluent will vary greatly in strength diurnally, weekly and seasonally. An average value of the strength is a biochemical oxygen demand of approximately 150 mg/l. The flow from a septic tank will always require further treatment as a part of the disposal process.

Suitable technology is available for safely treating and disposing of septic tank effluent on land, beneath the land surface, into surface waters, and into the atmosphere. The most common methods of treatment and disposal for septic tank effluents, such as from single-family homes, small clusters of homes, or flows from industrial or commercial establishments are subsurface soil absorption systems, evaporation systems, intermittent dosed slow sand filters, and aerobic treatment with or without filtration. Effluents from filtration and aerobic systems are discharged to surface waters. The selection of the most suitable method for a particular application will usually be straightforward.

Subsurface soil absorption systems are the most widely used and represent the most economical and reliable method if soil conditions are suitable. The individual homeowner is not prepared to maintain a system that requires daily or even monthly attention. The soil absorption system, if properly designed and protected from major hazards such as tree roots and heavy surface loads such as vehicle traffic, should operate for many years with little maintenance required. Any wastewater treatment system that requires special maintenance and land use will detract from the value of the land as a building site and will make the completed dwelling less marketable. The following sections present design information for the most common types of on-site treatment and disposal methods.

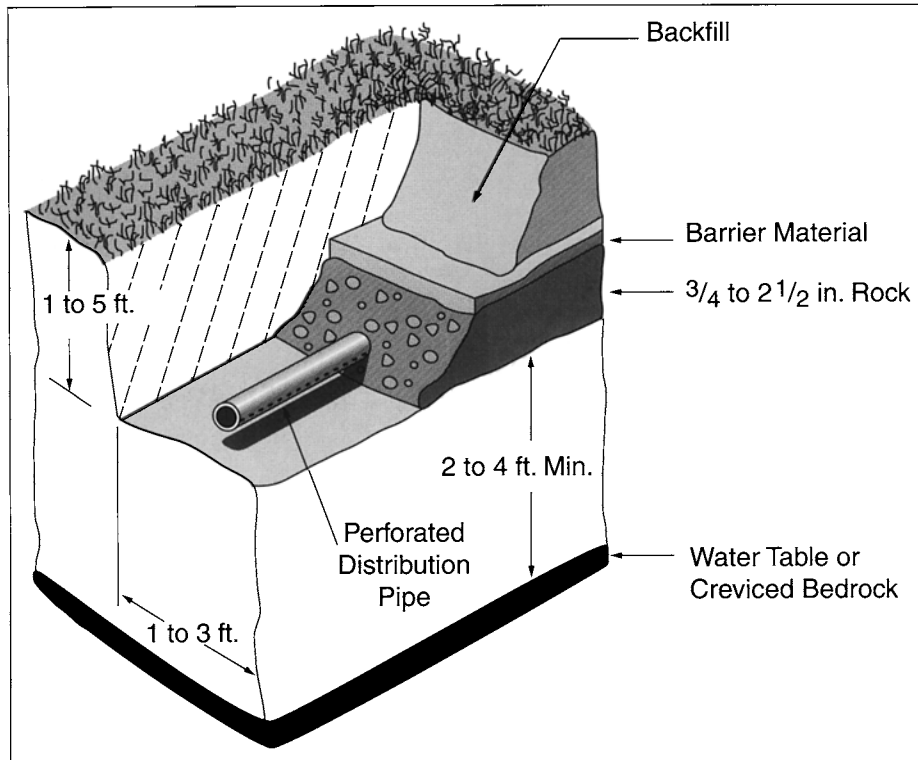
**Subsurface Soil Absorption.** As noted above, where soil conditions are suitable, subsurface soil absorption is the desired method of septic tank effluent treatment and disposal. This method is simple, reliable, requires little attention and has the least cost of installation. This method involves distributing the flow through a subsurface system where it is absorbed and treated as it percolates through the soil. It is important that the flow be applied at an intermittent dosing rate such that the soil drains to an unsaturated condition between applications. The normal rate of water use in the home, such as the flushing of a lavatory or shower, and with little water draining to the septic tank between such uses in the home, automatically gives the intermittent dos-

ing. Larger systems may incorporate pumps or dosing devices to insure intermittent application. System efficiency is improved by incorporation of a holding tank and pump into the system to provide for filling the absorption field distribution piping with each pump cycle. This may mean that the absorption field is dosed less than once per 24-hr period. However, special dosing facilities are not normally provided except where they are needed because of poor soil conditions.

Figure 27.3 shows a section of a typical absorption trench. Care should be exercised during the digging of the trench to insure that the bottom is maintained on a constant grade with no depressions, as this will cause septic tank effluent to accumulate at the low points. It is important that the flow be distributed as evenly as possible along the entire length of the trench. The trench bottom should be sloped about 3 in. per 100 ft. The bottom and sides of the trench should be scarified with a rake or similar tool to break the smooth compacted surface formed by the excavating bucket. The trench is then filled with about 6 in. of clean durable gravel. A 4-in. perforated plastic pipe is placed on the gravel surface after the gravel has been raked to the same uniform slope as the trench. This slope (0.25%) of the pipe should provide good distribution of incoming flow, but not be so steep that all the flow will run to the low end before leaving the pipe. The pipe is then covered with additional clean gravel. A plastic sheet is placed on top of the gravel to prevent soil from infiltrating into the gravel voids and the remainder of the trench is then backfilled with native soil. The trenches are normally less than 5 ft deep, but should be deep enough to provide at least one foot of soil cover over the gravel.

When septic tank effluent is distributed to the soil absorption trench, a biological film develops in the rock fill and on the trench wall. The biological film on the soil trench interface becomes semi-clogging. It is important that the surface area of the gravel be sufficient such that this film does not become so massive as to block percolation into the soil. If the percolation through this film or mat becomes less than the application rate to the trench, the system will fail. Some state and local authorities may require that two absorption fields be provided and that a distribution box be installed to permit seasonal or annual rotation between the two fields. This technique provides a rest period for the system, allowing the growth on the stone and the soil interface to decompose, returning the trench to essentially a new condition.

Soil acts as a filter, exchanger, an absorber, and provides a surface area for biological growth. The interface mat and retention time in the soil provides for entrapment and die-off of pathogens. Ammonia nitrogen and phosphorus are adsorbed onto the soil particles. Organic matter is adsorbed and decomposed by the biological growth in the trench, and this, together with the filtering action of the mat and the soil between the trench and the ground water table, produces a treated effluent that does not pollute the ground



**FIGURE 27.3** Typical soil-absorption trench. (U.S. EPA 1979)

water. A minimum of 2 ft and preferably 4 ft of soil should exist between the trench bottom and the water table. It is important that the soil strata between the trench bottom and the water table not be saturated, as this will reduce adsorption and cause channeling, resulting in poor treatment. Very porous soil such as coarse sand will not give proper treatment, as the flow will move rapidly to the water table without receiving proper treatment.

Soil is primarily composed of mineral matter, resulting from the weathering or decomposition of rock. Residual soils are those found directly above the bedrock source. Many soil deposits have been transported from distant locations, primarily by water (alluvial deposits). Soil may contain refractory, or relative stable organic matter called humus, which changes the physical and chemical characteristics of the soil. Soil containing this mixture of refractory and stable organic matter is called topsoil. It is normal to find a wide variation in soil types at any location. Most open cuts will show more than one soil type. The soil profile in a highway cut may show several soil types or horizons.

The type and condition of the soil, percolation rate and depth to bedrock or water table are important parameters in the location and design of an absorption field. If the soil percolation rate is inadequate, the trench will become saturated and the system will fail. If the flow does not percolate through a sufficient depth of unsaturated soil, adequate treatment will not be provided, resulting in groundwater pollution.

Significant information is available on soils for most locations. The U.S. Department of Agriculture, Soil Conservation Service (SCS) Soil Survey Maps are thought to be available for all areas of the country. These maps are usually compiled and issued for a local governmental jurisdiction such as a county. They are also available from the local SCS office. Furthermore, the Agriculture Extension Service maintains local and state offices and is a local source of soil information. Soil survey maps include aerial photographs, soil classification, interpretative data on potential uses and a description of each soil series. These maps are compiled at small scale, and while very useful, a site-specific soil survey will likely be needed to complete an evaluation of a site.

An experienced individual is required to conduct a field soil survey and prepare a report. The first step in conducting a site-specific soil survey is an investigation on the operation of other absorption fields in the area. This information is normally available from the local agency having permitting responsibility for septic tank systems. This agency can also provide information on soils in the area and their general suitability for soil absorption fields. The next step is the collection of field soil types and conditions. Field soil data can best be obtained by using a backhoe to dig a short length of trench to sufficient depth to expose the soil profile 4 to 6 ft below the proposed elevation of the absorption trench. The exposed soil profile allows the investigator to determine the soil texture (the relative fractions of the various sizes of soil particles), soil structure (the

aggregation of soil particles into clusters), soil color (a good indication of drainage characteristics—standard color charts are available), soil mottling (also described by the color, streaks or areas of darker color), soil profile, and the presence of seasonally saturated soils. Seasonal saturation can be identified during the wet season or generally by the color or mottling of the soil. Additional information on mottling can be found within the Soils chapter of this text.

If the soil survey shows an area suitable for a soil absorption treatment and disposal system, a percolation test is conducted. While a percolation test is subject to wide variability and often poor reproducibility, it remains the principal means of determining the hydraulic conductivity for an area and thus the basis of the absorption field design.

A percolation test is conducted by boring three or more test holes, 6 in. in diameter, to the depth of the proposed absorption trench. The borings are located so as to cover the entire area of the proposed absorption field. If the soil profile varies significantly in the test holes, additional holes may be needed. Figure 27.4 shows a typical percolation test setup. After the hole is bored, the sides are scarified to eliminate the smeared compacted surface left by the auger. Two inches of gravel is placed in the bottom of the hole to prevent scour and the hole is then filled with 12 in. of clean water. This water depth is then maintained for at least 4 hr to soak the soil around the hole. The water level is then adjusted to a 6-in. depth over the gravel and the rate of fall in the surface is observed at a suitable interval, usually 30 min, but this will depend on the percolation rate of the soil. Water is added, as needed, to maintain the 6-in. depth, and measurements of the surface fall rate are recorded until the rate of fall becomes constant for at least three successive observations. The percolation rate is described as the time in minutes required for the water surface to fall 1 in.

The length of required absorption trench is established by the percolation rate, with the application rate being based on the area of trench bottom. Table 27.2 gives some recommended application rates for the indicated ranges of percolation rate. The maximum length of a trench is usually 100 ft because of the inability to distribute the flow over greater lengths. A minimum of two trenches and 200 ft of trench length should be provided for a system. For example, a four-bedroom house is assumed for design purposes to produce 150 gpd/bedroom or a total flow of 600 gpd. If the percolation rate is 45, the application rate to be used in the design of the absorption field is found from Table 27.2 to be 0.45 gpd/ft<sup>2</sup>. Thus, the absorption trench area would be  $600 \text{ gpd} / 0.45 \text{ gpd/ft}^2 = 1333 \text{ ft}^2$ . This is equal to 667 linear ft of a 2-ft wide trench. Eight absorption trenches, each 2 ft wide and 83 ft long, are selected for the design. Trenches are usually spaced 6 ft apart or more, center to center.

Regulations also limit placement of septic tank-soil absorption fields with respect to buildings, wells and other activities. Safe distances vary from site to site, but generally,

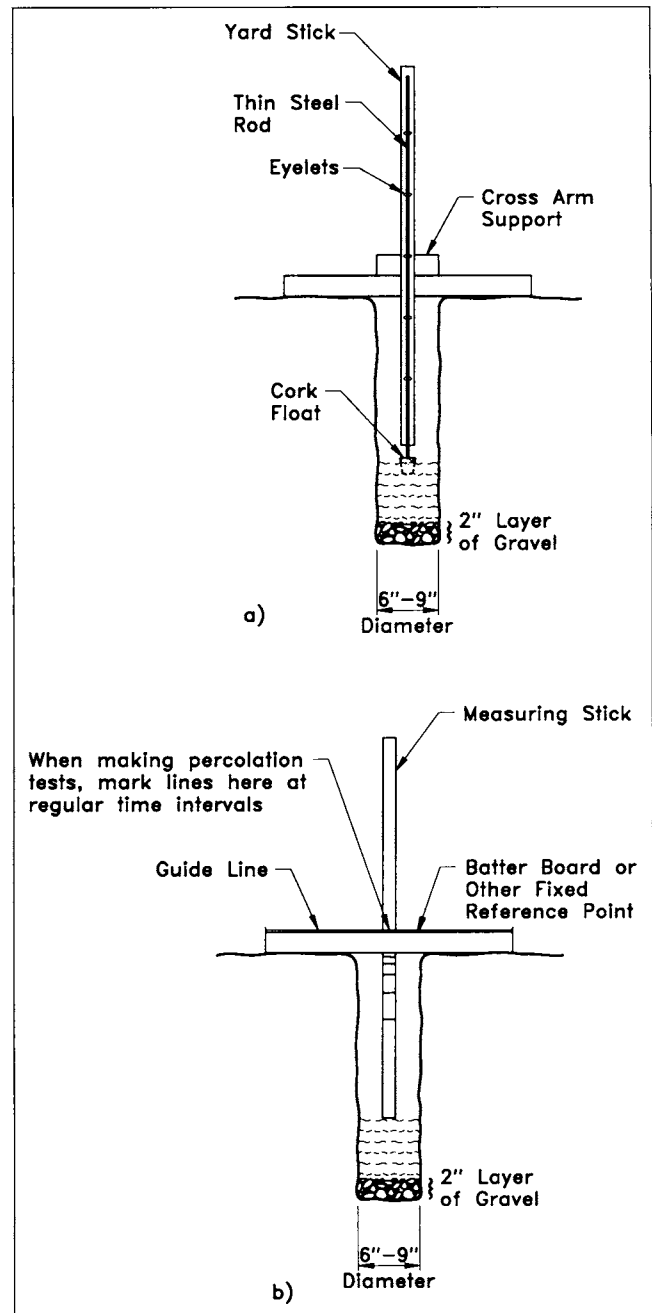


FIGURE 27.4 Typical percolation test setup. (U.S. EPA 1979)

they should not be located within 50 ft of wells and surface waters or within 10 ft of building foundations and property boundaries. Absorption fields are located in well-drained areas, with crest of slopes or convex slopes being desirable. Concave slopes are undesirable because they are usually drainage ways and subject to surface water percolation.

Areas having percolation rates less than 1 min/in. can be used by replacing 4 ft of soil below the trench with suitable soil if permitted by local codes. A 4-ft depth of proper soil below the trench will ensure protection of the ground water below. The flow path for a septic tank—soil

TABLE 27.2 Rates of Septic Tank Effluent Application

SOIL TEXTURE	OBSERVED PERCOLATION RATE	APPLICATION RATE*
Gravel, coarse sand	less than 1	not suitable
Coarse to medium sand	1 to 5	1.2 gpd/ft <sup>2</sup>
Fine sand, loamy sand	6 to 15	0.8
Sandy loam, loam	16 to 30	0.6
Loam, porous silt loam	31 to 60	0.45
Silty clay loam, clay loam	61 to 120	0.2

\*As with all criteria for on-site systems, local requirements may be different. The engineer must always be guided by the regulations of the local permitting agency. Special requirements may apply to areas having a percolation rate greater than 60 minutes per inch. (U.S. EPA 1979)

absorption field may have the components as shown by Figure 27.5. It is not uncommon for a pumping station as shown to be required as part of the system. Normal open-impeller centrifugal pumps can be used with septic tank effluent, but a grinder pump is required if pumping is needed prior to the septic tank. Small grinder pumping stations are available for this purpose.

When pumping is not required, the septic tank effluent flows directly to the distribution box. The distribution box is designed to distribute the flow equally to all the absorption lines. Systems sometimes have a distribution box to distribute flow equally to two or more additional boxes, which in turn supply the absorption lines. Figure 27.5 shows a field selection valve that is used to direct flow to the proper field. Where field alternation is required, the valve is included as a means of directing the flow to the proper field. When used, the fields are usually alternated annually, but some agencies may require alternation that is more frequent. A single distribution box can be used to alternate specific trenches. For example, the eight trenches served by a single box in Figure 27.5 could be divided at the box into two four-trench systems by properly placing a slide gate in the box to permit the homeowner to direct the flow as required. Each of the four trench systems would be designed to handle the total flow.

Water-tight pipe is installed from the distribution box to the beginning of the absorption trench. The absorption trenches may be curved to follow the surface contour of the land. The absorption trenches served by a single distribution box are all the same length so that flow application is uniform. The designer has unlimited options in laying out the absorption field to accommodate soil conditions and topography.

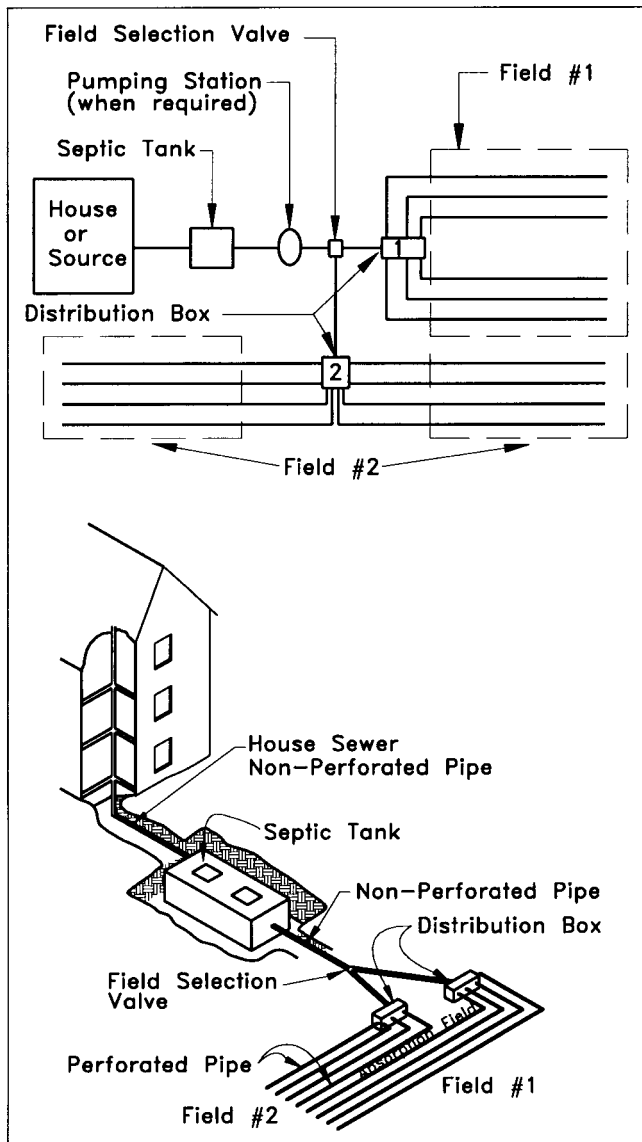
**Mound Systems.** The mound system is an engineered modification of the conventional absorption field to overcome problems with slowly permeable soils and insufficient depths of unsaturated soil. While the technology behind

mound systems has been known for several decades, the practice, while not widespread, is now finding uses with a variety of soil conditions. A mound system is an absorption system that has been constructed above the natural soil elevation. A mound system is constructed above or partially above grade by importing fill that has suitable characteristics. Note that an absorption bed and not trenches is employed in the mound system. No other general layout changes are required in a mound soil absorption field system.

A mound system can be designed to overcome some site conditions that would normally make a site unsuitable for an absorption field. These conditions include soils having a high percolation rate, shallow depths of suitably permeable soil over creviced or porous bedrock and permeable soils with a shallow depth to the water table. When the bottom of the absorption trench bottom is sited over 4 ft of suitable soil material, proper treatment of the wastewater will be provided. The soil below this level will be less prone to clog because only well-treated water will pass from the imported soil, making areas with soils having low permeabilities more suitable. In addition, the mound system provides proper treatment for areas having a shallow depth to the water table or to porous bedrock, eliminating the potential for ground water pollution.

Four views related to a typical mound system are shown in Figure 27.6. Note that the absorption bed is constructed similar to a conventional bed except for being in an artificial fill. Design criteria for a mound system are as follows:

- Land topography—locate in well-drained areas.
- Separation from wells, etc.—same as for a conventional absorption field.
- Natural soil profile—prefer relatively undisturbed area, avoiding filled areas until a thorough study has shown that filled changes will not affect water movement.



**FIGURE 27.5** Typical dual-field septic tank—soil-absorption field layout.

- Unsaturated soil depth—a minimum of 2 ft of natural unsaturated soil should exist over porous or creviced rock or a high water table.
- Depth to impermeable soils or rock barrier—3 to 5 ft. A sufficient depth of permeable soil must exist over impermeable strata to carry the percolating water away from the mound and to prevent the treated effluent from seeping to the surrounding natural ground surface. The required depth depends on soil permeability, climate, and topographic site conditions. Frost penetration depth must be considered. Each site must be evaluated.
- Percolation rate—natural soil should have a percolation rate of not more than 120 min/in. The rate is measured at a depth of 20 in. or 4 in. above the water

table depth. Where shallow soils exist over pervious or creviced bedrock, the test is percolation run at a 12-in. depth.

Mound dimensions as designated in Figure 27.6 are as follows:

Fill depth "D" & "E"	1 ft or more
Absorption bed depth "F"	0.75 ft or more
Cap at edge of bed "G"	1 ft
Cap at center of bed "H"	1.5 ft

Since mound systems are installed in a raised area, the percolation field is dosed by a pump following the septic tank. The use of a pumping arrangement where the pump wet well capacity is 25% of the design daily flow will provide four bed dosings per day. The pressure loading insures that the flow is spread to the entire bed with each dosing.

Table 27.3 provides data required for sizing the absorption bed. The bed area depends on the permeability of the fill material. The selection of fill material may depend on transportation costs. The most suitable material may not be found locally, in which case a material requiring a greater area may be used.

Basal area requirements ((B times A) + I in Figure 27.6) is the area to be provided to insure seepage into the natural ground before reaching the bottom of the mound area. Dimensions "J" and "K" are based on a 3 to 1 side slope as shown in Figure 27.6. "B" and "A" are the absorption bed dimensions. Dimension "I" is the down slope distance required to provide the basal area. Basal area requirements determined by the infiltration rates are given in Table 27.4.

**EXAMPLE** Determine the absorption bed area and mound basal area to serve a three-bedroom house. Medium sand is to be used for fill material and the natural soil is a silty clay loam having a percolation rate of 50 min/in.

Absorption bed area requirements from Table 27.3, infiltration rate = 1.2 gpd/ft<sup>2</sup>.

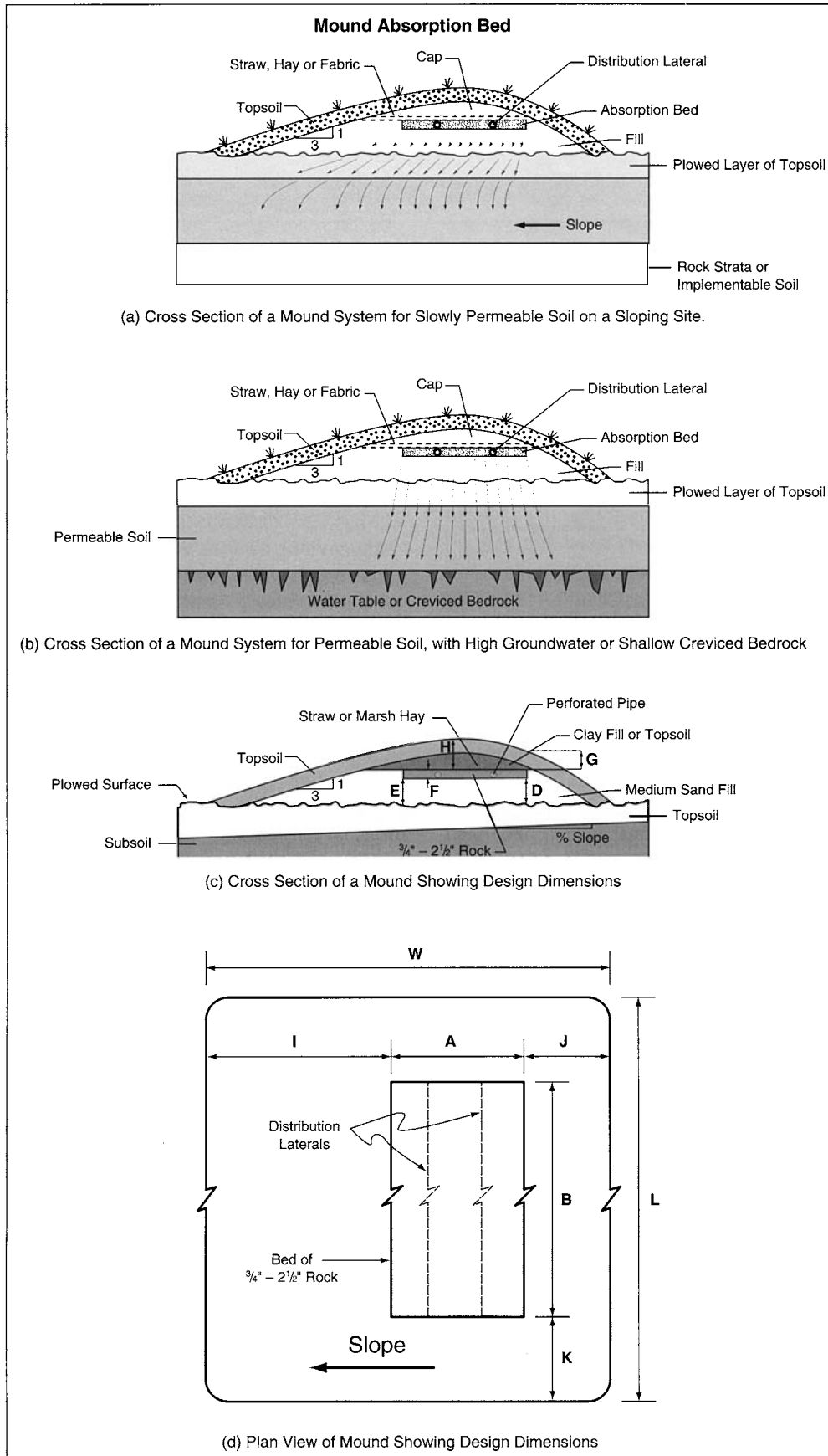
Flow = 450 gpd, Bed area = 450 gpd/ 1.2 gpd per ft<sup>2</sup> = 375 ft<sup>2</sup>.

From available area for mound, a bed length of 50 ft is selected; therefore, bed width is 7.5 ft.

Basal area requirement determined from infiltration rate found in Table 27.4.

Infiltration rate = 0.5 gpd/ft<sup>2</sup>. Bed length dimension "B" is 50 ft.

Basal area = 450 gpd/0.5 gpd per ft<sup>2</sup> = 900 ft<sup>2</sup>. Therefore, Dimensions "I" + "A" = 18 ft. Dimensions "L" and "W" are then determined from the topography and depth of the mound.



**FIGURE 27.6** Mound absorption bed. (U.S. EPA 1979)

TABLE 27.3 Absorption Bed Area Requirements

FILL MATERIAL	CHARACTERISTICS*	DESIGN INFILTRATION RATE
Medium sand	25% 0.25–2.0 mm <30–35% 0.05–0.25 mm <5–10% 0.002–0.05 mm	0.2 gpd/ft <sup>2</sup>
Sandy loam	5–15% clay content	0.6
Sand/sandy loam mix	88–93% sand 7–12% finer-grained material	0.2
Bottom ash		1.2

\*Percent by weight  
(U.S. EPA 1979).

**Evapotranspiration (ET) and Evapotranspiration/Absorption (ETA) Beds.** ET and ETA beds are used as means of liquid disposal through evaporation and transpiration at some locations where subsurface absorption or discharge to a surface discharge is not feasible. ETA beds may also be designed to augment subsurface absorption at locations where the percolation rate is too low for absorption to be effective alone. The system functions best with a high-quality influent, such as that produced by an aerobic process or by an intermittent slow sand filter. However, these systems can be designed to function where the influent is from a septic tank.

ET beds are designed such that capillary action will bring the treated liquid to the surface where it is removed by evaporation and/or transpiration. Planting trees and bushes having a large silhouette can improve the appearance of the bed area and aid in water removal. Transpiration is moisture uptake by the vegetative roots followed by evaporation to the atmosphere through transpiration. However, transpiration is only effective during the growing season, and a vegetative cover reduces evaporation throughout the year.

If evaporation is to be effective, three conditions must be present. First, there must be a continuous supply of heat to meet the latent heat requirement for evaporation (approximately 540 cal/gr of water evaporated at 15 degrees C). Second, a vapor-pressure gradient must exist between the evaporative surface and the atmosphere to remove va-

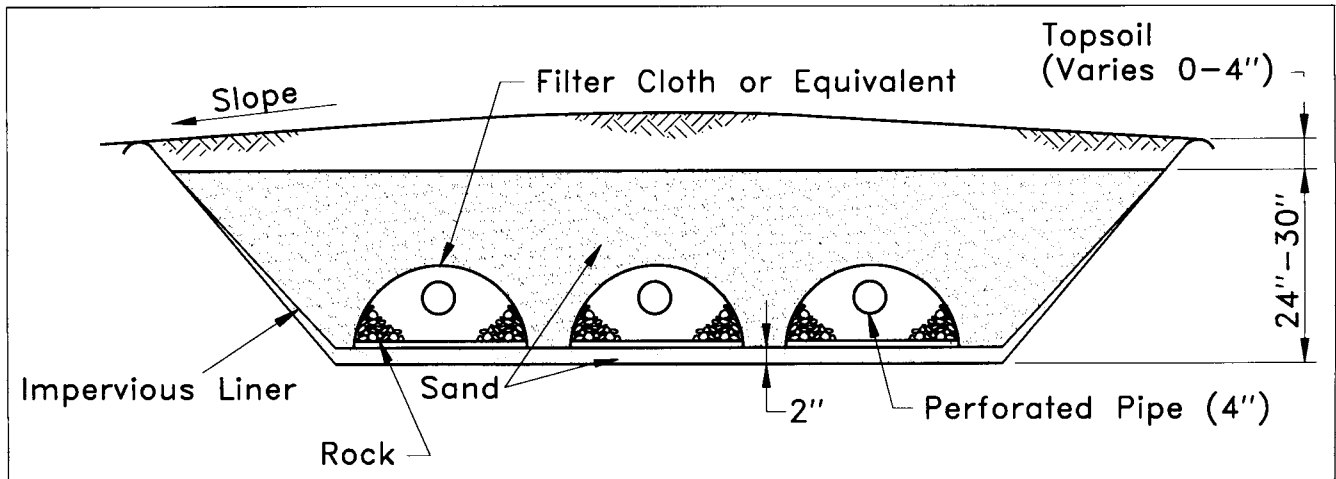
por by diffusion and convection. Meteorological factors, such as air temperature, humidity, wind velocity and radiation influence both energy supply and vapor removal. Both evaporation and transpiration may be reduced to near zero in the winter months. Winter rainfall generally makes ET negative during this period. Third, the moisture must be brought to the soil surface. The soil material must be fine-textured for capillary transport to be effective, but not so fine that water movement is restricted. These requirements for effective evaporation limit its applicability to a majority of sites. The dissolved salts from the water supply are deposited at the surface as evaporation occurs and may influence the long-term operation of the ET bed. Excessive growth within the bed, resulting from the organic loading contained in the wastewater, may further restrict water movement.

All rainfall seeps into the bed, which adds to the evaporative requirements for the process. ET bed design must provide for the maximum annual rainfall expected at least every 10 years. A moisture balance (water budget) should be carried through several years of a projected wet rainfall cycle.

A typical ET bed is shown in Figure 27.7. The bed consists of 1.5 to 3 ft of selected sand over an impermeable plastic liner. Uniform sand having an average size of 0.10 mm has been found to be capable of raising water about 3 ft through capillary action. A perforated plastic piping sys-

TABLE 27.4 Infiltration Rates for Determining Mound Basal Area

NATURAL SOIL TEXTURE	PERCOLATION RATE	INFILTRATION RATE
Sand, sandy loam	0–30 min/in.	1.2 gpd/ft <sup>2</sup>
Loams, silt loams	31–45	0.75
Silt loams, silty clay loams	46–60	0.5
Clay loams, clay	61–120	0.25



**FIGURE 27.7** Cross section of a typical ET bed. (U.S. EPA 1979)

tem with a rock cover is provided to distribute the influent throughout the bed. The area is landscaped to direct all runoff away from the bed.

An ET bed can only function in areas having a low annual rainfall. Annual evaporation should exceed the rainfall by at least 24 in. The average annual temperature must be sufficient to supply the latent heat of vaporization. These restrictions limit acceptable operation of ET systems to the southwest section of the U.S. Much of the upper west region between the Mississippi basin and the east side of the Cascade Range has a high positive evaporation rate, but low winter temperatures makes the operation less efficient. Most likely, areas that permit the installation of ET systems will have developed local design criteria.

## ON-SITE WASTEWATER TREATMENT FOLLOWED BY A SURFACE DISCHARGE

### General

Relatively few homes are sited to permit a surface discharge of treated wastewater. Surface discharges include land surface application, discharge to a permanent flowing stream, an intermittent stream, a dry ditch, or a pond or lake. For single-family homes, this means a building lot size of more than one acre is needed. Also, the cost of installation, operation and maintenance is greater than for the on-site absorption system. However, several suitable treatment technologies are available for serving single-family homes, as well as small communities. Because of the cost associated with the required treatment units, their use will generally be limited to locations where land and home values are substantial and other alternatives are not available for wastewater management.

A permit is required by federal law for all surface discharges. National Pollution Discharge Elimination System Permits (NPDES permits) were established under the 1972 Water Pollution Control Act (P.L. 92-500). Most of the states now administer this program and issue a state permit. The level of treatment required will depend on the dis-

charge conditions, but in all instances secondary treatment or equivalent followed by disinfection is required. Secondary treatment involves subjecting the wastewater flow to biological treatment and clarification. Effluents from intermittent sand filters are superior to those from a conventional secondary biological process. Both BOD and suspended solids must be reduced to less than 30 mg/l. Discharges to small streams where little or no dilution is provided, to impounded water like lakes and ponds, and special designated waters such as shellfish harvesting areas or trout waters may require treatment that is more stringent or may even prohibit a discharge. Treatment beyond the secondary level may involve nitrogen and phosphorus removal and/or a greater reduction in BOD and suspended solids.

### Intermittent Sand Filtration

This technology has been used for many years with excellent success. There are many sand filters in use throughout the U.S., treating wastewater from individual homes, small commercial and institutional developments and small clusters of homes. If properly designed and operated, an intermittent sand filter will produce an effluent having very low BOD and suspended solids. The process is well-suited as an on-site method of treating septic tank effluent prior to a surface discharge.

An intermittent sand filter is a bed of sand 24 to 36 in. deep and underlain with graded gravel and a flow-collecting manifold. General design criteria are given in Table 27.5. Conditions that have significant impact on the performance of an intermittent sand filter include:

- The quality of the applied wastewater. A septic tank effluent should be considered the minimum acceptable pretreatment. Furthermore, grease and solids loading should be kept to a minimum.
- Temperature and dissolved oxygen. The organics removed by the filter must be decomposed by biologi-



TABLE 27.5 Design Criteria for Intermittent Sand Filters

ITEM	CRITERIA
Pretreatment	Minimum—septic tank or equivalent
Hydraulic loading gpd/ft <sup>2</sup>	1 to 5 gpd/ft <sup>2</sup> , smaller buried filters Larger open filters to 5 gpd/ft <sup>2</sup>
Filtering media	Washed durable granular material
Effective size	0.5 to 1.0 mm
Uniformity coefficient	<3.5
Depth	24 to 36 in.
Underdrains	
Material	Perforated plastic pipe, 4-in. diameter
Slope of underdrain	0.5 to 1.0%
Bedding	Washed durable gravel, ¼ to 1½ in. size graded bedding with small-size gravel on the top
Venting	Upstream end of filter bottom
Dosing	Complete 2-in. flooding twice per day Hydraulic loading rate 2.5 gpd/ft <sup>2</sup> May range from 0.75 to 15 gpd/ft <sup>2</sup>
Distribution	Splash plates

cal activity. The rate of decomposition is temperature-dependent, and this limits the loading rate to the filter.

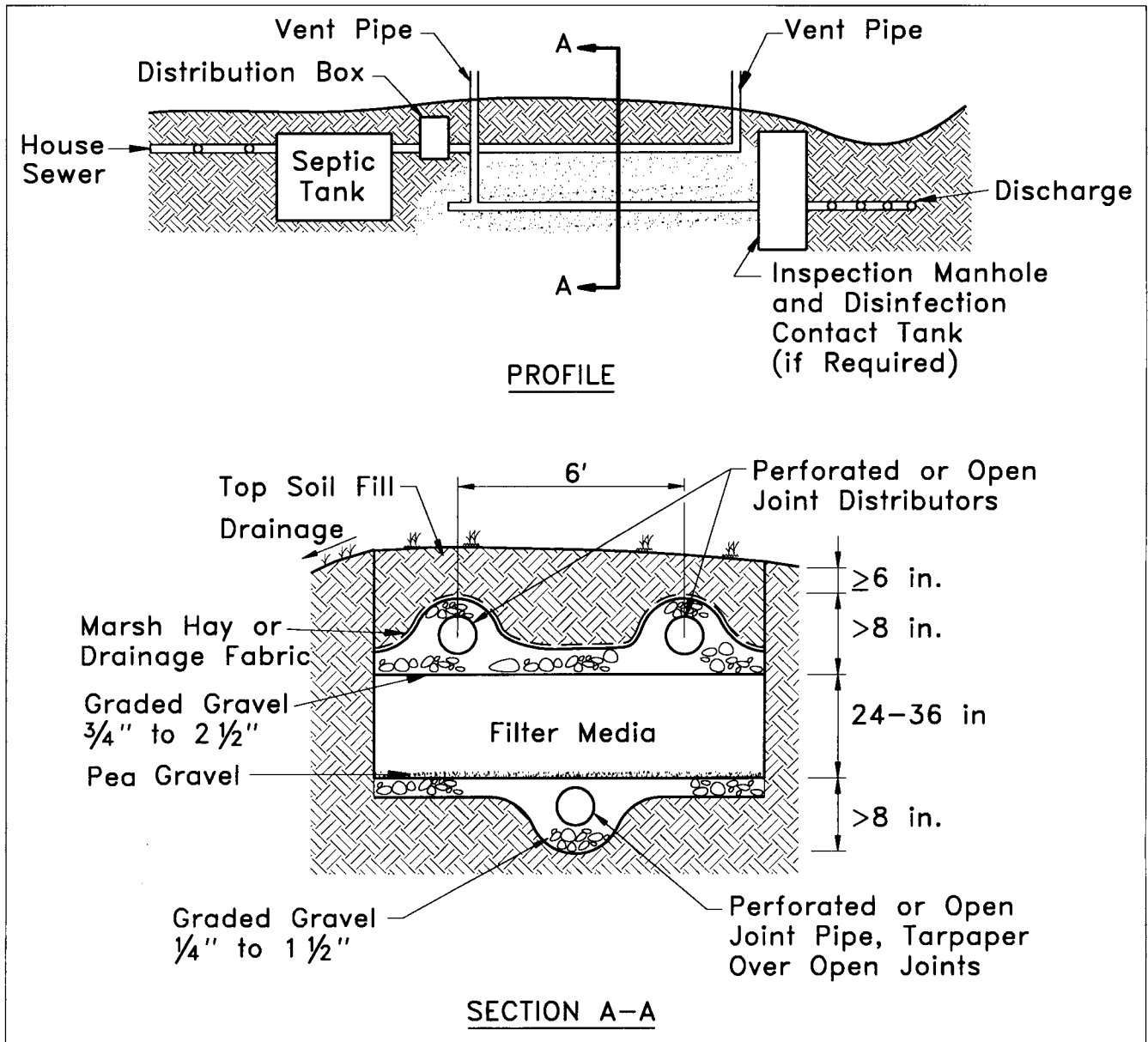
■ Sand size and uniformity. The allowable loading rate for a sand filter is affected by the sand grain size, filtration rate, penetration depth of particulate matter, and the effluent quality. The uniformity of the sand establishes the porosity, with a small uniform grain size giving the best filtration and highest porosity. The effective size is the size where 10% by weight of the particles are smaller = E10. Uniformity in size is expressed by the “uniformity coefficient,” which is the sieve size that will pass 60% of the mass (E60) to the effective size = E60/E10. A uniformity coefficient of less than 4 is considered acceptable, but a material having a lower value will give better performance. The small additional cost of a uniform sand will materially improve the long-term operation of the filter, will probably reduce the maintenance cost and will improve the effluent quality. It is important that the sand size be uniform throughout the filter.

The filters remove pollutants by trapping them on the sand surface, by straining the flow as it percolates through the bed, and by absorption and adsorption by the biological growth within the filter. Excessive loading will result in filter clogging, which causes ponding on the filter surface, leading to deterioration of effluent quality and, hence, filter

failure. Good treatment depends on oxygen being drawn into the bed to maintain aerobic conditions. This is accomplished by intermittent flooding of the bed. As the applied flow percolates into the bed, air is drawn in, replenishing the consumed oxygen.

Figure 27.8 shows a typical buried intermittent sand filter installation that is applicable for use at a single-family residence where a surface discharge is required. The filter follows the septic tank. Provisions must also be made for disinfection by chlorination, prior to discharge. Intermittent sand filters can be buried, covered with a means of access, or left open to the atmosphere. In colder climates, protection against freezing is required. The buried filter may employ rock distribution of the influent, similar to the design used for a mound bed. The rock layer will support a soil cover. Otherwise, a covered filter could be constructed similar to the recirculating filter shown in Figure 27.9.

Figure 27.9 shows a typical flow diagram for a small recirculating intermittent sand filter. Flow from the pretreatment septic tank is piped to a wet well/dosing/recirculation tank. The tank is sized to provide a 2-in. flooding of the filter. The figure shows a splash plate located at the filter center for distributing the flow without eroding the sand. The filter effluent piping is routed through the dosing tank where a float valve permits the flow to fill the dosing tank before any flow is discharged. Recirculation permits higher daily hydraulic loading to the filter without increasing the



**FIGURE 27.8** Typical buried intermittent sand filter. (U.S. EPA 1979)

organic load, with some installations being dosed as often as every 30 minutes. This improves filter performance by drawing more air into the filter with the more frequent dosing. Non-recirculating filters are generally designed for two dosings per day when septic tank effluent is applied because of the higher organic strength of the applied loading.

The engineer should always consider an intermittent sand filter where a surface discharge is required for a small wastewater flow. The  $BOD_5$  ( $BOD$  for first 5 days at  $20^\circ C$ ) and the suspended solids of the filter effluent will each normally be less than 10 mg/l. Most of the nitrogen will be oxidized to nitrates. When properly designed and operated, reliable treatment is achieved with little maintenance being required. As with all wastewater, disinfection is required

prior to discharge. The simplest means of accomplishing disinfection for small flows is by use of a tablet chlorinator as shown in Figure 27.10. Calcium hypochlorite tablets are stacked in the feed tube, and the filter effluent flows through the chamber. The size of the chamber and number of feed tubes is varied in the construction to provide the proper chlorine dosage. The flow is not retained in the chlorinator but flows out into a holding tank called a contact tank, where a 30-min contact time is provided to insure the kill-off of pathogens. One to three mg/l of chlorine residual should remain after the 30-min contact period.

The permitting authority may impose restrictions on the area surrounding a surface discharge for health and safety purposes. These include:

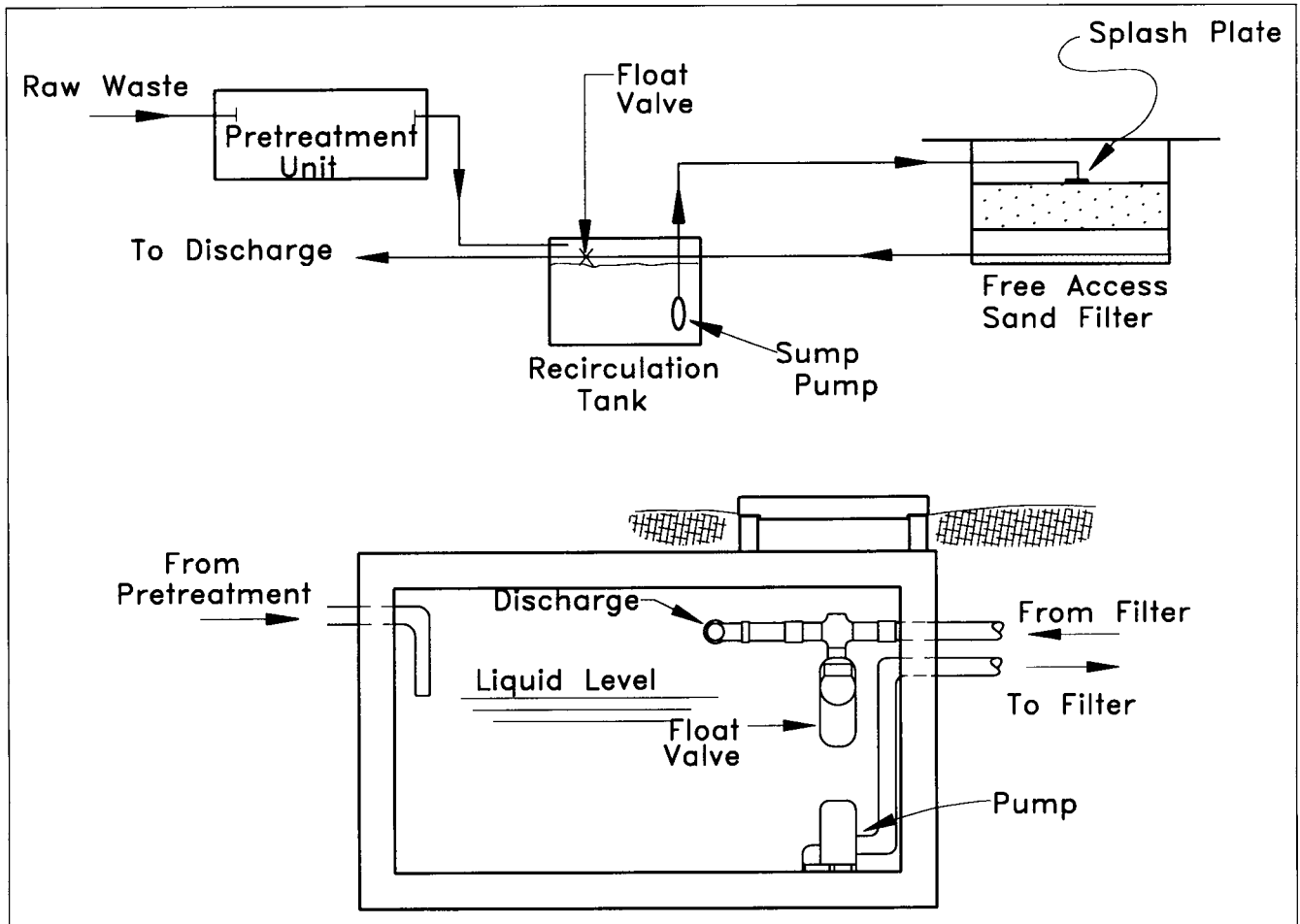


FIGURE 27.9 Typical recirculating intermittent sand filter. (U.S. EPA 1979)

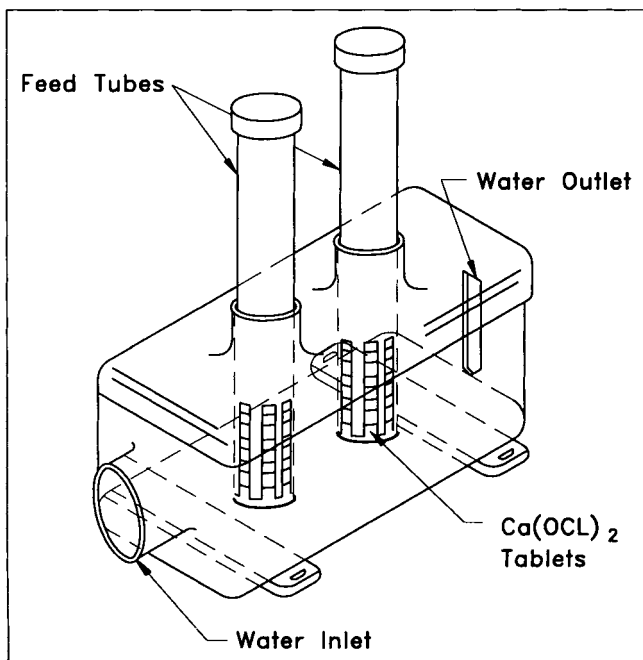


FIGURE 27.10 Tablet chlorinator. (U.S. EPA 1979)

- When discharging to a point providing a dilution of less than 10 to 1, direct human water contact should be restricted by ownership or easement of the flow path downstream for 500 ft.
- When discharging to a dry ditch or to an intermittent stream, the channel slope should be greater than 1% and ponding should not be permitted.
- Where two or more discharges are to the same dry or intermittent flow channel, the discharge points should be more than 500 ft apart.
- Where subdivisions require a surface discharge, a single treatment plant and discharge point is recommended instead of individual systems. The engineer should review the regulations of the permitting agencies before starting a design.

### Biological Treatment

Both anaerobic (in the absence of free oxygen) and aerobic (in the presence of free oxygen) processes are utilized in wastewater treatment. The septic tank is an example of an anaerobic process. Anaerobic processes perform best at tem-

peratures between 90 and 95°F. Only a low level of biological decomposition and stabilization is accomplished in the conventional septic tank, except in very warm climates. Also, the anaerobic process is not as efficient as aerobic processes in terms of level of decomposition and quality of liquid effluent produced. Other than the septic tank, anaerobic treatment is not common to small systems.

Theoretically, the end products of aerobic decomposition are carbon dioxide and water and a small quantity of salts. In reality, there is a significant fraction of all organic matter that is refractory to oxidation. This is the organic residual found in topsoil and stable forms of humus. Since the normal aerobic process is never continued for a sufficient period to accomplish complete oxidation, and because of the presence of the refractory component, there is always a residual called sludge that must be disposed. The stability of this sludge depends on the conditioning process employed. Most local governments provide a central facility for this function. Aerobic digestion is the most suitable means of on-site septic tank sludge stabilization.

Aerobic processes employed in the treatment of small flows are of several types, including fixed film and suspended growth processes. With both the fixed film and the suspended growth processes, the organic constituents in the wastewater are adsorbed and absorbed by a biological growth. These organic constituents are then used as an energy source and for growth. With the fixed film process, biological growth is supported on an inert surface. An early form of a fixed film process that has been widely used in wastewater treatment is the trickling filter, which was originally a bed of rock with the rock surfaces supporting the fixed film growth. Various plastic forms are now available, and these have largely replaced the use of rock in the beds because better circulation of air is achieved and a greater surface area per unit volume is provided. Another form of fixed film support is the rotating biological contractor, where discs are mounted on a shaft such that the lower portion of the discs are submerged in the flow. As the shaft is rotated, the discs rotate into the air where oxygen is available. Air may also be added to the flow and used to assist with the rotation of the discs. These processes must be followed by final clarification and disinfection. In addition, the sludge from the clarification process must be processed. Dewatering and air drying on a small sand bed is one method of handling this sludge. Also common for treating small flows are aerated and facultative lagoons.

The suspended-growth aerobic process is more common to all wastewater treatment, including small-flow treatment. A variety of designed "package units" are available for use as a means of providing on-site treatment. All aerobic suspended-growth processes (commonly called "activated sludge") include a contact chamber where the incoming flow is mixed with a biological population. The term "mixed liquor" is used to describe the combination of the incoming flow and the biological population. The solids that are within the mixed liquor suspension are called

"mixed liquor suspended solids" (MLSS). The mixed liquor is agitated by aeration to supply oxygen and to prevent separation of the biomass from the mixed liquor. Diffused air is usually used to accomplish both mixing and aeration. This period of aeration and mixing is usually 24 to 48 hr for small systems, and the process is called "extended aeration." During this period, the organic material is oxidized and converted to cell mass. The increase in cell mass is 30 to 50% of the BOD added to the system. Therefore, biomass (waste activated sludge) must be wasted from the system. With the extended aeration process, the residual biomass is relatively stable and can usually be spread on land. However, air drying beds or other disposal means may be required at many locations. A local wastewater treatment plant may take the sludge for stabilization. In addition, the local facility for treating septic tank sludge may be a suitable means of disposal.

Effective biological treatment depends on keeping the biomass in a proper physiological state, whereby the biomass can be separated by gravitational settling, producing a clarified effluent. The level and method of feeding largely determine the physiological state of the biomass. The level of feeding or "food to micro-organism ratio" (F/M ratio) is controlled by adjusting the concentration of the biomass to the feeding rate. The biomass is expressed as the concentration of the MLSS in mg/l, and the feed rate is the BOD in mg/l in the incoming flow. Some operating parameters for extended-aeration systems are given in Table 27.6. The normal per capita contribution of BOD<sub>5</sub> is 0.17 lbs./day, and for single-family dwellings the per bedroom contribution may be taken as 0.4 lb/day. The activated-sludge process is stable and produces a high-quality effluent when properly designed and operated. The process is subject to upset, resulting in poor removal of BOD and suspended solids or poor separation of the mixed liquor from the flow in the clarification process, producing a poor-quality effluent in either instance. The extended-aeration process is relatively more stable. Where a proper food to micro-organism ratio, oxygen level and mixing is maintained, few problems should develop. The dumping of excessive amounts of cleaning agents or other toxic agents into the sewer where on-site treatment is practiced may cause failure of the process.

It is common for small treatment units to be of the "package" type, as it is not economical to design and construct an on-site system in places where the daily flow is less than 200,000 gal. Most suppliers of smaller units, such as those less than 10,000 gpd capacity, have the product tested and performance certified. The National Sanitation Foundation (NSF) provides this service, and the local permitting agency may require data of this type. When a package unit is specified, catalog data will provide the needed design information. All design conditions, as shown in Table 27.6, will be available from manufacturer literature.

All on-site aerobic systems should have a high-water alarm to notify the homeowner of a malfunction. As noted

**TABLE 27.6 Operating Parameters for Extended-Aeration Processes for Small On-site Units**

PARAMETER	AVERAGE PARAMETER
MLSS, mg/l	2000–6000 (5000 to 6000 common)
F/M Ratio, lb BOD per day/lb MLSS	0.05–0.1
Solids retention time, days (related to F/M)	20–100
Hydraulic retention time, days	1 to 5 (1.5 to 2.0 common)
Dissolved oxygen, minimum level in mg/l	>2.0
Discharge oxygen concentration generally	>5.0, so unit should carry this level
Energy input for mixing, Hp/1000 ft <sup>3</sup>	0.5–1.0
Energy input is usually by diffused air.	
Clarifier overflow rate, gpd/ft <sup>2</sup>	200–400
Clarifier solids loading, lb/d/ft <sup>2</sup>	20–30
Clarifier weir loading, gpd/ft of weir	10,000–30,000
Sludge wasting frequency, months	6–12

Note: Design parameters of “package units” controlled by the manufacturer.

in Table 27.6, sludge will need to be wasted from the system about every 6 months. The operating permit will also require a monthly or quarterly report on the plant performance. This report will usually need to come from a qualified operator. In fact, some suppliers of equipment provide this service. The operator should also be responsible for having sludge removed from the system as required (see Table 27.6). Normally, the sludge is removed by a firm that pumps septic tanks and can take the sludge to a seepage disposal facility. As noted during the discussion of intermittent sand filters, all surface discharges should be disinfected, with the tablet chlorinator being the most suitable technology. Some locations may require dechlorination after the 30-min contact time and before discharge. A tablet sulfonator, having a design similar to the tablet chlorinator, is available for this purpose. The sulfonator can be placed on the discharge line from the contact tank to oxidize the residual free chlorine forms. The reaction is instantaneous, and only flow through the chamber is required (no contact time needed, as required for disinfection).

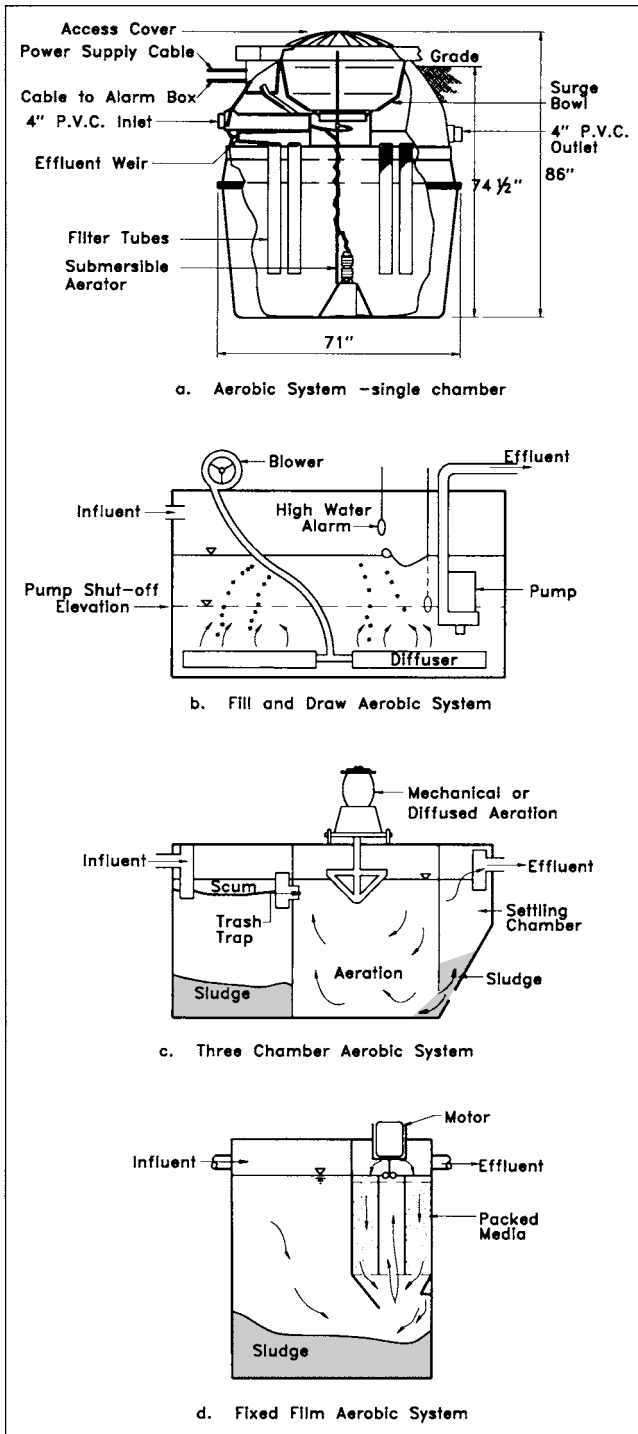
Figures 27.11 a, b, c and d show four types of on-site “package” units that are available in several sizes for treating flows up to 2000 gpd. The unit shown in Figure 27.11-a is available in five sizes from 500 to 1500 gpd. It is a developed and tested technology, with the entire process housed in a barrel-shaped fiberglass container. A 4-in. inlet and outlet are provided for connecting the house sewer to the inlet and the disinfection chamber to the outlet. Electrical connections are provided for a power supply and a

high-water alarm. A submersible aerator is used to mix the liquid and provide oxygen to the mixed liquor. The aerator does not always need to operate continuously, but may be controlled by a timer. Patented filter tubes are used to filter the effluent, eliminating the need for a clarifier. The manufacturer of these filter tubes reports that solids need to be removed every 6 to 12 months. The filter tubes control the quality of the effluent, making temporary biological upsets less likely to cause a poor-quality effluent to be discharged.

The unit shown in Figure 27.11-b is a batch reactor having a fill-and-draw operation. Diffused air is used for aeration and mixing. A pump control turns off the aeration blower when the high operating liquid level is reached. After a reasonable settling time (1 hour or more), the effluent pump discharges clarified effluent until the low-water level is reached, after which the pump stops, the aeration is started, and the fill-and-draw cycle is repeated. Sludge must be wasted periodically, as was noted for the unit shown in Figure 27.11-a, above.

Figure 27.11-c shows a three-chambered process that employs a surface mechanical aerator for mixing and aeration. An initial chamber similar to a small septic tank is located ahead of the aeration basin. This chamber serves to remove scum, heavier sludge particles and trash. A clarifier or settling chamber follows the aeration chamber where, through its design, the solids automatically fall back into the aeration basin.

Figure 27.11-d shows an on-site process employing a fixed-film treatment design. A surface aerator draws water



**FIGURE 27.11** Typical on-site "package" biological treatment units. Reproduced with permission from Multi-Flo Waste Treatment Systems, Dayton, Ohio (a); and U.S. EPA 1979 (b, c, and d).

up through a center chamber, where it flows back through the packed media and into the quiescent chamber, where suspended solids are allowed to settle to the bottom. The effluent flow rate is equal to the inflow, with a constant level being maintained in the biological reactor.

The homeowner or developer should use the services of a qualified engineer in selecting an on-site treatment unit. It is recommended that the local health department be contacted for information on what types of units are being marketed in the area and the reliability of operation of existing units. Useful information on types of units available can be obtained from several sources. The National Sanitation Foundation (NSF) provides testing services to equipment manufacturers of small treatment units. Some regulatory agencies require that the NSF test new equipment before approving its use.

The homeowner must use good judgment and caution in controlling what is flushed down the drain where an on-site system is used. Diapers, towels, plastic items and any material that will not be readily dissolved in water will decrease system performance and lead to a need for additional maintenance.

On-site wastewater treatment is a developing field. Current information on available technology is available from the sources listed in Table 27.7.

## SMALL COMMUNITY WASTEWATER TREATMENT PLANTS

### General

The type of facility required for a small community treatment plant will depend on local conditions and practices. The terms "primary," "secondary" and "tertiary" are used to define levels of treatment. The term "advanced wastewater treatment" is often used to indicate a level of treatment beyond secondary. Domestic wastewater is an ever-changing mixture that is more than 99.9% water and less than 0.1% suspended and dissolved matter ( $0.1\% = 1000 \text{ mg/l}$ ). The suspended and dissolved matter consists of a multitude of chemical and biological compounds, including the inert dissolved salts from the water supply, dissolved organic material, grit (fine inorganic particulate matter), rags, paper and plastic matter that is flushed down the drain. Additionally, the suspended and dissolved matter will include a high concentration of enteric organisms, which may include pathogenic forms. While wastewater is an ever-changing mixture, with the characteristics changing from minute to minute, the broad measures of strength such as BOD and suspended solids for domestic sewage are always within a defined range on a daily basis. The diurnal strength varies with the community activity, with the strength being highest during periods of highest flow. Sewers serving new communities should not be subject to problems with inflow and infiltration as suitable construction materials and technology are available to the engineer for insuring water-tight systems.

When a central facility is needed for wastewater treatment, the engineer should conduct a study to determine the availability of capacity at existing plants. Routing the flow to an existing treatment plant where possible is preferred to the construction of a new plant. The unnecessary proliferation of small plants is not an environmentally

TABLE 27.7 Sources for Information on On-site Wastewater Treatment

EPA National Small Flow Clearinghouse P.O. Box 6064 Morgantown, WV 26506-6064 Toll Free 1-800-624-8301	Your State Water Quality Agency Your State USDA Farmers Home Administration Office
National Sanitation Foundation P.O. Box 1468 Ann Arbor, MI 48106 (313) 769-8010	EPA Center for Environmental Research Information 26 W. Saint Clair Cincinnati, OH 24268
The National Rural Water Association Wastewater Technical Program (Local office in most States)	Catalog Data from Purveyors of Wastewater Treatment Equipment
National Association of Towns and Townships 1522 K Street N.W., Suite 730 Washington, D.C. 20005 (202) 737-5200	EPA Regional Office

sound practice. The cost of treatment will generally be less at a larger facility.

When a decision is made to construct a new plant to serve a community, a determination must be made on the plant capacity, the site location, treatment level required, who will own, operate, maintain and control the completed facility and how the construction cost will be funded. Principal items related to these four considerations include:

- *Treatment plant capacity.* A permit to construct and operate a new plant will come under the authority of both state and local government. These agencies together with the local planning and zoning departments must be consulted about zoning, plant location, area to be served, and total population to be served. The projected growth of the proposed service area should be coordinated with the local planning department staff. The initial design capacity will generally be based on the area needs for a period of 15 to 20 yr. In most instances, land should be set aside for future expansion and provisions made in the design to accommodate any projected expansion. There may be instances where the treatment facility will be designed solely to serve a single development, but this will generally not be the case if the local government has developed a comprehensive plan. It is incumbent upon the engineer to insure that suitable planning precedes the location and sizing of a new plant.
- *Plant site.* A suitable discharge point must be available. It is generally preferred that the plant be located adjacent to the discharge point, but this is not a requirement. The plant must be protected against the

100-yr flood elevation. Suitable buffering of the site from the surrounding community must be provided. Buffer requirements are generally several hundred feet from the nearest residence. An all-weather road must be provided to the plant site. Additionally, a reliable water supply and three-phase power must be provided. Local zoning, or a zoning variance, must be obtained for the intended land use. Downstream water uses, such as an existing public water supply intake, a public recreational site, a shellfish harvesting area, or other water quality considerations may eliminate a potential site or discharge point from further consideration.

- *Level of treatment.* Treatment requirements will depend on several factors, including the class of the receiving water, the dilution provided by the receiving stream (generally based on the 7-day, 10-yr low flow), and the hydraulic features of the stream related to mixing and reaeration, such as slope, depth, type of stream bottom and downstream conditions. The level of treatment required will be established by the state water pollution control agency, but in all cases, secondary treatment will be required as a minimum by federal law. The state agency having jurisdiction for permitting wastewater treatment plants may conduct a site-specific field study of the receiving water body to establish the assimilative capacity, or it may require the engineer to conduct the study prior to establishing the treatment requirements. Some states will only issue a discharge permit to a public agency. In this case, arrangements will need to be made with the agency that will own and operate the plant before starting design,

as the operating agency may have specific design requirements.

- *Funding of construction and operation.* The local government is unlikely to participate in funding the construction of a new plant unless it will meet a regional need. If the area has been planned for sewer, some level of cost participation may be arranged, and the willingness of the developer to assist with the cost may expedite the construction program. Otherwise, the developer may be faced with providing the entire construction cost. Operating costs should come from a service fee. The fee for sewer service should cover amortization of construction costs, operation and maintenance costs, a replacement fund and general administration costs. The construction cost is often prorated to each building unit served and collected as a part of the cost for a construction permit.

An environmental assessment will be required for the selected plant site. After the engineer has completed a preliminary investigation covering at least the four subjects presented above and found that the selected site is, in general, acceptable, an in-depth environmental assessment is prepared and copies of the report presented to the local and state authorities. A presentation at a public hearing, which is generally held as part of the approval process, will often be required. The environmental assessment will include a thorough investigation of the esthetics, ecological, physical and social aspects of the proposed project. General potential impacts associated with the construction and operation of a wastewater treatment plant include lighting at the plant, neighborhood aesthetics, noise, odor, traffic, and air pollution. These impacts can be mitigated through good design and operation.

### Pollutant Characteristics

The principal pollutional characteristics of domestic sewage are:

- Floatable matter or scum, which is unsightly if discharged to a surface water.
- Settleable matter that will form sludge deposits on the stream bottom.
- An oxygen demand that may exceed the oxygen resources in the stream, resulting in septic conditions. The pollutant potential of scum and settleable solids is easily recognized, but the oxygen balance of surface water is not always as well understood.
- The potential for pathogenic organisms to be present.
- Domestic sewage should not contain toxic substances.

The indigenous life in surface water is aerobic, which means dissolved oxygen is required to maintain that life.

The atmosphere is approximately 20% oxygen and 80% nitrogen. At normal water temperature (60 to 70 degrees), oxygen solubility in surface water is in the range of 9.5 mg/l. Saturation or equilibrium solubility is a function of water temperature and atmospheric pressure. As oxygen is used by aquatic life it must be replenished. The sources for this oxygen replenishment include the atmosphere and autotrophic growth (photosynthetic growth where oxygen is liberated when sufficient UV light is available for growth) as algae respire under this condition. Atmospheric replenishment depends primarily on surface renewal for oxygen transfer. There is a finite surface layer of water that quickly comes into equilibrium with the oxygen in the atmosphere. However, the molecular penetration of oxygen to greater depths is a slow process. If mixing of the water is provided, such as found in a rapidly flowing stream, the rate of surface renewal is rapid since the saturated surface layer is continually mixed with water at depths beneath the surface while a new layer is brought to the surface. Thus, aquatic life like the mountain trout, which requires a high level of oxygen, is found in a shallow rapidly flowing stream, while the more oxygen tolerant aquatic forms are found in deeper, slower moving water where the oxygen level may be only 2 to 3 mg/l.

While floating matter and bottom sludge deposits are degrading to stream quality, it is the oxygen level that generally has the most overall degradation impact. Oxygen demand from sewage results from the oxidation of the organic load (BOD) to stable forms. The oxidation is primarily bacterial activity. Visual evidence, such as floating dead fish that have suffocated, is one type of degradation resulting from exceeding the oxygen resources of a stream. However, when fish are dying, the damage to the aquatic environment is much broader. In the past, most aerobic aquatic forms were eliminated from segments of streams downstream from many wastewater discharge points. The most noteworthy research relating to surface water pollution is that conducted by two U.S. Public Health Service Officers, H.W. Streeter and E.B. Phelps, "A Study of the Pollution and Natural Purification of the Ohio River," Public Health Service Bulletin No. 146, U.S. Public Health Service, Washington, D.C. (1925). This work remains the foundation that is used in establishing the assimilative capacity of flowing surface waters. Secondary treatment is now required for all wastewater prior to discharge by the 1972 Water Quality Act. A higher level of treatment may be required to prevent stream degradation. This means that the level of treatment must reduce the oxygen demand (BOD) of the plant effluent to less than the reaeration capacity of the receiving water.

### Wastewater Treatment

The wastewater treatment process creates a controlled environment for reducing the characteristics of the effluent that will cause degradation of surface waters or cause a potential health hazard. As noted above, for domestic sewage, the characteristics include floatable material, settleable



material, material that will impose an oxygen demand, and pathogenic bacteria. Other potential pollutants common to all domestic wastewater are nitrogen and phosphorus. An increase in the availability of one or both of these elements may lead to eutrophication of the receiving water. Unoxidized forms of nitrogen, such as ammonia, represent an oxygen demand as the compounds are oxidized to nitrates.

The five primary elements making up all organic matter are carbon, oxygen, hydrogen, nitrogen and phosphorus. The composition of cell tissue is  $C_6H_7NO_2$ . Cell tissue is about 27.5% nitrogen and less than 2.0% phosphorus. Of the five primary elements, only nitrogen and phosphorus are potentially growth-limiting in natural waters, since oxygen and hydrogen are available from water and carbon is obtained from carbon dioxide. Excessive aquatic autotrophic growth (algae growth) is a form of water quality degradation and is undesirable for several reasons, with the oxygen demand expressed by algae during the night and on cloudy days being a principal consideration. Nitrogen oxidation to eliminate the oxygen demand and nitrogen and/or phosphorus removal to mitigate the eutrophication potential will be required for some discharge locations.

*Primary treatment* of wastewater is the removal of the larger suspended solids by gravitational settling. It is accomplished by passing the flow through a clarifier that will generally provide a holding time of 2 hr (volume of clarifier/design flow rate = 2 hr). Gravitational settling will remove much of the floatable and settleable suspended materials, with approximately 35% of the BOD and 60% of the suspended solids being removed by primary treatment. Before the 1950s, only primary treatment was provided at many wastewater treatment plants. This level of treatment removed most of the visual forms of pollution, such as floating and settleable matter. Primary treatment remains common to larger plants, where it is cheaper to handle the additional waste stream (primary sludge) separately than to treat the total waste load with a biological process. The clarifier provides a quiescent environment where the solids are allowed to settle and compact near the chamber bottom, where they are removed, and the clarified liquid is decanted over a peripheral weir. The design parameters for a primary clarifier are: surface loading in  $\text{gpd}/\text{ft}^2$  of surface area—surface loading determines the efficiency of solids removal; depth, which is required for solids compaction and for equipment; and weir loading, which must be maintained below rates where weir approach velocity will draw solids from the lower depths. Common design parameters are: surface loadings 600 to 1200  $\text{gpd}/\text{ft}^2$ ; depth, 6 to 8 ft; and weir loading, 10,000 to 15,000  $\text{gpd}/\text{ft}$ . Surface loading and depth establishes retention time in the clarifier as shown by the data in Table 27.8.

A 40-ft-diameter primary clarifier (also called a primary sedimentation tank) that is 7 ft deep and having a surface loading of 600  $\text{gpd}/\text{sq ft}$  will have a capacity of 753,600  $\text{gpd}$  and a weir loading of 6000  $\text{gpd}/\text{ft}$ . The weir must extend completely around the periphery of the tank and be

**TABLE 27.8 Primary Clarifier Parameters**

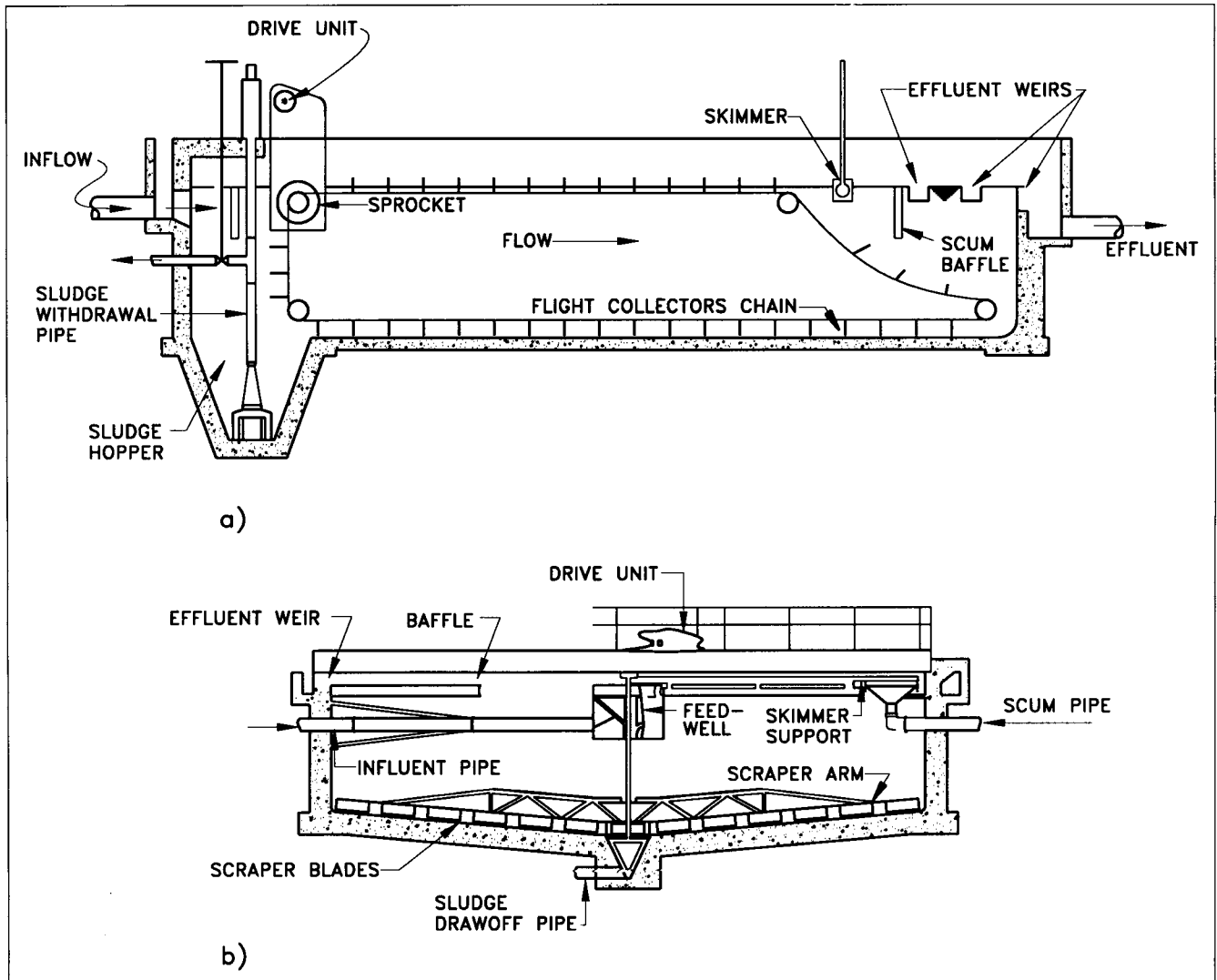
SURFACE LOADING	DEPTH	RETENTION TIME
600 $\text{gpd}/\text{ft}^2$	7 ft	2.1 hr
800	7	1.6
600	8	2.4

level so that flow is uniform across the tank. Generally, weir loading is never a problem with circular tanks. Low surface loading is money well spent in a clarifier design because efficiency of solids removal is related to this design parameter.

Clarifiers may be rectangular, but circular clarifiers are preferred because there is much less maintenance associated with the sludge collecting equipment with a circular shape. Figure 27.12 shows a cross-section view of both a circular and a rectangular clarifier. With the circular clarifier, the flow is piped to a center feed well, from which it flows to the periphery weir where the clarified supernatant is decanted into a trough. A revolving sludge collector scrapes the compacted solids to the tank center where they are withdrawn. Secondary clarifiers are similar to primary clarifiers, except surface loading may be lower to give greater solids-removal efficiency. The flow in a rectangular tank is from influent end to a weir at the effluent end of the tank. A continuous chain is located at each side of the tank for supporting sludge collector flights that move the settled sludge back to the influent end of the tank where it is withdrawn. Note that wear of the flights and chains are a maintenance consideration. Maintenance requirements, efficiency, and construction cost all favor the circular clarifier over the rectangular form.

Scum and bottom sludge are removed from the primary clarifier and generally stabilized by anaerobic digestion. Methane, a combustible gas, is a product of anaerobic digestion, and the gas may be used as a heat source in the plant operation. Anaerobic digestion is not an economical means of sludge stabilization for small treatment plants. At small plants where primary treatment is practiced, aerobic or chemical stabilization are suitable alternatives. The hauling of waste solids to a larger plant or to a septage disposal facility for treatment should always be considered when designing a small plant.

*Secondary treatment* is a biological process where the organic matter present is adsorbed and absorbed by a biomass (suspended-activated sludge or a fixed-film sludge), where it is oxidized for energy and used for cell growth (a food for the biomass). Secondary aeration reactors are rectangular in elevation and may be straight or circular in profile. The tanks are equipped with facilities to provide mixing to keep the biomass in contact with the organic matter in the



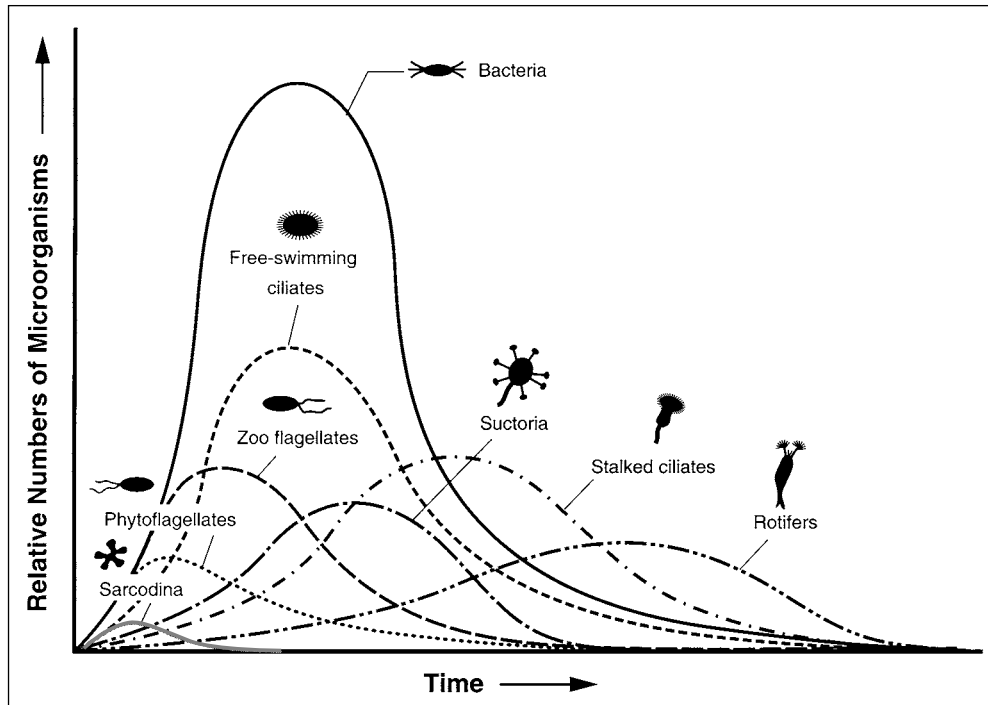
**FIGURE 27.12** Sectional views of clarifiers. (U.S. EPA 1979)

wastewater, while also preventing settling and to supply the required oxygen transfer. If the biomass is a suspended growth (MLSS), the process is called activated-sludge, and if the biological exists as film on a solid surface, it may be termed a tricking filter or biological contractor process.

It is important to distinguish between an activated-sludge process and plain aeration. Aeration of sewage will result in some coagulation of solids, but after aeration for a few hours or even a few days, little treatment will be achieved. Aeration for 12 to 20 days will result in stabilization of the organic material present as this amount of time permits a complete cycle of the biological process from a growth phase through decay for the bacteria and other life forms (see Figure 27.13). When untreated wastewater is discharged to a surface stream, the natural biological stabilization process will occur over a downstream reach of the stream. If the oxygen requirement associated with the stabilization process exceeds the reaeration capacity of the stream, the oxygen content of the water will be reduced to

a low level or may be depleted entirely, eliminating the aerobic life forms indigenous to the environment. Wastewater treatment is the creation of a controlled environment for this natural stabilization process.

Activated sludge is an active biomass that feeds upon the organic matter in the wastewater. By maintaining this active biomass, wastewater treatment can be accomplished in a matter of minutes instead of the 2 weeks that is required by plain aeration. In order to produce a quality effluent, the biomass must readily settle to the bottom of a clarifier, leaving a clear effluent, low in BOD and suspended solids for discharge. Thus, it is necessary to maintain the biomass in its proper physiological state and to insure that it is composed of the proper types of micro-organisms to accomplish good separation. In all modifications of the activated-sludge process, the initial adsorption/absorption phase, which requires only a few minutes, must be followed by 2 to 6 hr of aeration for the biomass to assimilate the organic material through growth and respiration. The operation of an acti-



**FIGURE 27.13** Relative-growth phases found in an activated sludge. Reproduced with permission from McKinney, R. 1962, *Microbiology for Sanitary Engineers*, New York: McGraw-Hill.

vated-sludge process involves keeping a proper balance between the biomass (feeders) and the incoming organic load (food), and feeding the biomass in a fashion that will give the desired species selection.

Activated sludge is a gelatinous mass of organisms that includes bacteria, protozoa, metazoa, nematodes and worms. The gelatinous quality of the biomass, which is important for providing the agglomeration needed for good separation, results from keeping the bulk of the biomass in a declining growth phase, whereby extracellular polysaccharides are produced, and by fostering the growth of zooglycol-type bacteria. The protozoa and metazoa feed upon the relatively small number of dispersed bacteria growth present. Protozoa and metazoa feeding is necessary for the production of a quality effluent. If the quantity of dispersed growth exceeds the feeding capacity of the protozoa and metazoa population, a cloudy, poor quality effluent will result. Figure 27.13 is a generalized presentation showing the complex, interrelated, mixed biological population of an activated sludge. A well-operated process will maintain an active population of stalked ciliates and rotifers, which requires that a significant fraction of the bacteria biomass be in a declining-growth phase. A microscopic examination is a good means of checking on the quality of activated sludge.

The level of feeding (food to biomass ratio,  $F/M$ ) is the ratio of the BOD removed per day divided by the mixed liquor volatile suspended solids (MLVSS). Volatile suspended solids are the fraction of the solids that is represented by weight loss during combustion at  $600^{\circ}\text{C}$ .

Activated-sludge solids are about 70% volatile. Sludge age is the ratio of the MLVSS carried in the system to the cell production per day. Both terms are controlled by the amount of sludge removed from the process per day (sludge wasting). An activated-sludge, extended-aeration process treating domestic sludge is operated with a food to micro-organism (biomass) ratio of 0.05 to 0.15 lb BOD applied per day/lb MLSS, which corresponds to a sludge-age range of about 20 days. The  $F/M$  ratio for a process carrying a MLVSS of 2000 mg/l with an influent  $\text{BOD}_5$  of 200 mg/l is calculated as follows:

$$\begin{aligned}
 &\text{Pounds of BOD}_5 \text{ added per day} \\
 &= (\text{BOD}_{\text{in}} - \text{BOD}_{\text{eff}})(8.34 \text{ lb/gal})(Q, \text{gpd})/106 \\
 &\text{Pounds of MLVSS in aeration tank} \\
 &= (V, \text{ft}^3)(7.48 \text{ gal/ft}^3)(8.34) (\text{MLVSS})/106 \\
 F/M &= \frac{Q/V (0.133)(\text{BOD}_{\text{in}} - \text{BOD}_{\text{out}})}{\text{MLVSS}} \quad (27.1)
 \end{aligned}$$

where

$$\begin{aligned}
 Q &= \text{daily flow in gpd} \\
 V &= \text{volume of aeration tank in cu ft} \\
 \text{BOD}_{\text{in}} &= \text{BOD of influent to aeration tank in mg/l} \\
 \text{BOD}_{\text{out}} &= \text{BOD of plant effluent in mg/l}
 \end{aligned}$$

$$\begin{aligned}
 \text{Sludge age} \\
 &= \frac{(V, \text{ft}^3) 8.34 (\text{MLVSS, mg/l}) (7.48)}{(Q, \text{mgd})(8.34)(Y)(\text{BOD}_{\text{in}}) - k (\text{MLVSS, lb})} \quad (27.2)
 \end{aligned}$$

$$\text{Sludge age} = \frac{1}{(Y(F/M) - k)} \quad (27.3)$$

where

sludge age = pounds of MLVSS in the aeration tank divided by the net cell growth per day  
 $k$  = decay constant for the MLVSS = 0.055 day<sup>-1</sup> for domestic sewage.

The decay constant applies to the oxidation of the biomass. The term is small, and  $Y$  = cell yield in lbs. cell produced per lb. BOD removed = 0.5, VSS for domestic sewage.

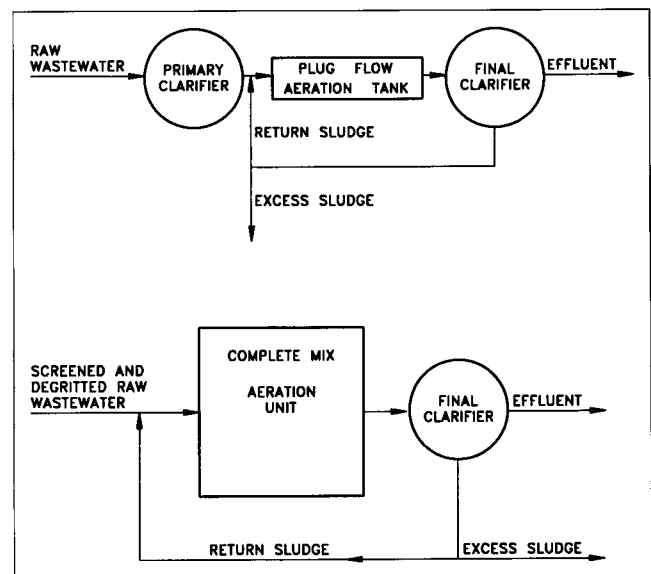
The MLSS concentration for an extended-aeration process is usually maintained between 3000 and 6000 mg/l, with operation at the lower end of the range generally producing the better-quality effluent. A high MLSS produces a more stable waste-sludge for disposal, but an excessively old sludge may result in a lower-quality effluent because of small particles of stable sludge that settle poorly and break away from the agglomerated biomass. The most important test for the operator in process control is the settleability test. It requires only 30 min and tells a lot about the process operation. The test is conducted by filling a 1-liter graduated cylinder with MLSS from the aeration tank. The solids are allowed to settle under complete quiescent conditions for 30 min. The level of the interface between the sludge and the supernatant, along with the quality of the supernatant, provides the operator with a visual indication of the effluent quality, the settleability and compactability of the sludge, and the amount of sludge to be wasted. Most extended-aeration processes operate well when the interface between the sludge and the supernatant is at about 200 to 250 ml. As the level of the interface increases, the operator can determine the fraction of sludge that needs to be wasted.

While the operator has the final control of the wastewater treatment process, the decisions of the design engineer are very important in establishing the overall treatment reliability. The settleability and compactability of the biomass largely determines the quality of the effluent. The absence of good settling and compactability is termed sludge-bulking. Sludge-bulking reflects the biomass composition, wherein a suspended growth that is predominantly composed of zoogal-type bacteria will show good compactability; a MLSS high in filamentous growth will be bulky, showing poor compactability and settleability. Poor compactability and settleability lead to poor-quality effluents because biomass is washed out in the effluent.

Several conditions may lead to promoting filamentous-type growths, including dissolved oxygen concentration, F/M ratio, sludge age, and flow regime. The plant design establishes the flow regime and the ability of the operator to control dissolved-oxygen levels. Filamentous growth may be promoted by either a high or a low F/M ratio. A high

F/M ratio occurs when the organic content of the incoming flow exceeds the initial absorption and adsorption capacity of the zoogal biomass present, leaving food for filamentous growth. This is a particular problem when treating high-strength wastewater. If the operator maintains a suitable F/M ratio, there should not be a problem with domestic sewage if the facility has been designed to mix the return sludge with the incoming wastewater properly. A low F/M ratio occurs when the incoming flow is dispersed, such as with a complete-mixed regime, before the zoogal biomass can achieve an initial BOD uptake. Filamentous forms have a greater surface-area-per-unit mass than a zoogal growth and can become predominant in a dispersed-food regime. The filamentous bacteria have little capacity to store food, compared to the zoogal-type biological growth. Therefore, under a regime where the zoogal forms are provided with the opportunity to adsorb food from a concentrated condition, they will utilize the stored food for the remainder of the aeration time, while the filamentous forms will be deprived of a food source. A low-level food concentration is available at all times in the complete-mixed process, and this regime can foster the selection of filamentous growth. Where the plant design and operation provides for uptake of the incoming food by the biomass before the flow is dispersed, and other operation conditions are favorable, the desired bacteria selection should occur. Figure 27.14 shows a schematic of each regime. In the complete-mix regime, conditions are the same throughout the chamber, making the location of the effluent of little significance. Under the plug-flow regime, the flow enters one end of the aeration tank and leaves at the opposite end, with one objective of the design being to have little longitudinal mixing.

The operator must conduct several tests on a routine basis for proper process control. These include: the strength



**FIGURE 27.14** Plug-flow and complete mixed regimes. (U.S. EPA 1977)

of the incoming wastewater (BOD and solids—the results from these tests will not vary greatly for a strictly domestic sewage, and variation in test results will generally not be of a nature that will require modification to the plant operation); the settleability test; MLSS, mg solids per liter; sludge return rate; and the sludge volume index ( $SVI = \text{ml of MLSS per gram of solids} = \text{equal the volume required to produce one gram of solids after settling 30 minutes}$ ). The settleability test is a visual test giving results in 30 minutes, whereas the SVI, and other solids testing, requires the completion of a laboratory test taking a few hours. The BOD test requires 5 days for completion. The operator must be able to interpret the settleability test, as it provides timely operational information, particularly for the small plant operator. The experienced operator will generally know from visual observations of the effluent and the settleability test how the plant is functioning. Laboratory analysis should be based on a 24-hr composite sample. A portable sampler should be a part of the operational equipment available at all wastewater treatment plants.

### Modifications to the Activated-Sludge Processes

There are several modifications to the activated-sludge process, but only five are considered to be of significance to small plants. These include plug flow, contact stabilization, complete mix, batch reactor and the oxidation ditch. The oxidation ditch represents a complete-regime. The plug-flow and complete-mix regimes are shown in Figure 27.14. Figure 27.15 shows a schematic view of an oxidation ditch, and the batch reactor is shown in Figure 27.16. With all complete-mixed regimes, the effluent has the same composition as the reactor and includes a fraction of the influent that just entered the tank. This does not contribute to poor-quality effluent because of the rapid uptake of the food supply by the biomass. Influent and effluent locations are relatively unimportant for a complete-mixed system since the reactor contents are uniform throughout. As noted previously, bioselection is more difficult to control in the complete-mix regime. Package plants are often designed as a “contact stabilization” process, as less tankage volume is required for this treatment regime. The process involves a short aeration period followed by clarification. The compacted, and thus smaller, sludge volume is then aerated for 6 or more hours before being recycled back to mix with the incoming flow.

**Batch Reactor.** One modification to the activated-sludge process that has been employed for several decades but not widely used is the sequential batch reactor. This modification was initially marketed as a series of concrete tanks connected in series. The precast tanks could be delivered to the plant site, placed on a suitable foundation, and connected together with piping. A benefit of this modification was that additional tanks or an additional treatment train could be added as additional capacity was needed. A later modification to this design alternative called for the placement of two tanks in parallel.

For most small community systems, two reactors are sufficient. Raw wastewater, following screening and grit removal, is split with the use of a valve along the influent line to each of the two reactors. A valve is provided to the influent line of each reactor that not only provides for flow control but also isolation of either unit for maintenance. Each reactor moves through four stages to accomplish treatment: (1) filling (aeration is provided during the filling stage), (2) aeration, (3) settling, and (4) decanting of clarified flow (see Figure 27.16-a). Under normal conditions, the entire cycle is completed in 4 hr (see Figure 27.16-b). Note that the flow is aerated for 2 hr after filling is complete. The aeration period is followed by 2 hr of settling, and a 2-hr decant period. The process is then repeated. As shown by Figure 27.16-b, the influent control valves are timed to fill one unit while the other is in the settling and decant mode. Note that the design shown includes a baffle that is located to provide two zones in the reactor. The initial zone is termed a “pre-react zone,” and the remainder of the tank is termed the “main-react zone.” The influent flow to the reactor is mixed with the consolidated biomass providing for rapid uptake of the organic constituents in the pre-react zone. There is no additional feeding of the biomass in the main-react zone. This is an effective means of promoting the growth of a desired zoogeal biopopulation while minimizing the growth of filamentous bacteria. The pre-react zone is normally 10 to 15% of the total basin volume. Sludge is wasted from the underflow near the end of the decant period. Disinfection facilities must be provided following the reactor.

The sequence timing for the process, as shown in Figure 27.16-c, can be adjusted to accommodate higher inflow rates, such as may occur during wet periods (high inflow during wet periods should not be a problem with a system

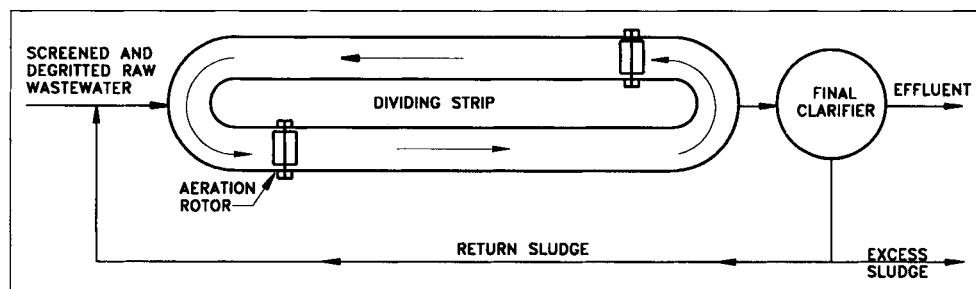
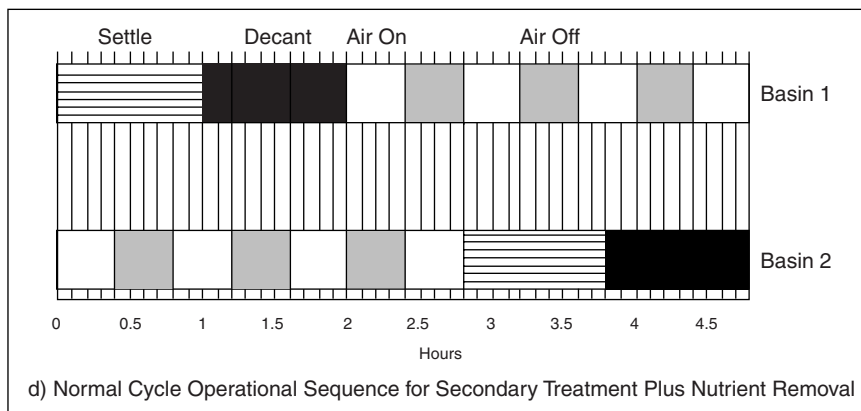
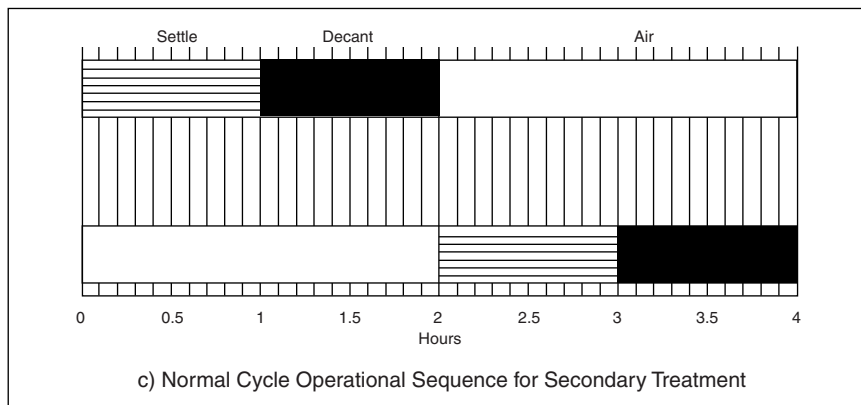
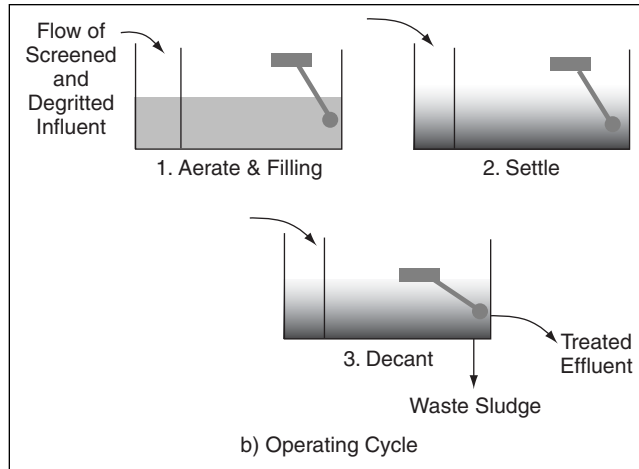
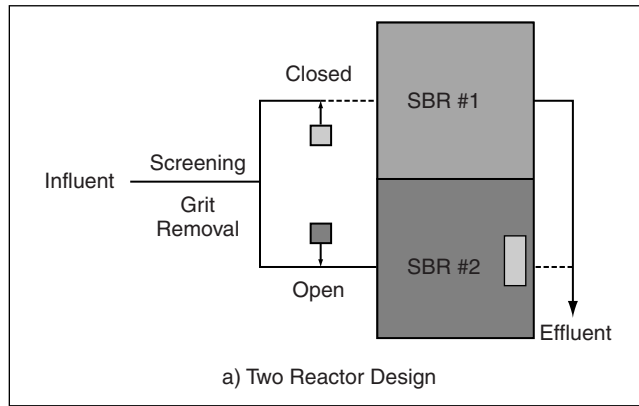


FIGURE 27.15 Schematic of an oxidation-ditch-flow regime. (U.S. EPA 1979)



**FIGURE 27.16** Batch reactor.

servicing a new development). The process is a modified plug-flow operation. High-level secondary treatment and nitrification can be accomplished. Note that the entire biological process and clarification occurs in a single tank.

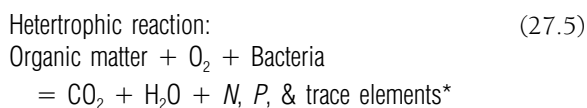
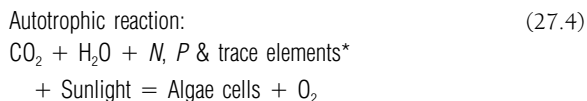
Nitrogen and phosphorus removal can also be accomplished as a part of the process by control of the aeration process (see Figures 27.16-d). The aerobic phases promote nitrification and phosphorus uptake. The air is turned off to create an anoxic phase, which accomplishes denitrification and phosphorus release. Please note that nitrogen and phosphorus control is covered in more detail in a later section.

The above reactor design, including the decant and aeration equipment and controls, is proprietary equipment marketed by the Sanitaire Corporation. The manufacturer's catalog should be consulted for more extensive data on design and operation.

While the batch reactor eliminates the need for a secondary clarifier and the associated valving and piping, the plant design must include facilities for grit, screenings and waste sludge.

**Stabilization Lagoons (Ponds).** This wastewater stabilization process may be considered for serving small communities where sufficient land is available for the lagoon system and the required buffer. Buffer requirements will vary, based on local conditions. The severity of winter conditions will largely determine the potential for odor problems and buffer needs. A buffer requirement of up to 500 ft is not uncommon. A 500-ft buffer establishes a plant site of 5 to 7 acres. The principal advantages of a lagoon system are the reliability of treatment and the low operational requirements.

The two types of lagoon designs that are applicable for small systems are the aerobic and the facultative lagoons. Wastewater treatment can be accomplished in temperate climates when employing a lagoon system without creating an environmental nuisance (malodors, etc.) because of the symbiotic conditions maintained between algae growth and bacterial decomposition. In the presence of sunlight and with the availability of nutrients that are provided by the wastewater, the following biological reactions occur.



In an aerobic lagoon, sufficient mixing and aeration must be provided to keep the solids suspended and to provide free oxygen throughout the lagoon depth. The completely

aerated lagoon is not common. Most lagoons are facultative in terms of biological activity. In the facultative lagoon, the only mixing that occurs is from surface wind action and from thermal density currents. Thus, there are two zones in the facultative lagoon. The top 2 ft or so contains algae growth and is generally supersaturated with oxygen from mid-morning until late afternoon. This top layer serves as a cap over the facultative and anaerobic activity occurring at the lower depths and in the bottom sludge. Malodorous gases and reduced compounds, such as ammonia and sulfides that are released into solution by the anaerobic activity in the bottom sludge, are oxidized to such stable compounds as  $\text{CO}_2$ , sulfates and nitrates as they reach the top layer.

When lagoons are operated at northern latitudes, where an ice cover will exist for weeks during the winter, the biological activity will be reduced and the lagoon will become septic throughout its depth. When the spring thaw comes, the septic gases will be released to the atmosphere causing a severe odor problem that may last for several weeks. A similar condition, though less severe, may occur where the winter water temperature drops to near freezing for several weeks and decomposition is slowed to where organic sludge accumulates under septic conditions. When the water temperature raises in the spring, the release of septic gases may exceed the solubility of the lagoon contents, releasing malodorous gases to the atmosphere.

Most states have regulations for the siting and design of lagoon systems. Retention times may vary from 10 to 15 days for the complete aerobic operation in temperate climates to more than 120 days for a facultative lagoon located at a northern latitude. The engineer should obtain information on design requirements in the state where the lagoon will be located. The information in Table 27.9 is provided as general guideline for lagoons located south of the 38th parallel of latitude.

Pond depth is limited to more than 4 ft to discourage bottom-rooted plant growth. The depth should be kept as shallow as possible to obtain the benefit of mixing from wind action at the surface and to limit the facultative zone. The lagoon side slopes should be uniform and constructed to the proposed grade. The cells must be water-tight below the water line. Plastic curtains are suitable for cell division barriers. A loop is formed at the top and bottom of the sheet plastic. This loop is used to hold the flotation material at the top of the sheet and a chain at the bottom. The chain rests firmly on the bottom to prevent movement from wind action. A window is cut in the plastic to permit flow between cells.

Most of the heterotrophic activity will occur in the first cell, with the second and third cells providing polishing treatment. The second and third cells, and if the system is not heavily loaded—all three cells, should develop a cover-surface of duckweed. This is desirable for the third lagoon, as suspended algae growth is essentially eliminated beneath the surface growth, producing a better-quality effluent.

\* There are many elements incorporated into organic matter, but *H*, *O*, *C*, *N*, and *P* are the five principal elements.

**TABLE 27.9 General Guidelines for Lagoons South of the 38th Parallel**

Pretreatment	Screening to remove floatable solids
Detention time	30–40 days
Water depth	4–6 feet
Number of cells	3 in series 50% of volume in first cell 30% of volume in second cell 20% of volume in third cell
Side slopes	3 horiz. to 1 vert., (3:1)
Cell bottom	Level
Inlet pipe—outlet at center of first cell to prevent solids buildup near the side slopes	
Fence to prevent access to lagoons—post notices around fence of polluted water	
Landscape site, including side slopes for easy mowing	
Lighting—should be sufficient to enable a trespasser to be aware of the environment.	

It is recommended that some mechanical mixing and aeration be provided for all lagoon operations. There are several suppliers of excellent floating aerators that provide mixing with a small impeller and aeration by induction along the shaft. These can be low horsepower units located to encourage a low level of circular flow in each cell. This minimizes sludge buildup and materially improves treatment performance.

Final clarification has become common with lagoon operations. Clarification is needed to reduce the algae concentration in the effluent. Clarifier design is the same as that which is used for extended aeration treatment. It is recommended that the surface loading be in the range of 400 to 500 gpd/ft<sup>2</sup>. Disinfection is also required, with the tablet chlorinator being acceptable. Liquid chlorine feeders are also suitable. The use of UV disinfection will eliminate the need for a chlorine contact-vessel and dechlorination.

### Air Requirements

The air requirements for an activated-sludge process include satisfying the applied BOD demand, the endogenous respiration of the biomass demand, the nitrogenous demand (nitrogen demand for oxidizing forms present to nitrate), and providing adequate mixing. There are several commonly used rules for estimating the air requirements. These include: 1 ft<sup>3</sup> of air per gallon of sewage; 500 to 900 ft<sup>3</sup> of air per lb of BOD applied; and with the extended aeration process, 2000 to 3000 ft<sup>3</sup> per lb of applied BOD.

Air volumes are based on standard pressure and temperature (1 atmosphere pressure and 20°C temperature). Blower capacity is rated as “standard cubic feet per minute” (SCFM). A pound mole of air at standard conditions occupies 359 ft<sup>3</sup>. This volume will contain about 5.76 pounds of oxygen. Oxygen transfer equipment used in wastewater treatment gives about 6% to 15% efficiency. This efficiency depends on the type of equipment used, how the air is applied and the driving force (saturation deficit). Thus, 1000 ft<sup>3</sup> of air will supply about 2 lb of oxygen to the aeration tank. Applying 2000 ft<sup>3</sup> of air per pound of BOD represents a potential oxygen transfer of 2.6 lb of oxygen at a transfer efficiency of 8%. If the domestic sewage being treated has a BOD<sub>5</sub> of 200 mg/l and an ammonia content of 30 mg/l, the daily oxygen demand will be:

$$(200)(8.34)(\text{flow, mgd}) = \text{Carbonaceous demand}$$

$$(30)(4.6)(8.34)(\text{flow, mgd}) = \text{Nitrogenous demand}$$

If the flow is 100,000 gpd, the total oxygen demand will be 167 lb (carbonaceous demand) + 115 lb (nitrogenous demand) = 282 lb/day.

Note: mg/l is equivalent to parts per million parts by weight, flow is in mgd and one gallon of water weighs 8.34 lb.

For oxidation to nitrate, 4.6 mg/l of oxygen is required per mg/l of ammonia.

The nitrogenous demand is 69% of the carbonaceous demand in the above example. This is not a trivial per-



centage, and as such, the nitrogenous demand should not be neglected in determining the oxygen demand of a wastewater.  $BOD_5$  is a measure of the carbonaceous oxygen demand during a 5-day incubation period at  $20^\circ\text{C}$ .

An air supply of  $2000\text{ ft}^3/\text{lb}$  of BOD will ensure an oxygen transfer of more than  $2.5\text{ mg/l}$ , if a reasonably efficient diffuser is used. This should be adequate to meet both the carbonaceous and the nitrogenous demands, and adequate mixing of the tank contents (extended-aeration process).

### Final Clarification

Secondary treatment is a two-phase process consisting of the adsorption/absorption-stabilization phase and the solids-separation phase. Final clarifiers are designed to optimize the solids-separation phase and to produce a clarified effluent. Solids separation is accomplished by the gravitational subsidence of the agglomerated sludge particles to the clarifier bottom, where they are removed and recirculated back to the influent end of the aeration reactor or wasted to a sludge-stabilization reactor.

Small clarifiers are generally rectangular with hopper bottoms. The mixed liquor from the aeration reactor is introduced into the clarifier several feet below the surface, generally at mid-depth or lower, and the decant is removed at the surface. The upflow regime is more efficient than an across-the-vessel flow because, with the short-flow path, minor short-circuiting, such as will occur if the weir is not absolutely level, may cause solids to be carried into the effluent before separation occurs. The clarifier generally shares a common wall with the aeration reactor in package plant design. Figure 27.17 shows a typical package plant with the clarifier attached to the aeration tank.

As noted for the primary clarifier, the design parameters include surface loading, weir loading and depth. The

sludge-collecting equipment is eliminated with small clarifiers by using a hopper bottom. An air lift is used to lift the sludge from the hopper to a pipe or trough. The sludge is then returned to the influent of the aeration tank or to sludge waste. The sides of the hopper must be inclined  $60$  degrees or more to the horizontal so sludge will not accumulate on the hopper sides. Where the liquid depth is  $11$  ft and a  $2\text{-ft}^2$  hopper bottom is provided, the surface dimension is limited to  $14$  ft when the inclined sides are extended to the liquid surface.

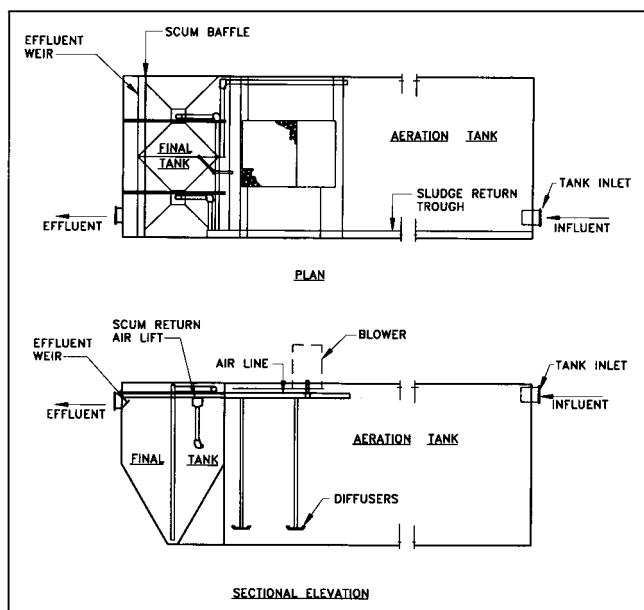
A clarifier having a surface loading of  $600\text{ gpd}/\text{ft}^2$  and surface dimension of  $14\text{-ft}$  has a daily capacity of  $117,600$  gpd. The weir loading, where the weir extends along a  $14\text{-ft}$  side, is  $8400\text{ gpd}/\text{ft}$ . Additional weir length can be included in the design to increase the clarifier capacity.

Grease and other floating substances are not entirely removed in the activated-sludge process. Also, a long sludge age may foster the growth of scum-forming compounds and organisms. Clarifier design should include scum removal equipment, and the effluent weir should be baffled to prevent overflow of scum into the effluent trough. A second air lift may be provided to lift the scum into a pipe leading to the waste-sludge vessel. Larger clarifiers are generally circular and are designed similar to the primary clarifiers shown in Figure 27.12.

### Disinfection

Biological treatment does not eliminate all enteric organisms. Therefore, enteric pathogens are assumed to be always present. The principal three pathogenic forms found in wastewater are bacteria, viruses and amoebic cysts. No routine direct test is made for pathogens, but tests are performed to determine the presence of the enteric coliform group of bacteria, which are always present in sewage in great numbers. If this group has been eliminated, it is assumed that the pathogens have been eliminated.

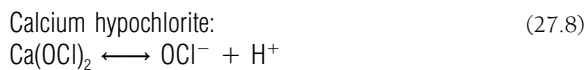
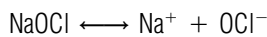
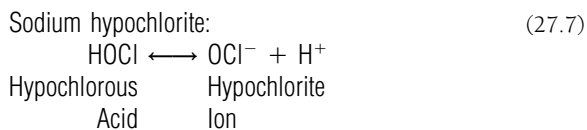
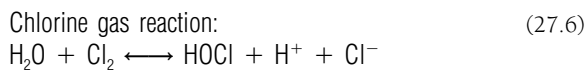
**Chlorine Disinfection.** Disinfection can be accomplished by heat, chemicals or radiation. Wastewater disinfection is usually accomplished by the use of oxidizing chemicals or by radiation. Chlorine is the most common chemical employed for disinfection at larger installations. Chlorine is plentiful, relatively inexpensive and the technology and equipment required for its use are well-developed. Chlorine is available as a liquid, a solid and as a gas. Liquefied gas is available in  $150$  and  $2000\text{-lb}$  containers. Chlorine gas is a very strong oxidizing chemical and very dangerous if not properly handled. The gas form was used as a war gas in World War I. In wastewater treatment, the gas is evaporated off and dissolved in water, with the solution being fed into the wastewater. The water solution is similar to household bleach. Household bleach is a  $5.25\%$  sodium hypochlorite solution that is available in plastic containers. Stronger industrial hypochlorite solutions are available for use at smaller treatment plants. Chemical feed pumps are used to meter the hypochlorite solution into the wastewater. Calcium hypochlorite is available as a tablet or in granular form. The tablet form is fed by means of the tablet chlori-



**FIGURE 27.17** Plug-flow-aeration unit followed by hopper bottom clarifier.

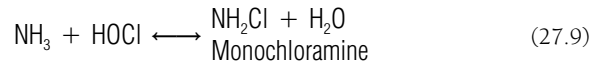
nator as shown in Figure 27.10. Figure 27.18 shows a typical tank-mounted chlorination feed system. Water flow through the eductor is required to produce a vacuum at the feed controller mounted on the 150-lb chlorine cylinder for chlorine to be released from the cylinder.

When chlorine reacts with water, hypochlorous acid or the hypochlorite ion is formed as shown below. All three reactions produce the hypochlorite ion, which in the presence of water will be in equilibrium with the hypochlorous acid form. Chlorine gas tends to decrease the pH, whereas the hypochlorites tend to increase the pH of the water. However, sewage is normally fairly well-buffered, and the amount of chlorine added has little effect on the pH. The equilibrium between the acid and the ion forms depends on the pH of the water, with the acid form predominating at a pH below 6.3. The acid is the most active disinfecting form, but no attempt is made to adjust the pH to optimize disinfection.



Chlorine reacts with most any electron donor to form a chloride. The oxidizing chemical reacts with organic matter in the wastewater, including bacteria and viruses. It also

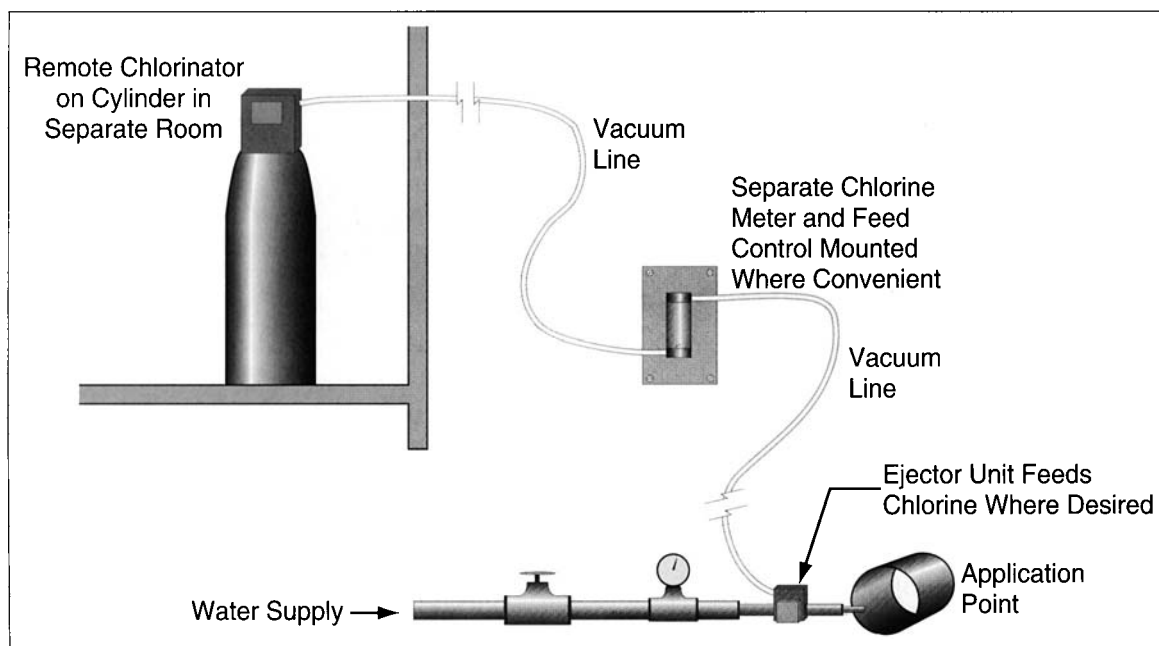
reacts with nitrogen compounds such as ammonia to create a “combined form” of chlorine that has disinfection capability. The first reaction forms a monochloramine.



There is sufficient ammonia in most wastewater to react with all the added chlorine. Therefore, only the combined form of chlorine is normally found in wastewater. If enough chlorine is added, dichloramines and trichloramines (a toxic gas) may be formed. This quantity of chlorine would not normally be added to sewage.

Disinfection with chlorine is a “time-concentration” process. Therefore, the chlorine solution must be well mixed with the wastewater and a contact period must be provided. The combined form is not as strong an oxidizing agent as the free forms. This decrease in effectiveness is overcome by increasing the dosage of chlorine and/or by increasing the contact time. Wastewater disinfection requirements are usually based on a 30-min contact time with a 3.0 mg/l total chlorine residual at the end of the contact period. The chlorine dosage for secondary wastewater is usually about 10 to 12 mg/l to provide the 3.0 mg/l residual. One mg/l is equal to a dosage of 8.34 lbs per 1,000,000 gallons of wastewater.

**Ultraviolet Radiation Disinfection.** Figure 27.19 shows a typical UV disinfection design. Whereas a chemical disinfectant dose is determined by residual concentrations times, a contact time (in minutes), UV dose is determined by UV intensity times exposure time (in seconds). For typical water quality, the equipment provides 8 seconds of contact time in the unit where the UV intensity is greater than 4000 microwatts per cm<sup>2</sup> to give a dose of 32,000 microwatts-



**FIGURE 27.18** Typical vacuum chlorine feed system.

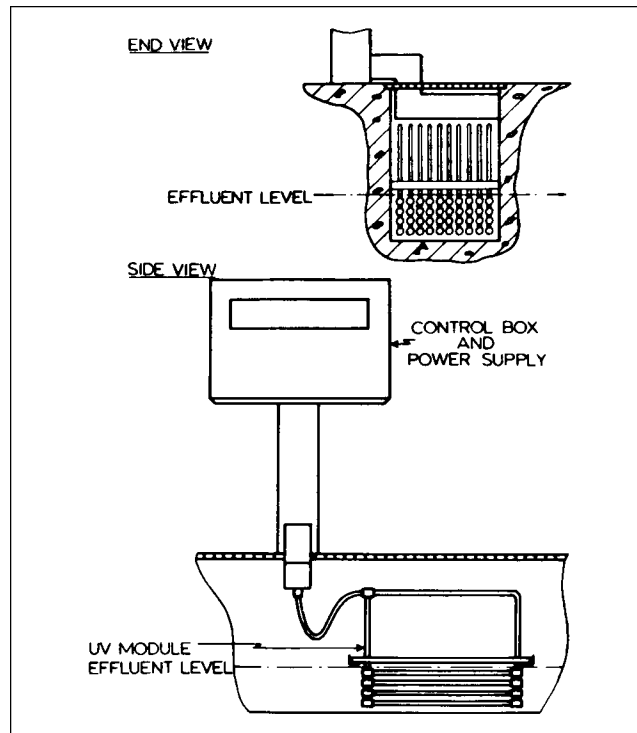


FIGURE 27.19 UV unit-disinfection unit.

seconds per  $\text{cm}^2$ . A quartz tube protects the UV lamps, with the entire assembly being removable for cleaning.

Ultraviolet radiation with conventional UV technology is an effective means of disinfection and is suitable for tertiary and secondary-quality effluents. UV equipment is designed to provide sufficient radiation to accomplish disinfection in a flow-through reactor. This method of disinfection should be considered the preferred means for use at small treatment plants. When UV disinfection is used, sampling for BOD and bacterial counts should be from the equipment discharge. If samples are collected at the plant outfall, the effluent will have been reseeded from the film on the effluent conduit, and this can lead to very misleading test results. This is particularly true for effluents having an ammonia residual, as the reseeded will likely include nitrifiers leading to nitrification in the BOD bottle.

### Process Design

**General.** While the geometric design, piping and operating procedures may vary, the following unit operations and processes are common to all wastewater treatment plants.

- Screening and/or comminution
- Grit removal (not always practiced in small plants except in some plants located near a coast where sand is a problem)
- Flow measurement
- Lift station
- Activated-sludge process

- Clarification with return sludge
- Disinfection
- Sludge wasting

The hydraulic components, such as all piping, pumping, channels or other flow-carrying items, are designed to pass two and a half times the plant design capacity. The hydraulic capacity may be higher if the plant is designed to handle large infiltration and inflow volumes or where the plant is subject to higher-than-normal peak flows associated with the service area. The aeration reactor is designed based on the average daily-design-flow. Clarifiers, contact tanks, etc. must be designed to treat the peak flows, so as to provide the required loading and/or retention time. Figure 27.20 shows a generalized flow diagram for an activated-sludge treatment plant.

**Screening.** The initial screen or comminutor (mechanical shredding equipment designed to chop up large solids) will be similar to the screen section shown in Figure 11.22. Screening is provided to remove large particles that are not amenable to treatment along with the other constituents of the wastewater flow. If a comminutor is provided, a bypass screen is included for use in case of mechanical failure of the comminutor. Screening will remove 1 to 5% of the incoming BOD.

**Flow Measurement.** Flow-measurement location may depend on local or state requirements. An effluent-flow measurement, which is the flow being discharged, is generally required by the permit conditions, but some engineers and operators also like to have a record of incoming flow. The

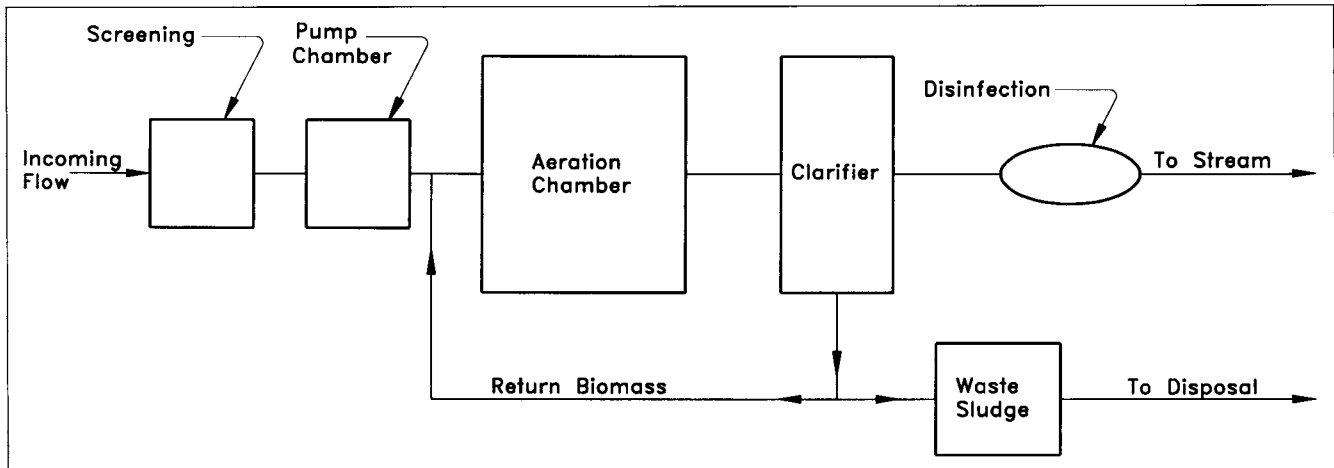


FIGURE 27.20 Typical activated-sludge flow diagram.

designer must use care to insure that no recirculated flow is returned back ahead of the influent-flow measuring equipment. Ultrasonic meters that measure the depth through a control section are preferred. The flow-measurement data is transferred to a totalizer and a chart, with the data being included in the daily records. Figure 27.21 shows a typical installation of an ultrasonic meter in a manhole. The Palmer-Bowlus flume is grouted into the manhole bench, forming the flow through channel.

**Pumps.** Sewage pumps are normally sized to pass two and a half times the average design flow. Submersible pumps are recommended, because at small plants there is generally only one operator present at the plant, and the design should not include any equipment that requires below-ground maintenance. Solid-state frequency controllers for varying the pump rotational speed to match the pumping rate to the incoming flow rate should always be considered. This modification will eliminate surging through the plant that is normally associated with pumps that cycle on and off. Two pumps should always be provided, with each pump having a pumping capacity equal to two and a half times the plant design capacity. The pump design and selection should always provide the required peak-pumping rate to be available even with the largest pump out of service. Automatic alternation of the lead pump is needed to insure that each pump is subject to daily operation. A typical pump installation is shown in Figure 25.19.

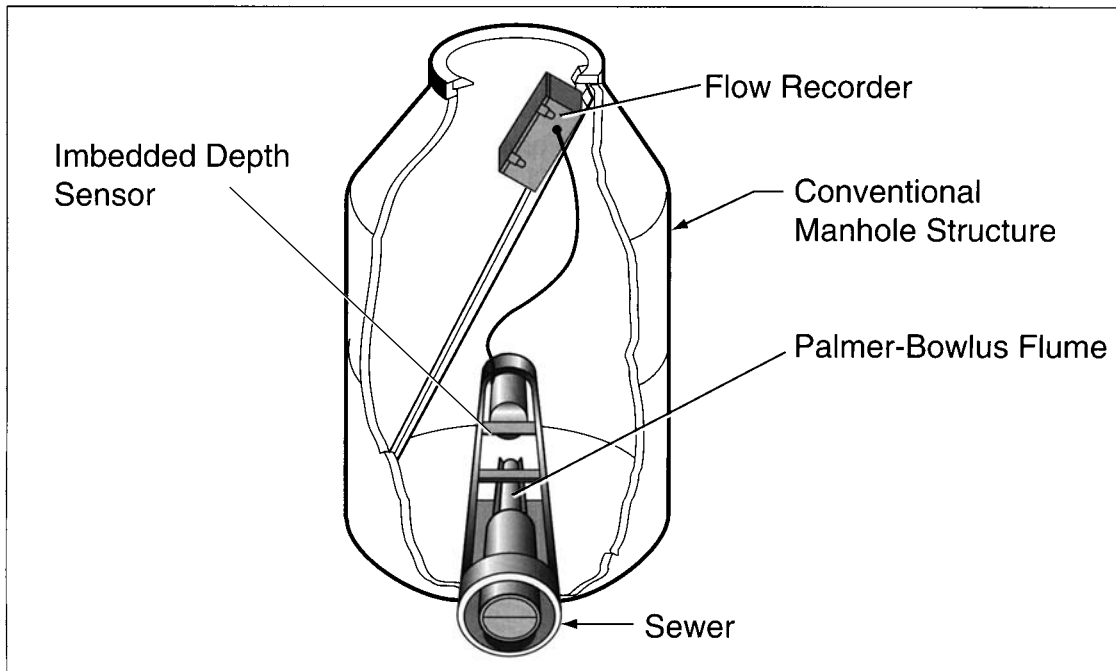
**Activated-Sludge Process.** The extended aeration process is designed to provide 24 hr of aeration. At plants having a capacity of less than 40,000 gpd, one aeration chamber is usually provided. Where only one tank is provided, maintenance of all components must be possible without dewatering the tank. This means that the diffusers must be mounted for removal and replacement from above the water surface. Plug-flow tanks are generally long, rectangular chambers where there is little longitudinal mixing. The flow is injected at one end of the tank and moves through the tank to the effluent end. The return sludge-flow is mixed with the incoming flow at or near the entrance to the aer-

ation tank. The aeration is achieved by passing compressed air through diffusers located approximately 1 ft off the bottom of the tank. Normally, the diffusers are located across the tank bottom in rows transverse to the direction of wastewater flow. This alignment reduces short-circuiting and provides for good mixing of the contents. Table 27.10 provides a summary of design criteria for the activated-sludge process treating domestic sewage.

Aeration is required in order to provide mixing of the tank contents, which keeps the biomass in contact with the wastewater and allows for the transfer of oxygen to the flow. Oxygen is transferred from the rising bubble surfaces as well as from the surface renewal that results from the pumping action of the rising bubbles. The smaller the bubbles, the greater the surface area of transfer (per unit volume of air introduced). Fine or medium-bubble diffusers are recommended because of the more efficient oxygen transfer and because the mixing pattern is superior to that of coarse-bubble diffusers. Manufacturers' literature indicates that some of the fine-bubble diffusers have a transfer efficiency of 15 to 20%. The aeration equipment should be installed in accordance with catalogue data.

Compressed air may be provided by either a centrifugal or positive-displacement blower. Small plants usually use the positive-displacement blower because of cost. The positive-displacement blower will give satisfactory service, but the air-pressure vibrations associated with the operation results in a high noise factor. The noise is exacerbated at higher rotational speed. The engineer should specify a rotational speed of not greater than 1160 rpm.

Where the design flow is greater than 40,000 gpd, dual aeration tanks should be provided with each having one-half the plant design capacity. Conventional blowers will provide air at up to about 8 psi pressure. This is equivalent to 18.5 ft of water pressure. Oxygen transfer is related to the travel distance and size of the air bubble. Aeration tanks should be about 16 ft deep to obtain the greatest efficiency from the air-compression operation. However, since most small plants are fabricated and painted before shipment to



**FIGURE 27.21** Ultrasonic flow meter installation.

the site, highway restrictions limit the height and width of the tanks to 12 ft by 12 ft. This size of tank would have at least 1 ft of freeboard, making a 40,000-gallon tank 40 ft long. The diffusers would be placed about 1 ft off the tank bottom, giving a 10-foot submergence depth. Plants providing capacities up to about 50,000 gpd are available as factory assembled and painted units. Two units will give a 100,000-gpd design capacity.

Plants having a capacity greater than 100,000 gpd are usually field-erected. Package-fabricated and field-erected

plants are often circular for capacities greater than 50,000 gpd. The plants are shipped complete and ready for field-assembly. A concrete base is poured in place with the factory-supplied embedded-section installed. The tank sections are erected by field-welding or bolting. The circular shape is selected for structural strength and to minimize piping and pumping of the flow. The clarifier is located at the center of the unit. The aeration basin, waste-sludge-stabilization basin, and chlorine-contact basin are generally located in the annular area outside the clarifier. Air-lift

**TABLE 27.10** Summary of Design Criteria for the Activated-Sludge Processes

PARAMETER	CONVENTIONAL	EXTENDED AERATION	CONTACT STABILIZATION	OXIDATION DITCH
F/M (lb/day)/lb MLVSS	0.2–0.4	0.05–0.15	0.2–0.6	0.03–0.10
Sludge age	5–15	20–30	6–12	30+
MLSS in reactor	1500–3000	3000–5000	1200–3000	3000+
Sludge recycle ratio	0.25–1.0	0.75–1.5	0.25–1.0	0.25–1.5
Aeration time, hours	4–8, 6 normal	24–36	0.5	24+
Air, SCFM/lb BOD removed	500–1000	3000+	800–1200	N/A
MLVSS/MLSS	0.6–0.8	0.6–0.7	0.6–0.8	0.6–0.7

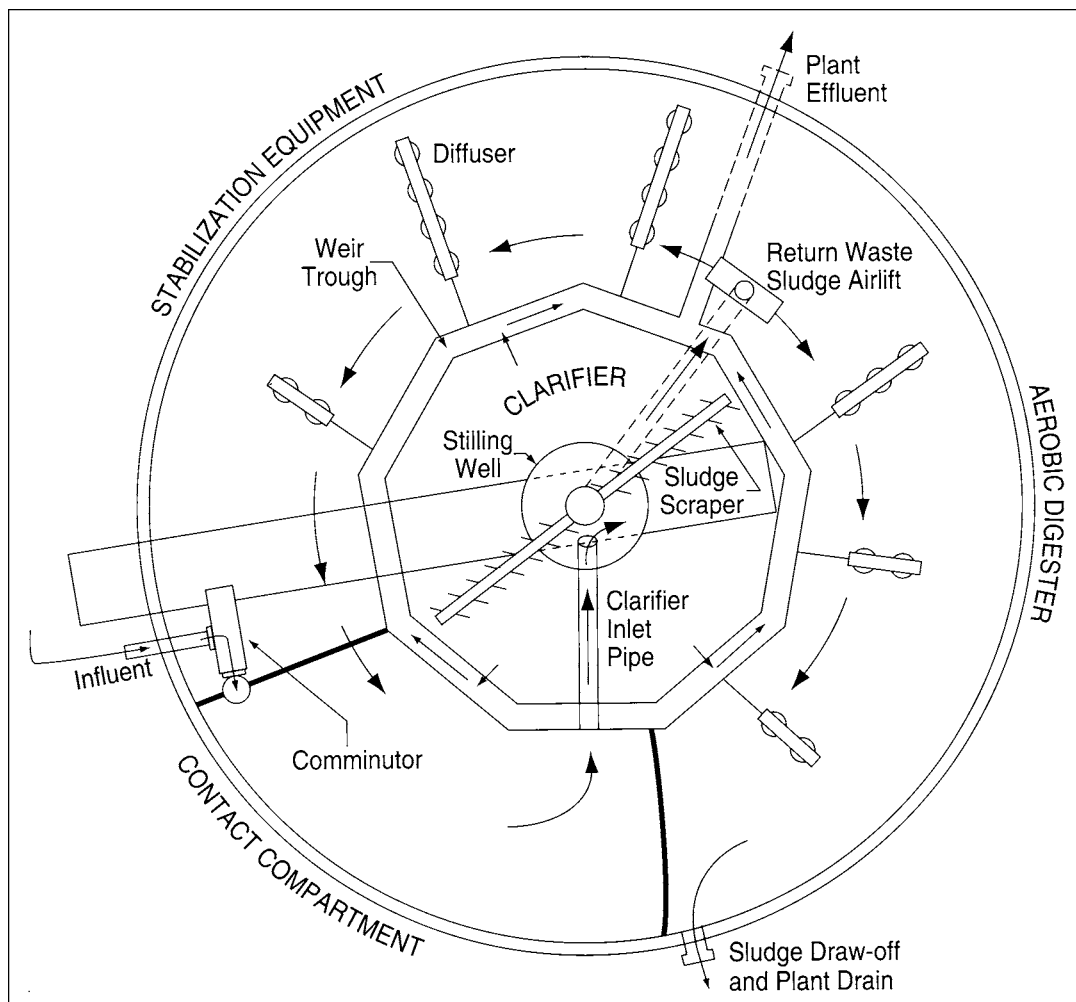
Note: all the secondary processes should remove more than 88% of the applied BOD. Regulations require that the BOD of the effluent be less than 30 mg/l. Most well-operated, secondary treatments routinely produce an effluent having a BOD<sub>5</sub> between 10 and 15 mg/l.

pumps are used to transfer flow from one compartment to another. Figure 27.22 shows a typical layout. The design shown is a contact-stabilization process where a 30-min contact period is provided (aeration of mixed liquor). The mixed liquor is then piped to the clarifier. The clarifier underflow of concentrated sludge is air-lifted to the stabilization compartment, where it is aerated for 4 to 6 hr before being returned to the inlet of the aeration basin where it is mixed with the incoming flow. The time periods used are based on plant-design flow. All treatment processes are completed within the single unit except for disinfection. Some designs also include a chlorine-contact tank. UV disinfection should always be considered for small plants. The requirement for a contact tank and the need to store and handle chlorine are eliminated when UV disinfection is used. The sidewall-depth of field-erected tanks will generally be about 15 ft. See section on the "Batch Reactor" design, as a common fabricated plant.

**Clarifier Design.** A clarifier having a surface area of less than about 200 ft<sup>2</sup> can be a hopper bottom design, as shown in Figure 27.17. Larger clarifiers, as shown in Figure

27.12-b, must be designed to accommodate available equipment. The sludge-collecting equipment, such as the drive and access bridge, must be selected from a supplier's catalog. The diameter is determined by the selected surface loading. It is suggested that a surface loading of 500 to 600 gpd/ft<sup>2</sup> be used. The depth will normally be 7 to 8 ft. The selected equipment will determine the requirements for installing the inlet piping, effluent weirs, and sludge-collecting equipment. The sludge recycling pumps and piping should have a capacity for returning from 25 to 150% of the plant-design capacity. Scum from the clarifier should be piped to the sludge-stabilization basin.

**Disinfection.** As noted previously, UV radiation should always be considered as a suitable disinfection means for small plants. This selection will eliminate the need for a chlorine-contact tank, but more importantly, the need for storage and handling of chlorine will be eliminated. The potential hazards associated with handling chlorine are a particular concern at the small plant, where only one operator may be present. Another concern with chlorine, at the small plant, is the need to adjust the chlorine feed rate



**FIGURE 27.22** Typical layout for circular tank.

as the flow varies. The flow rate varies significantly at all plants during the 24-hr diurnal period, but the percentage variation is greater for the small plant. Serious overdosing of chlorine will occur if equipment is not provided to adjust the feed rate automatically as the flow rate varies.

There is a minor potential hazard from UV light exposure to the eyes if precaution is not exercised, but a trained operator should not subject himself to this hazard. The engineer should select proven equipment that does not require major disassembly for cleaning. Grease and slime accumulate on the tubes covering the lamps, reducing the UV light transmission. A strong phosphate detergent is required for tube cleaning.

**Waste-Sludge (Biosolids) Stabilization.** Some local authorities may permit sludge from an extended-aeration process to be disposed on land without additional stabilization. Normally, all waste-activated sludge is subjected to additional stabilization by aerobic digestion. Stabilized sludge is a humus material that has a musty odor. It also may contain pathogens. When applied to land, soil texture and moisture-retaining capacity are improved. The sludge is also a low-level source of nutrients. The primary operation cost of aerobic digestion is the power costs for aeration and mixing. Self-cleaning, medium-bubble diffusers are suitable for use in aerobic digesters.

Stabilization provides a reduction in the volatile suspended solids (VSS) and improves sludge compactability. The digester contents are consolidated by turning off the air and allowing the sludge to settle for 1 to 1.5 hr, after which the supernatant is decanted from the digester and returned to the aeration tank influent. The daily volume of sludge to be wasted from the clarifier return-sludge piping will depend on the solids content of the sludge. At a sludge yield of 0.5 and a plant influent BOD of 200 mg/l, the solids content of the waste sludge will be 834 lb/million gallons of sewage flow. If the solids can be transferred from the bottom of the clarifier as 1.5% solids (1.5 lb solids per 100 lbs of sludge), the daily volume wasted to the digester will be  $(834/0.015)/8.34$  lb/gal = 6667 gallons. If 50% of the VSS are destroyed in the stabilization process, the solids destroyed per million gallons of sewage flow will be  $(834 \text{ lb/day}) (0.7 \text{ VSS}) (0.5 \text{ destroyed}) = 292 \text{ lb/day}$ . The solids remaining for disposal =  $834 - 292 = 542 \text{ lb/day}$ . The stabilized solids should consolidate to a 2 to 3% sludge. At 2.5% solids, the accumulation rate will be  $= (542 \text{ lb}/0.025)/8.34 \text{ lb/gal} = 2599$  gallons per day, for a 39% reduction in volume during the stabilization process  $((6667 \text{ gal} - 2599 \text{ gal})/6667 = 0.61$  or 61% accumulation rate). The supernatant return will be approximately  $6667 \text{ gal} - 2599 \text{ gal} = 4068 \text{ gal}$ . The average sludge-accumulation rate will be  $(6667 + 2599)/2 = 4068 \text{ gal/day}$ . The air supply must provide adequate mixing of the digester contents and transfer the oxygen needed for aerobic stabilization.

Air requirements based on oxygen transfer:

oxygen requirements = 2 lb oxygen per lb VSS destroyed

Lb of solids destroyed per day = 292 per million gallons of sewage treated

Lb oxygen to be transferred per day =  $(2)(292) = 584$

At 10% transfer efficiency—5840 lb oxygen to be supplied

Air is 20% oxygen; therefore, air requirements =  $5840/0.20 = 29,200 \text{ lb/day}$

One lb mole of air = 28.8 lb; air requirement is  $29,200/28.8 = 1014$  moles

One lb mole =  $359 \text{ ft}^3$ , air volume =  $(1014)(359) = 364,026 \text{ ft}^3/\text{day} = 253 \text{ SCFM}/\text{million gallons of sewage treated}$

Air requirement based on mixing =  $25 \text{ SCFM}/1000 \text{ ft}^3$

Digester volume = 20-day retention at 4068 gpd =  $81,360 \text{ gal} = 10,877 \text{ ft}^3$

Air supply for mixing =  $(10,877/1000)(25) = 272 \text{ SCFM}$  per million gallons of sewage treated

The air requirement for mixing exceeds the oxygen-transfer needs; therefore, the mixing requirements should be supplied. It should be noted that some diffuser designs are more than 10% efficient.

Stabilized sludge (biosolids) is a valuable resource that should be recycled back to the land where possible. Hauling and spreading of liquid sludge is the most convenient means of disposal where this option is available. Additional on-site storage volume may be needed at locations where there are periods when the spreading equipment can not go onto the land. When sludge is applied to land, BOD, nitrogen, and phosphorus may be carried to surface streams by runoff during and following rainfall.

When it is not feasible to spread the liquid sludge, on-site air-drying beds can be provided. Dried sludge is usually taken to the local landfill for disposal. Sludge from domestic sewage should not contain any constituents that would make the sludge unsuitable for land application. The design parameters for aerobic digestion at domestic sewage treatment plants are summarized in Table 27.11. Design para-

**TABLE 27.11 Design Parameter for Aerobic Digestion**

Hydraulic retention period, days	15–20
Oxygen requirements, lb/lb VSS destroyed	2
Air requirements for mixing, SCFM/1000 ft <sup>3</sup>	20–30

meters will vary, depending on local winter temperatures. The engineer should be familiar with local requirements.

The engineer may design any size of plant to be constructed of reinforced concrete in place. The size where the cast-in-place plant becomes economical or preferred will depend on local conditions. Some owners will prefer a "constructed-in-place" plant for a 30,000-gpd capacity, while others may prefer the lower first cost of a steel plant for even much larger treatment capacities. Either selection should give satisfactory service. However, the engineer is able to incorporate a more tailored design into the cast-in-place plant. The "constructed-in-place" concrete plant is not economical for installations having a capacity less than a few hundred thousand gallons per day.

The flow regime and the type of equipment included in the design is predetermined for the pre-engineered package plant. However, the engineer has a variety of configurations to choose from among the designs offered by the various suppliers. Generally, the package plant will not only cost less than the tailored design, but it can be erected in less time. Additionally, steel tanks have a shorter life and require more maintenance than concrete tanks.

Several options for the design of a secondary-treatment plant are presented herein. The batch reactor is one viable choice that is well-proven over years of use. Many other options are also available to the engineer, and more will certainly be developed. The National Small Flow Clearinghouse is a good source of current information (see Table 27.7).

## TERTIARY TREATMENT

Three common forms of wastewater treatment beyond the normal secondary level include: filtration to reduce the BOD and suspended solids concentration; nitrification to reduce the nitrogenous oxygen demand; and, nitrogen and/or phosphorus removal to reduce the level of these nutrients in the effluent. The batch reactor available from the Sanitaire Corporation and shown in Figure 27.16 has been modified to accomplish both nitrogen and phosphoric removal. Other equipment suppliers also offer equipment suitable for use in treating small flows.

### Filtration

The BOD of a secondary effluent is composed of both dissolved and suspended matter. Direct filtration will reduce the suspended-matter concentration but will have little effect on the dissolved BOD. The ratio of suspended solids to BOD for a good-quality, secondary effluent will generally be between 1.5 and 2.0. Most of the BOD will be associated with the suspended solids. Normal granular media filtration will reduce the suspended solids to a 4 to 6 mg/l range and the BOD to between 5 and 8 mg/l. Chemical coagulation, flocculation and sedimentation followed by filtration can reduce the suspended solids to about 2 mg/l and the BOD to a 3 to 5 mg/l range.

Well-designed and operated secondary plants will routinely produce effluents having a BOD and suspended solids less than 10 mg/l. However, small plants often do not receive the level of process operation and facility maintenance found at larger plants, and excursions even higher than 30 mg/l may be common. Filtration should always be provided where permit effluent limits for BOD and suspended solids are less than 20 mg/l. This additional polishing operation should be evaluated for plants having effluent limits between 20 and 30 mg/l. Quality design and operation are required to meet an effluent limit of 20 mg/l on a continuing basis with secondary treatment alone. Chemical coagulation, flocculation and sedimentation followed by filtration should be considered where the plant effluent BOD and suspended solids must be reduced to below a 7 to 10 mg/l range.

Down-flow, granular-media filtration of the type employed for potable-water filtration is the most common technology used for wastewater polishing. This technology is well-developed and will give reliable performance (the engineer should be sure to specify a proven technology). Down-flow, granular-media filters are of two broad types: deep bed and shallow bed. Both types are classified as rapid sand filters. Conventional deep-bed filters contain 2 to 4 ft of filtering media supported by 1 to 1.5 ft of gravel for a total bed depth of between 3 and 5 ft. Some package units employ an underdrain and backwash manifold that do not require the gravel support. The filtering media may be silica sand or a combination of silica sand and anthracite coal (dual media), or a combination of sand, anthracite coal and a more dense material such as garnet (tri media).

Important characteristics of the filtering media include the porosity of the bed and the size of the voids between the media particles. Therefore, to control the media particle size and the uniformity of the particles is desirable. The more uniform the particle size, the greater the bed porosity. Since it is not possible to achieve a condition where every particle is the same size, it would be desirable to have the larger particles at the top and the smaller particles at the bottom of the filter bed. However, sand filters are cleaned by backwashing, where water is distributed uniformly over the filter bottom at a sufficient flow rate to expand the bed and provide adequate agitation and scouring to remove the solids that were filtered out of the wastewater from the bed. When the bed is expanded, the filter media is suspended by the upflowing water. This expansion and suspension result in a vertical stratification of particles based on particle size where all the particles have the same specific gravity. The particles are pulled down by gravity and are carried up by the rising water in relation to the surface area of the particle. The smaller particle has a greater surface area per unit volume. Thus, the backwashing of the filter stratifies the filter media according to particle size, with the smaller at the top and the largest at the bottom. This is opposite



to what is desired because the larger voids are at the bottom of the bed and not at the top.

The dual-media bed partly overcomes the problem of reverse stratification. By using two media materials having different specific gravity and selecting the proper particle size, the media having the smaller specific gravity will stratify at the top of the bed, and the more dense media will form the bottom layer. However, each of the two media will still be stratified in reverse to what is desired. Dual-media beds are composed of anthracite coal, which has a specific gravity of about 1.5, and silica sand, which has a specific gravity of 2.65. The anthracite can have a larger particle size, providing larger pores, which increases the bed's capacity for removing solids. Tri-media beds include a third media such as garnet, which has a specific gravity of about 4.0. Characteristics of filter media are shown in Table 27.12.

The dual-media, deep-bed filter is commonly used for direct filtration of wastewater. The anthracite, having a larger particle size and hence larger voids, will provide greater solids storage. The sand of smaller size forms a barrier that retards breakthrough of solids with the filter effluent. Filters are normally sized for a filtration rate of between 2 and 5 gpm/ft<sup>2</sup>. A filter sized to operate at 2.5 gpm/ft<sup>2</sup> at average design flow should not exceed a 5.0 gpm/ft<sup>2</sup> rate during peak flow periods. A filter designed for an average filtering rate of 2.5 gpm/ft<sup>2</sup> requires a surface area of 28 ft<sup>2</sup> per 100,000 gpd capacity.

During the filtering operation, solids are removed from the flow and stored within the filter bed. These solids are washed deeper into the bed as filtering continues and will eventually begin to appear in the filter effluent. Filters are backwashed to remove the collected solids when the head loss through the filter has reached a predetermined value (normally 8 ft for a conventional filter) or when there is a breakthrough of solids within the filter effluent. A run time between backwashing, when filtering a secondary effluent having suspended solids of 20 mg/l, will normally be about one day. The most important design aspect of a filter is the backwashing operation. It is necessary that the backwash water be applied uniformly over the entire filter bottom at a rate that will produce the required bed expansion and

scouring needed to remove the captured solids. Backwash rates are between 15 and 20 gpm/ft<sup>2</sup>. If the backwash water is not applied uniformly over the filter bottom, the media will be deposited in the areas where the flow is the lowest and the unexpanded areas will become compacted and ineffective for filtration. Inadequate application of backwash flow has been found to be a shortcoming with some package filters. If the bed expansion is insufficient to provide the necessary scour, the bed will not be cleaned and it will become necessary to backwash the filter again in a short time. The turbulence created by the high upflow rate of the backwash water scours the organic solids from the more dense silica sand and carries the organics from the filter.

It is becoming more common to incorporate compressed air as a part of the backwash program. Air will provide greater agitation and scouring, improving the removal of solids while reducing the water requirements and the backwash period. Air should always be considered as a part of the backwash procedure. Air is applied prior to the start of the water flow and continued through part of the period of water backwashing. However, the application of air is discontinued before completion of the backwash cycle so the bed will properly stratify with the water flow. Air is used both as a surface wash to break up the media surface and as a part of the flow to the bottom of the media.

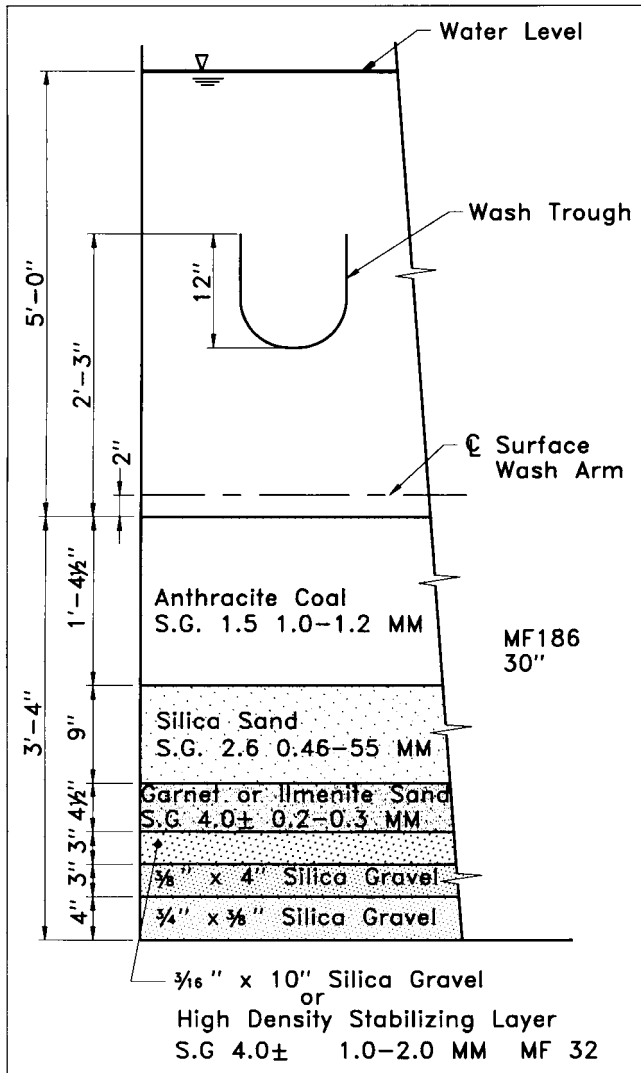
Figure 27.23 shows an elevation-view of a deep-bed, tri-media filter of a type that is available as a package unit. Note that there is 5 ft of water over the media during the filtering period and 27 in. from the top of the bed to the top of the backwash trough. This 27 in. is needed to allow for the bed expansion during backwash so as to prevent media loss. Design provisions should be made for a minimum of 15 min of backwash. A 28 ft<sup>2</sup>-filter washed at 20 gpm/ft<sup>2</sup> for 15 min will require 8400 gallons of backwash water (about 8–10% of the filtered water).

A conventional deep-bed filtration operation requires the following:

- A source of previously-filtered backwash water is needed. The chlorine-contact tank is a good source of backwash water because the chlorine residual in the water will retard the growth of slimes on the filter media. The design should also provide for adding chlo-

**TABLE 27.12 Filter Media Characteristics**

<b>MEDIA</b>	<b>SPECIFIC GRAVITY</b>	<b>EFFECTIVE SIZE</b>	<b>UNIFORMITY COEFFICIENT</b>
Silica sand	2.65	0.50–0.55 mm	<1.65
Anthracite coal	1.50	1.0–1.2 mm	<1.65
Garnet	4.0	0.2–0.3 mm	<1.65



**FIGURE 27.23** Section view of a tri-media filter. Reproduced with permission from Wheelabrator Engineered Systems, Sturbridge, Mass.

rine ahead of the filter as needed for this purpose. If a chlorine-contact tank supply is not available, a separate chamber must be provided to store backwash water. The volume stored should be sufficient to wash at least two filters.

- A receiving tank for the backwash water is needed because the high backwash flow rate cannot be piped directly back to the secondary clarifier or other points in the treatment train. The backwash water should only need clarification before filtering and disinfection. It is not necessary to return this hydraulic load to the headworks of the plant.

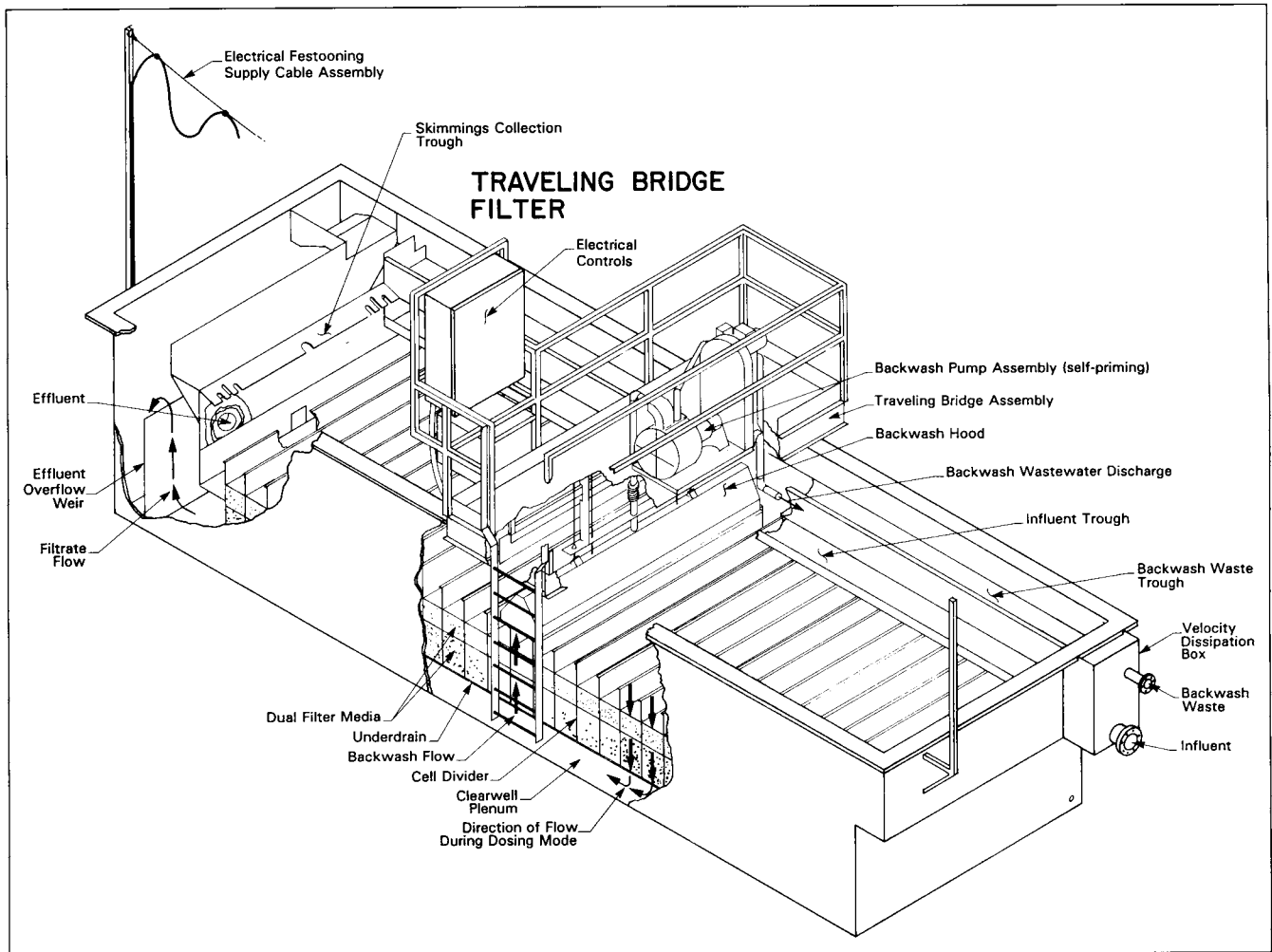
The backwash flow can be piped to a holding tank where the solids will settle to the bottom of the tank, permitting the decanting of most of the flow directly back to the filter influent, with the tank underflow containing the solids re-

moved from the filter being returned to the secondary clarifier influent at a flow rate that will not overload the operation.

At plants that are subject to frequent periods of poor secondary effluents, a polymer can be added to the clarifier influent to improve solids capture in the clarifier. A high solids loading to the filter will shorten filter run time between backwashing. This period can be shortened to where sufficient water cannot be filtered to supply the backwash needs. The addition of a polymer to the clarifier influent will not increase sludge volume while materially improving solids capture. However, polymers should not be added on a continuous basis, but only on the infrequent occasion when needed because of a biological process upset. Polymer addition to the filter influent will not be very effective because of the low solids concentration in the flow. The filtration process will be improved by adding a flow equalization basin between the secondary clarifier and the filter. This additional basin should be sized to equalize the diurnal flow variation that will provide a means of providing a constant filtration rate.

Another type of filter used in wastewater treatment is the "traveling bridge filter." This is a shallow-bed filter, having a media depth of 12 to 15 in. and generally a single filter media. The filter bed is divided into cells. A schematic view of a traveling bridge filter is shown in Figure 27.24. The traveling bridge carries a backwash pump and hood along the filter. As the hood passes over a cell formed between two adjacent partition plates, the pump on the bridge draws water from a plenum underdrain up through the sand, where it is discharged into a trough located along a wall on one side of the filter. The pump and hood are designed to provide the proper upflow rate uniformly over the area being backwashed. Since only a single cell is being backwashed at a time, the backwash flow rate is relatively low, and the flow can usually be returned directly to the influent of the secondary clarifier. Since the water for backwashing comes from the underflow plenum, the cells not being backwashed supply the backwash water for the cell being cleaned. The operation requirements of the traveling bridge filter make it an option that should be considered for a small plant. Filtration rates are similar to those for the deep-bed filter. Bed-load capacity is less than that expected from the deep-bed filter because of the shallow depth and the use of a single media. Therefore, the frequency of cell backwashing is shortened.

There are several purveyors of well-developed package filters, both dual and tri-media deep beds and the traveling bridge filter. The engineer is well advised to consider a package-unit design for small plants. The package units are generally piped and valved to provide automatic operation. Design and on-site construction of filter units is not economical for small plants. Catalog data for the selected unit should be used for incorporating the unit(s) into the overall



**FIGURE 27.24** Sectional view of a travelling bridge filter. Reprinted with permission from Davis, Water & Waste Industries, Inc., Thomasville, Ga.

design. Filters should be housed in a building at locations where freezing weather occurs in the winter months.

A pilot study would normally be conducted at larger plants to obtain design data, but studies of this type are not feasible for small plants. The engineer should be conservative in a filter design for a small plant. When the effluent limits are less than about 8 mg/l for BOD and suspended solids, chemical coagulation, flocculation and sedimentation should precede filtration. The process chain is the same as used for water treatment, and package units providing the entire operation from chemical addition through filtration in a single unit are available and should be used at small plants. A backwash water supply and receiving tank are needed to complete the operation. In addition, provisions must be included in the design for handling the sludge generated.

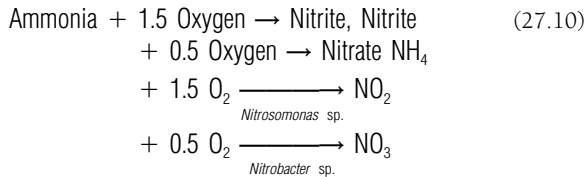
Filtration is not a means of correcting for poor operation of secondary treatment. If poor-quality secondary effluent is applied to a filter, the filtering system will fail.

### Nitrogen and Phosphorus Control

As noted earlier in this chapter, nitrogen and phosphorus are generally the growth-limiting nutrients in aquatic environments. Phosphorus is most often the growth-limiting element in fresh water and nitrogen in marine environments. Secondary effluents from plants treating domestic sewage will normally contain 20 to 40 mg/l of total nitrogen and 8 to 15 mg/l of total phosphorus. Normally, the total nitrogen will be primarily in the ammonia or nitrate form and the phosphorus will be present as phosphate. These forms are readily available to aquatic flora and can contribute to the eutrophication of lakes, ponds and estuaries.

**Nitrogen Oxidation and Removal.** Ammonia is an intermediate compound, which results from the decomposition of protein. Mammals, including man, eliminate most excess nitrogen from the body via the urinary pathway in the form of urea. Urea is rapidly hydrolyzed after leaving the body, releasing ammonia. Ammonia is common to all domestic

wastewater. Ammonia is the reduced form of inorganic nitrogen in surface waters. Ammonia should be oxidized to the nitrate form because in its reduced form it places an oxygen demand on the receiving water. Additionally, ammonia is toxic to many forms of aquatic life, even in low concentrations. The activated-sludge biomass generally contains both heterotrophic and autotrophic bacteria. The nitrifying autotrophs absorb ammonia and oxidize it to nitrates according to the following sequential reactions.

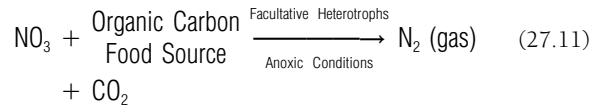


Two species of autotrophic bacteria are needed to complete the oxidation process. Both species are more sensitive to temperature and pH than the heterotrophic species. Nitrobacter is more sensitive to low temperature than nitrosomonas. It is not uncommon during winter months to find relatively high concentrations of nitrites in a secondary effluent. Nitrites are subject to chemical oxidation and will express a high chlorine demand when present in the effluent. When ammonia is expressed as nitrogen, 4.6 mg/l of oxygen is required to oxidize 1 mg/l of ammonia (64/14). Thus, an ammonia concentration of 25 mg/l requires 115 mg/l of oxygen to complete the oxidation. The engineer must provide sufficient oxygen to meet the nitrogenous demand.

Heterotrophic bacteria, which use carbon-based organic material for food and growth, have a high cell-yield and undergo rapid growth when fed. In contrast, the autotrophic bacteria use inorganic substances for growth and, therefore, have a much slower growth rate and cell-yield. In conventional activated sludge, the autotrophs are maintained by carrying a long sludge age. Even under these conditions, the autotrophs may be overgrown and unable to compete in the environment. The autotrophs prefer surfaces for growth, and nitrification can be improved by providing surfaces for maintaining the population. The surfaces should be located after the uptake phase for the heterotrophs has occurred. Plastic plates may be placed vertically near the effluent end of the aeration tank to provide growth surface for the nitrifiers. Air must be supplied to the area for mixing and to provide an oxygen supply. It may be impossible to achieve nitrification in small plants during winter months if the liquid temperature drops below 15°C. It is not uncommon for the NPDES permit not to require nitrification during winter months for plants located in northern latitudes.

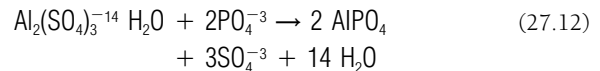
The most suitable means of nitrogen removal is a biological process called denitrification. In an anoxic environment (absence of free oxygen, but not anaerobic where reduction reactions occur), the facultative biomass will use oxygen from compounds such as nitrates as an electron

acceptor. The nitrates are not reduced back to ammonia but to nitrogen gas, as shown by the following generalized reaction.



There are several biological processes for accomplishing the above reaction as a part of an activated-sludge process. The batch reactor is one example. Older designs use a separate operation for denitrification. An energy source is required for the bacteria. The organic carbon can be supplied by raw sewage. The contact vessel is mechanically stirred to keep the biomass in suspension and in contact with the food source. The mixing releases the nitrogen gas to the atmosphere. When a separate operation is selected, the additional piping and tankage requirements, together with additional operational requirements, may represent a major burden for small treatment plants.

**Phosphorus Removal.** Phosphorus can readily be removed from wastewater by chemical precipitation, using alum, sodium aluminate, lime or iron salts. Alum is the most common physical-chemical process employed for phosphorus removal in wastewater treatment. When aluminum is used, phosphates are precipitated as aluminum phosphate according to the following reaction.



It is generally necessary to add 15 to 20 pounds of alum per pound of phosphorus present. Actual chemical requirements should be determined by laboratory tests on the wastewater to be treated. Some package plants incorporate biological phosphorus removal capability (see section on the batch reactor). At plants where an anoxic condition is created for nitrate reduction, the phosphorus is precipitated as elemental phosphorus. Thus, the anoxic process is effective in removing both nitrogen and phosphorus. Several equipment suppliers offer package units that can be operated to remove both phosphoric and nitrogen as a part of secondary treatment.

At plants employing primary treatment, alum or iron salts may be added ahead of the primary clarifier to remove phosphorus as a part of the primary-sludge volume.

## REFERENCES

- Davis Water and Wastes Industry, *Municipal and Industrial Water and Wastewater*, Product Catalog, Davis Water and Wastes Industries, 1828 Metcalf Avenue, Thomasville, Ga., 31792.
- Metcalf & Eddy, Inc., *Wastewater Engineering*, McGraw-Hill, New York, 1972.
- Multi-Flow Waste Treatment Systems, *Engineering and Technical Manual*, Multi-Flo Waste Treatment Systems, Inc., 2324 East River Road, Dayton, Ohio, 45439.

Prince William County, *Sewer Standards and Design Manual*, Prince William County Service Authority, P.O. Box 2266, Woodbridge, Va., 22193-0266.

Sanitaire Water Pollution Corporation, *Sewage Treatment Equipment*, Catalog, Sanitaire Water Pollution Control Corporation, P.O. Box 744, Milwaukee, Wisc., 53201.

Sawyer, Clair N., and Perry L. McCarty, *Chemistry For Sanitary Engineers*, McGraw-Hill, New York, 1967.

Steel, Ernest W., *Water Supply and Sewerage*, McGraw-Hill, New York, 1960.

Trojan Technologies, *UV Disinfection System*, Catalog., Trojan Technologies, Inc., 845 Consortium Court, London, Ontario, Canada, N6E 2S8.

U.S. Environmental Protection Agency, *Wastewater Treatment Facilities For Sewered Small Communities*, U.S. Environmental Protection Agency, Washington, D.C., 1977.

U.S. Environmental Protection Agency, *Design Manual—On-Site Wastewater Treatment and Disposal Systems*, U.S. Environmental Protection Agency, Washington, D.C., 1980.

U.S. Environmental Protection Agency, *Innovative and Alternative Technology Assessment Manual*, U.S. Environmental Protection Agency, Washington, D.C., 1980.

Viessman, Warren, Jr., and Mark J. Hammer, *Water Supply and Pollution Control*, International Textbook Company, Scranton, Pa., 1971.

Virginia Department of Health, *Alternative Discharging Sewage Treatment System Regulations for Individual Single-Family Dwellings*, Bureau of Sewage and Water Services, Virginia Department of Health, Richmond, Va.

Worthington Group, *Public Works Manual*, McGraw-Edison Company, 5310 Taneytown Pike, Taneytown, M., 21787.

## CHAPTER 28

# WATER SUPPLY AND TREATMENT

Gary L. Nickerson, P.E.

### SOURCE AND TREATMENT CONSIDERATIONS

The consulting engineer serving the private land developer client often need only coordinate with an established water utility company to provide potable-water service for a potential land development project. In that instance, he is required to design the distribution system for the development to the guidelines and standards of the utility. The previous chapter largely describes the planning and design of water-distribution systems. This chapter focuses on those developments that are to be located where there is no established potable-water system. The consultant must then determine how to establish sufficient water sources, implement required treatment facilities, and engineer the distribution system. The principles of water supply and treatment, as discussed in this chapter, apply to large and small systems. However, this discussion will be slanted more toward small systems, as the majority of land development projects serve less than 3000 people or have fewer than 1000 connections (the U.S. EPA definition of a small system).

In evaluating available water-supply sources, the primary objective is to obtain a source (or sources) that has acceptable sustainable quantity, the best available quality, and that is conveniently and economically located near the anticipated point of use. Most small, independent water-supply systems in the United States use ground water as their source. Typically, ground-water sources are less difficult and costly to develop and operate than surface-water sources. However, discussion of surface-water sources is included in this chapter for those instances where suitable ground water is not available.

### DRINKING WATER REGULATIONS

A residential or commercial land development site can be supplied drinking water from a public or private water utility, or from a supply system designed only for that particular development. The water source can be surface water (lake, river, or stream), ground water (wells, springs), or even ground water under the influence of surface water. Note for the purposes of treatment, the regulatory agencies consider ground water under the influence of surface water to be surface water. In addition, all state agencies administering the Safe Drinking Water Act (SDWA) require discharge permits for wastestream decant returned to surface waters. These state agencies can provide detailed guidance on their requirements.

#### The Safe Drinking Water Act

The Safe Drinking Water Act (SDWA) is the primary legislation affecting public drinking-water utilities in the United States. It was passed into legislation in 1974 and was re-authorized and modified by Congress in 1986 and 1996. The regulations issued by the U.S. EPA in response to this legislation continually evolve to address public health and water-quality issues. The primary role of the federal government under the SDWA is to develop national drinking water regulations to protect public health. The states have the responsibility of implementing and enforcing regulations for the public water systems under their jurisdictions. State and local regulatory agencies in some instances have water-quality standards that are more stringent than those of the SDWA. The private consultant must develop water-supply designs that not only adhere to the

SDWA regulations but also those of the state and local jurisdictions.

A community public-water supply, as defined under the SDWA, is one that serves a year-round residential population of 25 or more persons (or 15 residential units). This definition is important in that SDWA requirements are geared toward community public-water supply systems that serve a population of people that regularly drink the water over an extended period. The SDWA requires that community public water systems be periodically sampled and tested for maximum contaminate limits (MCL) and for bacteriological, organic, and inorganic contaminants.

### SDWA Rules

The 1974 SDWA established interim drinking-water standards that have been revised and finalized. After the 1986 re-authorization of the SDWA, revisions and additions have taken the form of rules. There are a number of rules that have been finalized and are now part of the requirements, as follows:

*Total coliform.* This rule revises the basic bacteriological monitoring requirements by providing instructions on provisions that must be taken after a positive test result. It also gives instructions for determining when the bacteriological monitoring MCL is exceeded.

*Surface water treatment.* This rule requires that water from surface sources receive filtration treatment. Additionally, it establishes disinfectant concentration time (CT) calculation requirements to ensure deactivation of *Giardia*, as well as other pathogens, through sufficient contact time. State regulatory primacy agencies (state agencies regulating on behalf of the EPA) can provide details of the requirements.

*Phase I, II, V chemical monitoring.* Many organic and inorganic chemicals are included on the list of required maximum contaminate limits (MCL) by these rules. Many pesticides, herbicides, and volatile organic chemicals are included in the MCL monitoring list.

*Lead and copper.* This rule is meant to protect the water consumer from corrosive water supplies that leach lead and copper from plumbing fixtures. A number of required samples, based on population served, are taken from residential fixtures that have been turned off for at least 6 hours. Action to reduce the corrosivity of the water is required if 10% of the samples have copper and lead content of 1.3 mg/l and 15 micrograms per liter ( $\mu\text{g/l}$ ), respectively.

### Proposed SWDA Rules

The previously discussed SDWA rules have become part of the law, but there are other rules that are under study or in the process of being finalized, as follows:

*Enhanced Surface Water Treatment Rule (ESWTR).* The ESWTR will set more rigorous standards for removal or inactivation of pathogens in drinking water, particularly *Cryptosporidium*. Continuous monitoring of filter effluent, as well as lower turbidity levels, will probably be required. This rule deals with severe short duration (acute) health risks, since the effects of pathogens are immediate.

*Disinfectants/Disinfection Byproducts Rule (D/DBPR).* This rule is designed to protect the public against the long-term, or chronic, effects of exposure to disinfection by-products, especially those created by the use of chlorine for disinfection. This rule will probably lower the allowable limits for trihalomethanes and set limits for other disinfection by-products.

*Ground Water Disinfection Rule.* It is likely that this rule will require all public community ground water systems to be disinfected.

*Arsenic Rule.* If enacted, this rule will set a lower allowable limit for arsenic in community public water supply systems. The existing MCL for this well-known poison and carcinogen is 50 mg/l, originally established in 1975. The proposed MCL is 10 mg/l which will place many ground water and some surface water systems in non-compliance without appropriate modifications.

*Radionuclides and radon.* The proposed radionuclides rule will revise the radium standards for drinking water and add a regulation addressing uranium. There is a proposed separate rule to address radon in water. Radon's principal danger to water consumers is from inhalation when this radioactive becomes airborne (e.g., in a bathroom shower). The primary danger of inhalation of radon gas is the development of lung cancer, while ingestion of uranium and radium can lead to bone and digestive-tract cancer.

### GROUND WATER SUPPLY

Within a development, it is not uncommon for the water supply of an individual homeowner to come from a well, nor is it uncommon for a single well to serve many houses. Using a ground water supply presents problems different from those of a conventional public system, the basic problems being whether sufficient water is available and whether the ground water is of potable standards. Additionally, once the ground water source is identified as usable, measures must be exercised to keep the source from becoming polluted.

While ground water is available nearly everywhere, the difficulty for recovering ground water, because of the depth of the water table, may prove costly. In a development where the same aquifer potentially will be used by numerous households, assurances must exist that there is sufficient water yield for everyone tapped into it and that

natural replenishment of the aquifer is compatible with the amount used.

### Ground Water

Ground water exists in two major subsurface zones, the boundary of which is the water table. Above the water table, the pore spaces of the aeration (or unsaturated) zone are filled with air and water. The high fluctuation in moisture content in this zone is affected by such things as temperature, humidity, precipitation, vegetation, and the intrinsic properties of the porous medium. Below the aeration zone (and the water table) lies the saturation zone. Here, the pore spaces contain only water, which is available for consumptive use. Although the shape of the water table generally follows the ground surface, its depth fluctuates seasonally and is affected in part by climate and geologic conditions.

An aquifer is a water-bearing geologic formation capable of storing, transmitting, and providing fresh water in usable quantities. In order to do this, the geologic formation must have interconnecting pores and open spaces of sufficient size to allow the movement of water. The open space within the aquifer, occupied by ground water, may only consist of small interstices between loose grains or may include large geologic cavities. As shown in Figure 28.1, there are two basic types of aquifers: an unconfined aquifer and a confined aquifer.

The unconfined aquifer has the water table as its upper boundary and an impermeable layer as the lower boundary. Water pumped from a well in an unconfined aquifer lowers

the water table near the well. Gravity then causes ground water to flow towards the well. Replenishment results from the surface waters in the overlying area that infiltrate deep enough into the unsaturated zone and eventually end up in the aquifer through lateral movement.

Ground water in a confined aquifer is bounded between two layers of impermeable material. Because of such confinement, the overburden and atmospheric pressure creates a pressure head that causes water in a well, placed in that aquifer, to rise above its upper boundary. Occasionally, when a well penetrates the confined aquifer, the pressure head is enough to cause the water to rise above the ground surface. This is known as an artesian well. It is important to note that the replenishment (recharge) area for a confined aquifer is not the overlying surface, because of the impermeable upper layer. The rate of ground water movement is highly variable and can range from several feet per year to several feet per day, depending on the soil and rock characteristics. This movement results from the combined effects of gravitational forces and the hydraulic gradient. The hydraulic gradient is the head loss per unit length of flow path. Darcy's equation states that the velocity of water through a porous medium is proportional to the hydraulic gradient, or in equation form:

$$v = Ki \quad (28.1)$$

where  $v$ , the discharge velocity for laminar flow conditions, is the quantity of water flowing in unit time through a unit

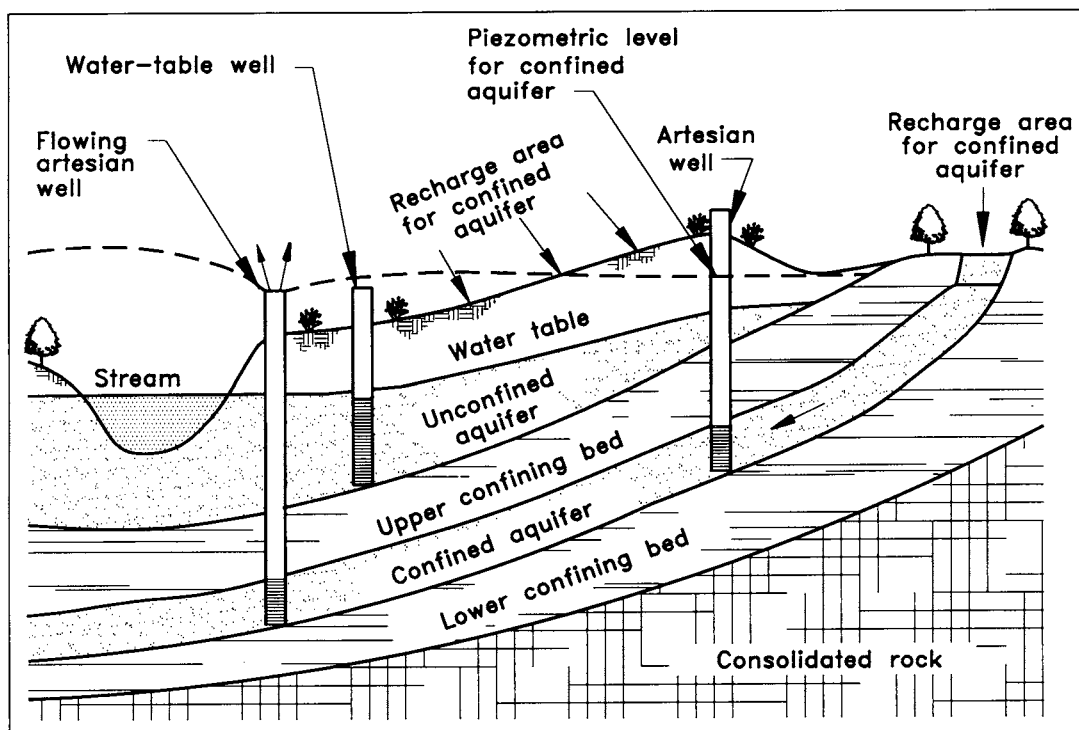


FIGURE 28.1 Types of aquifers.



gross cross-sectional area of soil at right angles to the direction of flow (fps).  $K$  is a proportionality constant (also if fps) referred to as the hydraulic conductivity, and  $i$  is the hydraulic gradient in feet per foot ( $i = h/L$ ). Combining Darcy's equation with the continuity equation gives the flow rate through a porous medium:

$$Q = KA \left( \frac{h}{L} \right) \quad (28.2)$$

where  $Q$  = flow (ft<sup>3</sup>/sec). Figure 28.2 identifies the terms used in Eq. (28.2) and shows the relationship of hydraulic gradient to hydraulic head using the concepts presented in Eq. (26.15). Note: the direction of flow is controlled by the hydraulic gradient (i.e., right to left) and not by what gravity might indicate (left to right). The underlying assumptions for Darcy's law are that inertial forces are negligible, flow is laminar, steady, and nearly uniform. The first two assumptions can be combined by the further restricting that the Reynolds number must be between 1 and 10.

Porosity and hydraulic conductivity affect ground-water movement rates. Porosity, the total volume of pore space within a given volume, is a measure of the medium's ability to hold a volume of water. Hydraulic conductivity is the relative ease with which water moves through the pores under a hydraulic gradient. The hydraulic conductivity,  $K$ , depends on the physical properties of the porous media and the fluid. In terms of fluid and media characteristics,  $K$  is expressed as

$$K = \frac{Cd\rho g}{\mu} \quad (28.3)$$

where  $d$  = mean grain diameter,  $\rho$  = fluid density,  $\mu$  = absolute viscosity of the fluid,  $g$  = gravitational accelera-

tion, and  $C$  is a dimensionless constant that is a function of grain size, sphericity, roundness, and the nature of the grain.

Table 28.1 and Table 28.2 identify values of hydraulic conductivity and porosity, respectively, for various soil types. Conceivably, a soil can have a high porosity and yet have a low permeability rate.

Although the total volume of the ground water is equivalent to the volume of the voids in the saturated zone, the actual amount of ground water available for use (i.e., specific yield) is less because of unavailable water that is semi-permanently attached to the surface of the soil and rock aggregates through surface retention. Specific yield is a function of particle size, shape and distribution of pore space, and degree of compaction of soil.

### Transmissivity and Cone of Depression

Many problems with developments using wells are caused by the underestimation of available ground water for the proposed number of dwelling units. If more wells are attached to the same aquifer, and if the increased pumping is beyond the natural recharge rate of the aquifer, draw-down of the water table will result. Furthermore, the draw-down affects the shape of the water table around the well.

Transmissivity is a measure of the entire aquifer's ability to transmit water. Transmissivity is the flow rate (in gpd) moving through the entire saturated thickness of an aquifer having a unit width under a unit hydraulic gradient. Mathematically, transmissivity is expressed as:

$$T = Kb \quad \text{for confined aquifers} \quad (28.4)$$

$$T = K\bar{D} \quad \text{for unconfined aquifers}$$

where  $T$  = transmissivity,  $K$  = hydraulic conductivity,  $b$  = saturated thickness in the confined aquifer, and  $\bar{D}$  = average saturated thickness of the unconfined aquifer.

The shape of the water table surface changes as water is pumped from the unconfined aquifer. In fact, the water table surface converges at the well intake and tends to resume the natural water table elevation as the distance away from the well increases. The resulting shape, an inverted cone, is referred to as the cone of depression (see Figure 28.3). The difference between the natural water table elevation and the depressed water surface is the drawdown. The shape of the cone depends on how far the well penetrates the aquifer, the length of the screen intake, the pumping rate, transmissivity of the aquifer, and other factors. All else being equal, the lower the transmissivity, the more pronounced the cone of depression. Low transmissivity causes a deep cone, with a small base and steep sides, whereas high transmissivity in the aquifer creates a shallow cone, with a large base and flat-sided slopes for the same pumping rate.

The well discharge rate is affected by both the radius of the cone and the drawdown height at the well casing. The

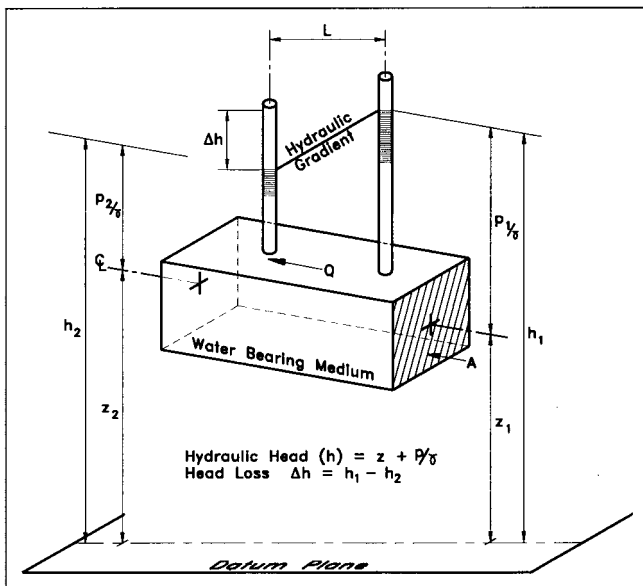
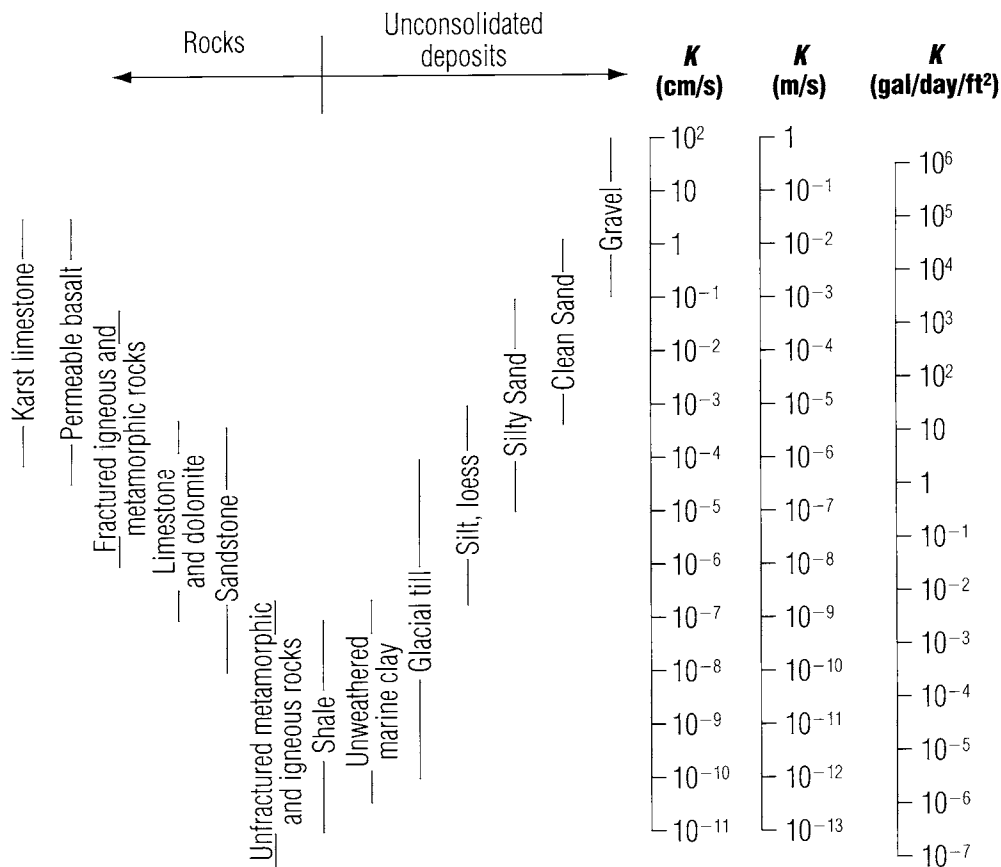


FIGURE 28.2 Darcy's Law.

**TABLE 28.1 Range of Values of Hydraulic Conductivity**



Courtesy of Freeze/Cherry, *Groundwater* ©. Reprinted by permission of Pearson Education, Inc., Upper Saddle River, NJ.

**TABLE 28.2 Range in Porosity for Selected Earth Materials**

MATERIAL	POROSITY (%)
Gravel, clean-uniform	20–50
Sand, clean-uniform	20–40
Sand and gravel mixed	10–30
Salt and clay	
As deposited	40–70
Compacted	0–40
Shale	1–20
Sandstone	5–30
Limestone	1–20

Courtesy of Freeze/Cherry, *Groundwater*, © 1979, p. 37. Reprinted by permission of Prentice Hall, Englewood Cliffs, New Jersey.  
 Courtesy of Freeze/Cherry, *Groundwater*, ©. Reprinted by permission of Pearson Education, Inc., Upper Saddle River, NJ.

well discharge equations for equilibrium conditions are based on the following assumptions:

- Aquifer material has uniform permeability within the radius of influence of the well, and the aquifer is not stratified.
- For an unconfined aquifer, the saturated thickness is constant, and for a confined aquifer, the aquifer thickness is constant before pumping commences.
- The water table or the piezometric surface is horizontal.
- The well extends to the bottom of the aquifer and the pumping efficiency is 100%.
- Laminar flow exists within the radius of influence of the well.
- Steady-state conditions exist, or in other words, the shape of the cone of depression remains constant, in that both drawdown and radius of influence of the well do not change with a constant pumping rate.

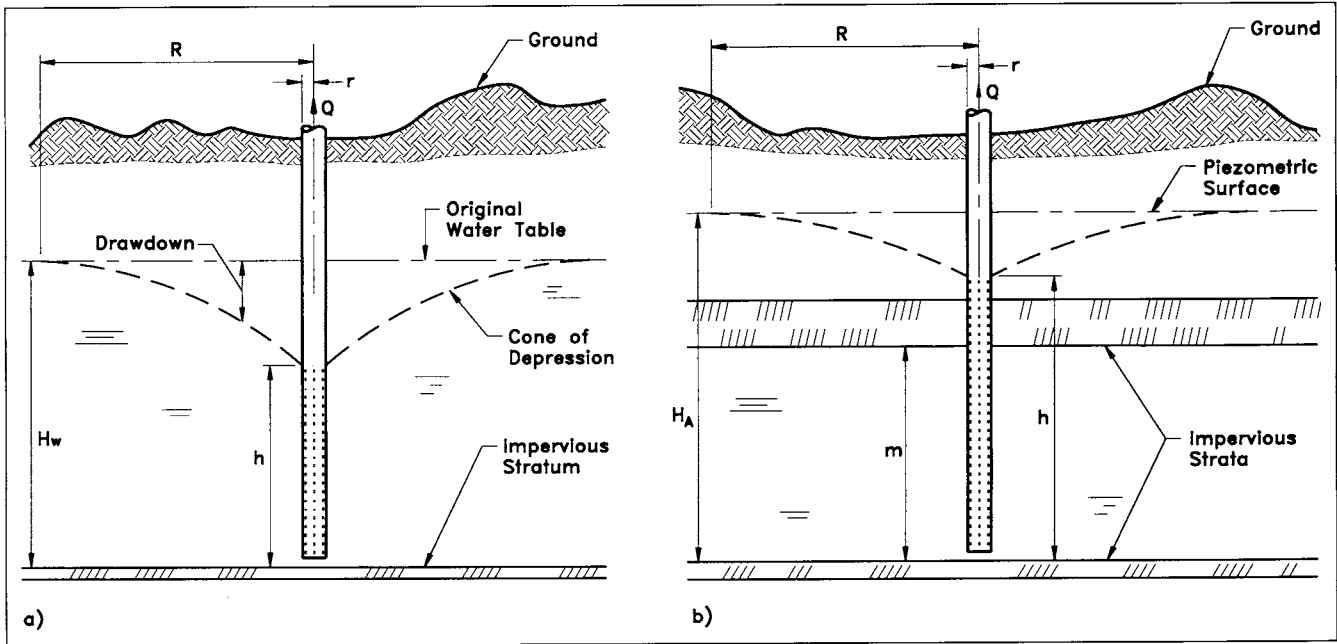


FIGURE 28.3 Radial flow to an aquifer; (a) unconfined flow; (b) confined flow.

From these assumptions, the following equations provide well discharge for steady unconfined radial flow and confined radial flow, respectively, toward a well:

$$Q = \frac{P(H_w^2 - h^2)}{1055 \log(R/r)} \quad (28.5)$$

$$Q = \frac{Pm(H_A - h)}{528 \log(R/r)} \quad (28.6)$$

where  $Q$  = well yield or pumping rate in gpm,  $P$  = permeability of water bearing sand in  $\text{gpd}/\text{ft}^2$ ; typically determined from laboratory or field tests,  $H_w$  = saturated thickness of the aquifer before pumping in feet,  $h$  = depth of water in the well while pumping in feet,  $R$  = radius of cone of depression in feet,  $r$  = radius of the well,  $m$  = thickness of aquifer and  $H_A$  = static head at the bottom of the aquifer.

Equations (28.4) and (28.5) are for steady-state flow, that is, where discharge is constant with time. Because of the numerous factors affecting ground-water flow, a long time may have to elapse before steady-state conditions are established. Therefore, the actual size and shape of the cone of depression can only be obtained after sustained periods of continuous pumping. Because of the variable composition of soils and climatic conditions, the actual time to steady-state flow is highly variable and may never exist. Additionally, several other factors affect well discharge, including proximity to other wells, presence of localized impermeable strata impeding flow to the well, anisotropic conditions leading to highly variable hydraulic conductivity, and large bodies of surface water.

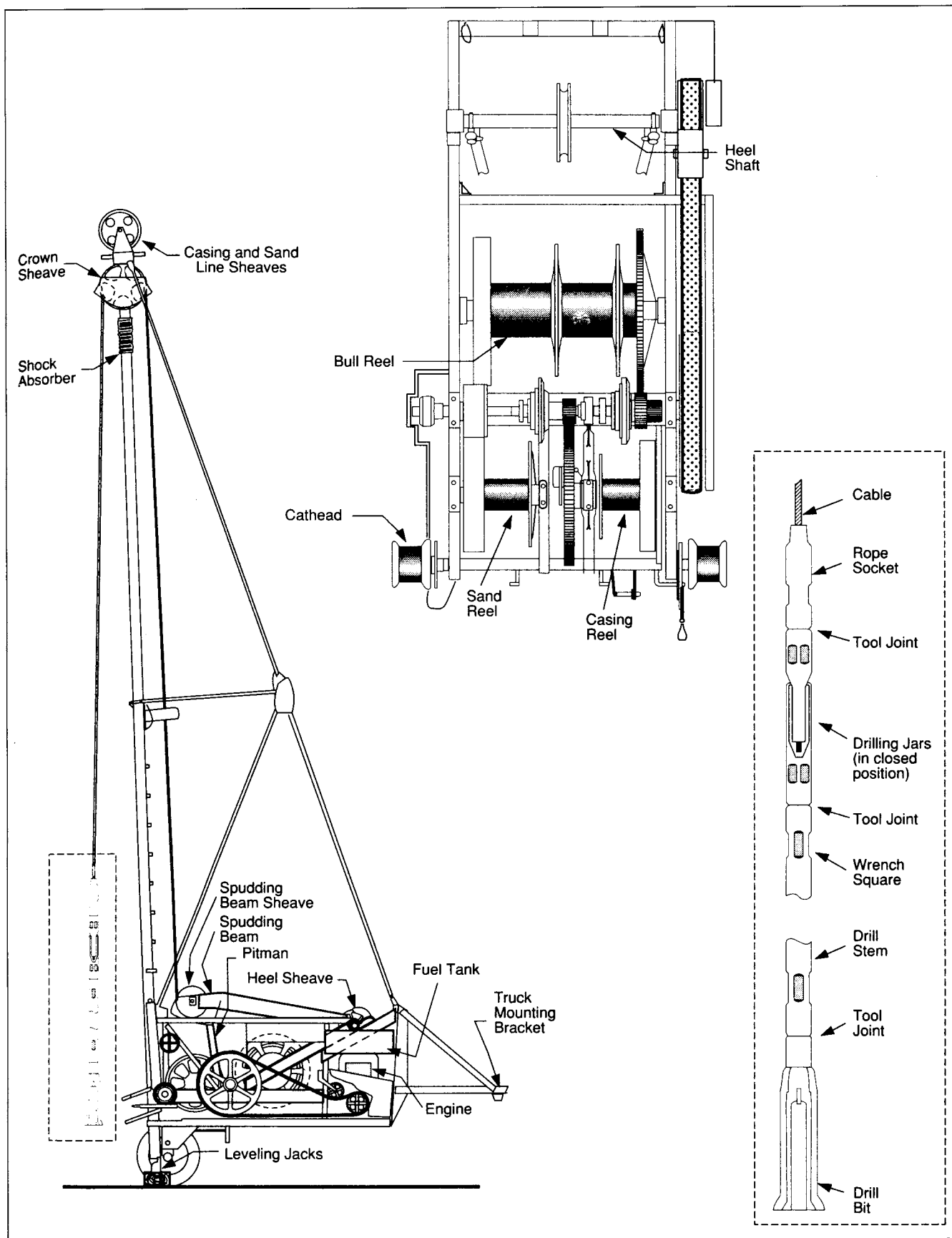
### Well Construction

Most wells are drilled or in some cases, driven. There are two basic types of drilling methods used in well construction: cable-tool and rotary. However, other methods are variations of these basic methods.

**Driven Wells.** Driven wells are constructed by driving a small-diameter pipe into shallow water-bearing sand, gravel, or other soft formations free from cobbles and boulders. A screened well point, attached at the bottom of the casing, acts as the intake portion. There are different types of well points to accommodate the various ground and aquifer conditions. In this type of method, selection of the appropriate well point is important to the success of the well. Ground water enters through the screen section and is pumped up through the casing. Although driven wells are simple and economical to construct, they are limited to depths of about 50 ft.

**The Cable Tool Method.** Sometimes referred to as the percussion method, this is the oldest method of well construction after hand digging. The method utilizes the action of lifting and dropping heavy drilling tools onto the bottom of the borehole in order to break and loosen the earth-rock material. The type of cable-tool rig (see Figure 28.4) and the size of the tool weight needed for well construction depends on the ground conditions and the depth of the hole.

As the drill bit breaks through the ground, the fragments of earth and rock must be removed so that the excavation can continue. The material is removed with a bailer—a section of pipe with a valve at the bottom and a bail at the top. As the bail is alternately raised and lowered, the valve is opened and closed allowing the cuttings and fluid to



**FIGURE 28.4** Cable-tool rig. Courtesy of *Handbook of Ground Water Development*, Roscoe Moss Co. Reprinted by permission of John Wiley & Sons, Inc.

enter the bailer. The bailer is hoisted to the surface once it becomes full and is then reinserted if additional cutting removal is necessary. Drilling operations resume after the bailing operation is complete. The well casing is then inserted into the hole after the drilling operations are complete.

**The Rotary-Drilling Method.** This method bores the hole with a rapidly rotating bit. An external force, which is applied to the bit as it is rotating, helps break the earth-rock material. Fluid, which is pumped into the borehole through the tool assembly, mixes with the cuttings and is pumped back to the surface through the annulus between the tool assembly and the sides of the borehole. A typical rotary-drilling rig is shown in Figure 28.5.

Cable-tool drilling is less expensive than rotary drilling, lends itself to drilling in isolated areas, has more reliable formation and water sampling, and the resulting well is simpler, more durable, and virtually maintenance-free. However, the limitations of this method, compared with the rotary-drilling method, include shallower well depths, smaller well diameters, and a slower drilling time. The rotary-drilling method produces an open borehole, which allows for a wider variety of casing and screen designs. Each method has its merits, and the one selected likely will depend on geologic conditions, site accessibility and availability of supplies.

### Well Components

Figure 28.6 illustrates the various elements of a well structure.

The well casing is the structural portion of the hole that supports the sides of the borehole, provides protection of the aquifer against contamination, and allows access to the aquifer. The casing must last for the life of the well, since replacing the casing inevitably causes some damage to the well.

Grout sealant is placed between the casing and the sides of the borehole to control vertical water movement and to protect the casing against deterioration.

The well intake for drilled wells is the screen and for driven wells is a well point. It is important to note that the length of the screen influences the capacity of the well far more than the diameter. This is demonstrated by Eq. (28.7), a form of the steady-state equation for the drawdown of an aquifer.

$$Q = \frac{Kbs}{528 \log \left( \frac{R}{r} \right)} \quad (28.7)$$

where:

$Q$  = discharge (gpm)

$s$  = aquifer drawdown (ft)

$b$  = saturated aquifer thickness (assumed equal to screen length) (ft)

$K$  = hydraulic conductivity (gpd/ft<sup>2</sup>)

$r$  = effective well radius (ft)

$R$  = radius of influence (ft) (distance away the center where drawdown is zero)

All things being equal, doubling the diameter of the well ( $r$ ) results in  $\approx 10\%$  increase in discharge. However, doubling the length of the screen intake directly results in a twofold increase in the discharge. The type of screen selected depends on the type of drilling, aquifer characteristics, and hydraulic characteristics. Various types of screens are shown in Figure 28.7. In general, the screen length requirements for domestic wells are much less than those for municipal, commercial, or agricultural wells.

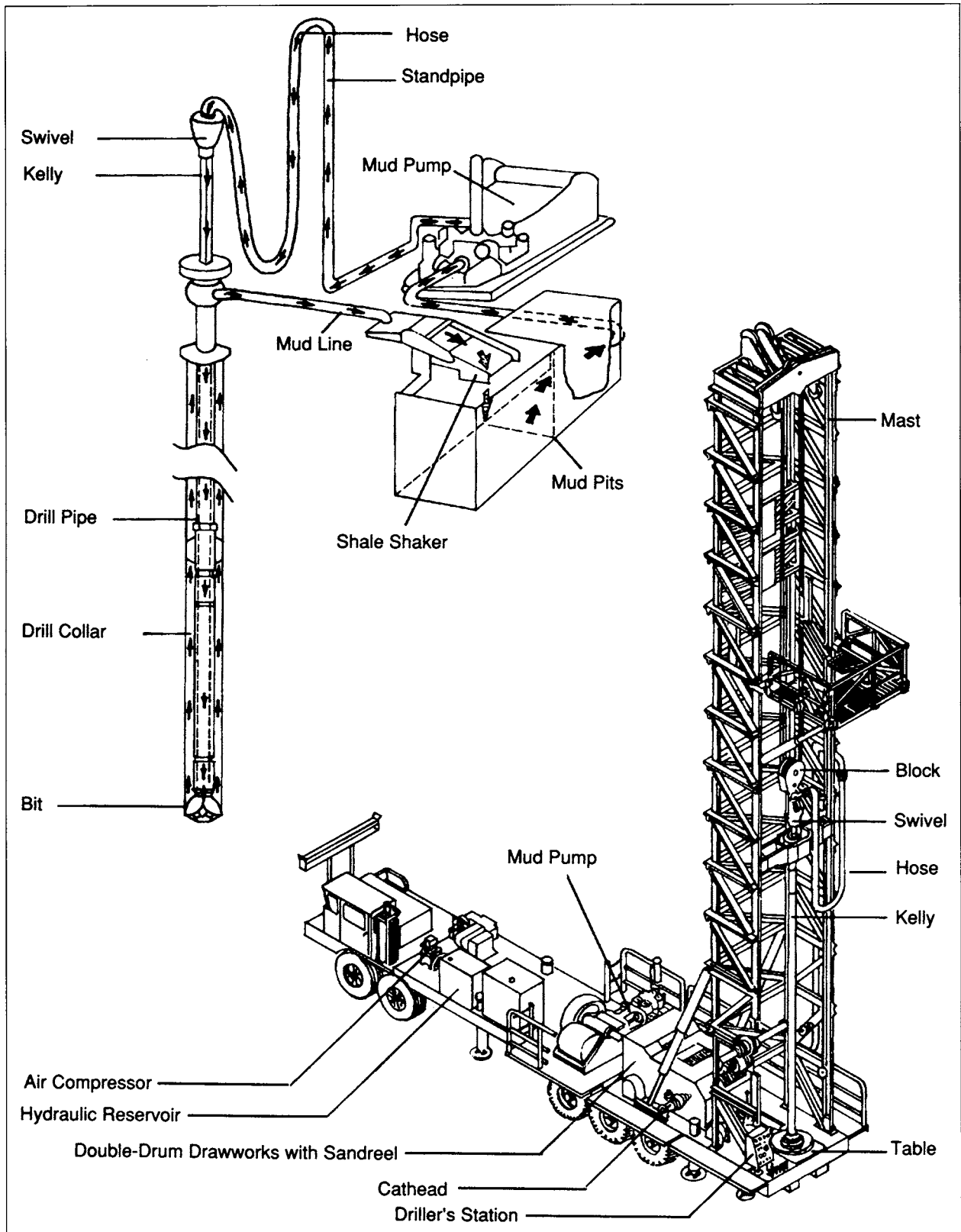
The well pump must be able to deliver the required pumping rate to meet the demand for the service area of the well. Typical individual domestic wells require approximately 5 to 20 gpm. Wells that serve a community of residences should provide a minimum of 0.5 gpm/dwelling, if storage is included in the design. If the aquifer is to serve large areas, auxiliary pumps may be necessary to maintain minimum pressures throughout the system. After the well is operating, a series of tests should be performed to determine the performance of the pump and well. To ensure proper performance of the well, periodic tests are necessary. The data from these tests, when compared with the original test data, can determine if the performance and efficiency falls within acceptable standards.

### Ground Water Contamination

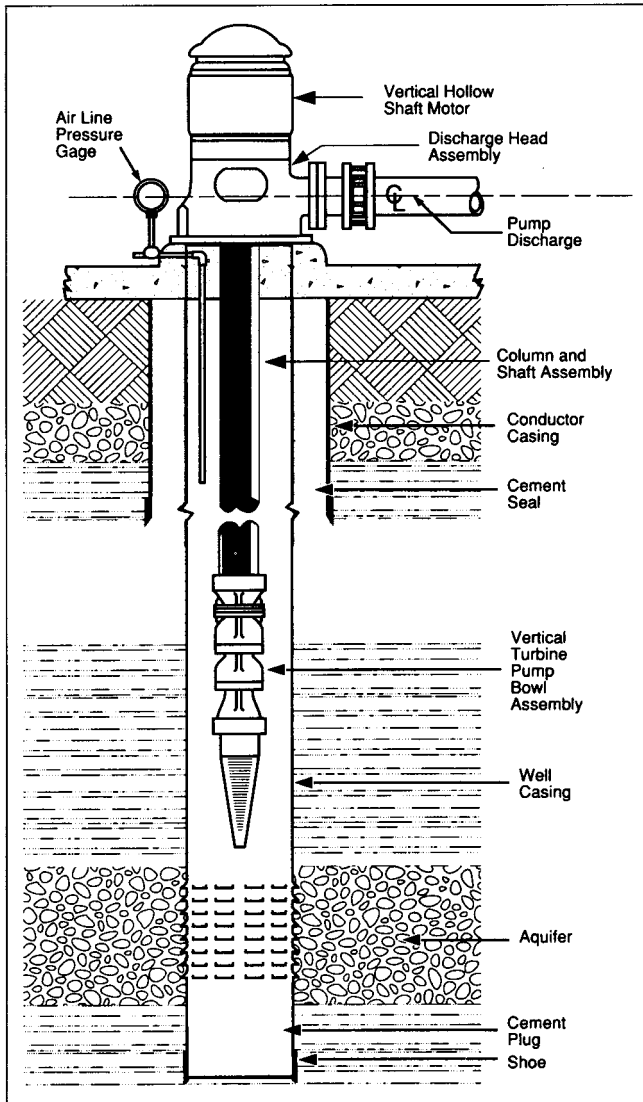
Before ground water can be used for consumptive purposes, it must be tested and approved by health authorities. In some locales, the developer needs such approval before any construction permits are issued. After the project is completed, state and local regulations may require periodic testing to ensure potable-water-quality standards. Treatment of the ground water may be necessary to remove (or reduce) some types of pollutants. In the extreme case, the water supply may have to be abandoned. Site conditions may provide clues to potential ground-water contamination. Following is a list of contamination sources that may affect ground-water quality.

- *Wastewater effluent.* The primary concern in semi-rural development projects is the relative location of the well to any subsurface, wastewater disposal system. Typically, houses connected to wells also utilize a subsurface septic system. Water percolating from the drain field into the water table is a serious health hazard. In order to prevent this, the well should be located upslope and a minimum of 50 ft (horizontally) away from the disposal system.

- *Barnyard runoff.* Wells constructed downslope of barnyards and farmhouses present similar bacteria problems as those from household wastewater effluent. Bacteria from animal waste can infiltrate to the aquifer.



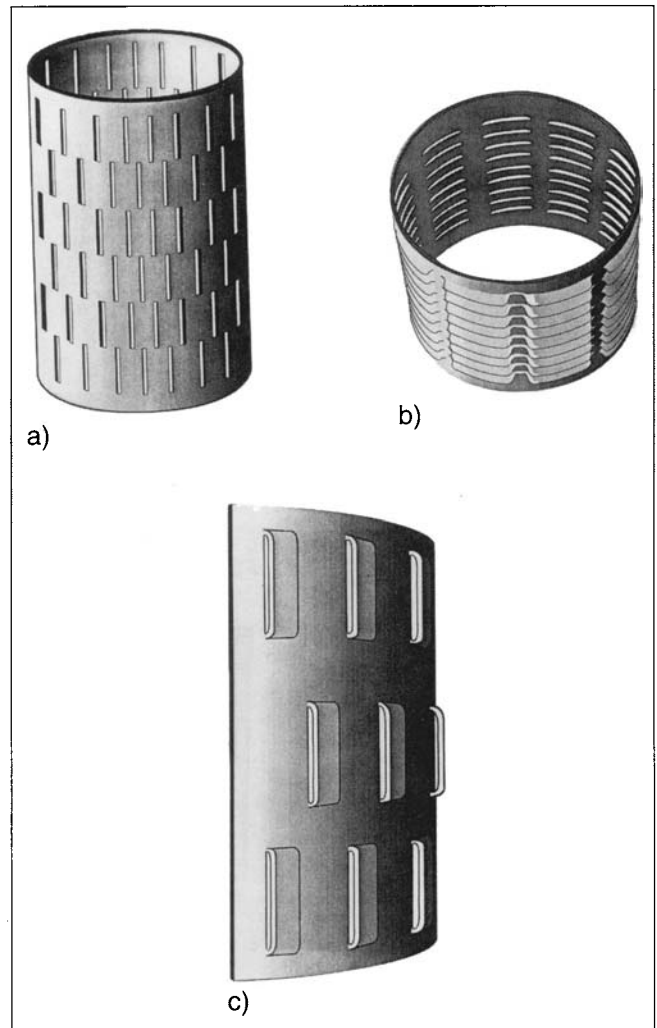
**FIGURE 28.5** Direct hydraulic rotary system. Courtesy of *Handbook of Ground Water Development*, Roscoe Moss Co. Reprinted by permission of John Wiley & Sons, Inc.



**FIGURE 28.6** Typical well installation. Courtesy of *Handbook of Ground Water Development*, Roscoe Moss Co. Reprinted by permission of John Wiley & Sons, Inc.

■ *Pesticides and fertilizers.* Not only are farm fertilizers and pesticides a contaminant hazard, but many homeowners use fertilizers and pesticides on their lawns and gardens. Historically, many farms dumped the pesticide and fertilizer containers in wooded areas and pits. Similarly, orchards and vegetable lands are potentially hazardous areas with high concentrations of pesticides and fertilizers. Houses and wells may now stand in the vicinity of these disposal areas and crop lands. The decomposition or neutralization of the synthetic chemicals used in pesticides and fertilizers has not been fully determined.

■ *Landfill areas.* Although present laws regulate the type of material that can be disposed of in a landfill, many closed landfills contain toxic chemicals that could cause long-term damage to the ground water.



**FIGURE 28.7** Types of screen intakes (a) vertical machine slotted screen; (b) shutter screen; (c) bridge slot screen. Courtesy of *Handbook of Ground Water Development*, Roscoe Moss Co. Reprinted by permission of John Wiley & Sons, Inc..

■ *Oil and gas fields.* Oil and gas fields present two potential ground-water contamination hazards. The first potential problem arises from the fact that a by-product of oil/gas development is saline water. Presently, most states have regulations on methods for disposal of this saline water to prevent it from contaminating ground water. In the older oil and gas fields, saline water is escaping from improperly sealed wells and contaminating the ground water. Additionally, oil and gas production is increased by injecting saline water into the geologic cavity. The increased pressure in the cavity forces oil, gas and saline water outward, into the ground water.

This list emphasizes the need to verify the previous land uses on any proposed developments with existing well systems. The site analysis and inspection may provide clues to potential contamination hazards. Additionally, subsurface

soil and water samples will verify suspicious areas. The various types of contaminants associated with various land activities are identified in Table 28.3. For water supplies already contaminated, the problem becomes one of pinpointing the type and location of the contaminating source. Table 28.5 summarizes some possible remedies for specific ground-water problems.

Table 28.4 provides recommendations for minimum distances between contamination sources and wells. Because of the many factors that influence ground-water movement, it must be recognized that there is no absolute safe distance between a well and a contaminant source. Even though health department and local regulations dictate minimum distances, the engineer should consult specialists when conditions appear questionable.

**MTBE**

Methyl Tertiary—Butyl Ether (MTBE) has been a highly effective, clean-fuel additive for reducing air pollution from motor vehicles. Unfortunately, it has also contaminated community ground water supplies across the country. MTBE was first used in the late 1970s as an automobile fuel additive for premium-grade fuels to increase octave rat-

**TABLE 28.4 Distance from Sources of Contamination to Private Water Supplies**

SOURCE	DISTANCE (ft)
Subsurface pits	50–75
Septic tank	25–50
Subsurface disposal fields	50–75
Sewage line	25–50
Seepage pits	50–100
Pasture	50–100
Barnyard	50–100
Farm silo	25–50
Petroleum or chemical Storage tank	50–100

**TABLE 28.3 Major Contaminants Associated with Waste-disposal Practices**

SOURCE	POSSIBLE MAJOR CONTAMINANTS
Landfills	
Municipal	Heavy metal, chlorides, sodium, calcium
Industrial	Wide variety of inorganic and organic constituents
Hazardous waste disposal sites	Wide variety of inorganic (particularly heavy metals) and organic compounds (pesticides, priority pollutants, etc.)
Liquid waste storage ponds (lagoons, leaching ponds, recharge basins)	Heavy metals, solvents, inorganic compounds
Subsurface sewage disposal systems	Organic compounds (degreasers, solvents), nitrogen compounds sulfates, sodium, microbiological contaminants
Deep well waste injection	Variety of inorganic and/or organic compounds
Agricultural activities	Fertilizers, herbicides, pesticides
Land application (sludge, wastewater)	Heavy metals, inorganic compounds, organic compounds
Urban runoff infiltration	Inorganic compounds, heavy metals, petroleum products
Deicing activities	Chlorides, sodium, calcium
Radioactive wastes	Radioactivity and radionucleides

From Hess, A.F., and J.E. Dyksen, *Utility Experiences Related to Existing and Proposed Drinking Water Regulations*. Reprinted from Annual Conference Proceedings, by permission. Copyright © 1984, American Water Works Association.



**TABLE 28.5 Groundwater Problems, Causes and Remedies**

<b>PROBLEM</b>	<b>PROBABLE CAUSE</b>	<b>REMEDY OR SOURCE OF HELP</b>
Inadequate water yield	Poor aquifer	Install larger, deeper well; have well cleaned by well driller; deepen well; contact water resource agency
	Well screen or pump intake encrusted	
	Lower water level	
Wet basement	Seasonally high water table Recovered water level Drainage from roof or slope	Add sump pump or drains; add sump pump or drains; add roof gutter, reslope land; contact SCS
Gas in water	Methane from bedrock state	Install vent on well head; aerate the water; install water treatment; drill new well away from house; contact geological survey
Salty water	Road salting Road-salt stockpile	Install new well farther upslope; provide better road drainage; install new well away from drainage; request correction by highway department; contact health department
Fuel-oil contamination	Leaky or spilled storage tank	Install new well upslope; adjust to low pumping rate
Oil/gasoline contamination	Nearby service station	Obtain new source of water; contact health department
Bacteria contamination	Septic effluent	Chlorinate as first step; contact health dept.; install new well upslope; install new leach field farther away; deepen well in some cases; seek control on neighboring system
Barnyard waste		Redirect waste flow; install new well upslope; seek control on neighbor's activity; contact agricultural agency
Organic chemical	Former land use contamination	Install new well farther away; deepen well in some cases; contact health department
	Current land application	Create a buffer zone around recharge area; dispose of wash water properly; seek control on neighbor's activity; contact health department
Land subsidence	Excessive ground-water withdrawal Rock solution	Contact state regulatory agency; relocate house; contact state geological survey
Sinkhole development	Rock solution	Relocate house; contact state geological survey

Source: Walter, R. M., *Groundwater and The Rural Homeowner*, U.S. Geological Survey, 1988.

ings. It was added in higher concentrations, starting in the 1990s, as an oxygenate to enable more complete combustion, in order to reduce carbon monoxide and other toxic air emissions. It readily dissolves in water and permeates rapidly through soils and aquifers. MTBE is more likely to

contaminate ground water than other components of gasoline due to its resistance to microbial decomposition and mobility in water. The U.S. Geological Survey has reported that approximately 3% of ground-water wells in areas of the country where MTBE has been used as a fuel additive

have detection levels of 5 parts per billion (ppb) or above. Based on carcinogenic effects observed in laboratory animals, the action level for concern of potential cancer is 13 ppb. The best available technology for removal of MTBE is adsorption with granular-activated carbon (GAC).

Several states, and the U.S. Congress, are considering legislation to eliminate MTBE as a gasoline additive. Ethanol is the likely replacement for MTBE. It has proved to be effective in reducing air emissions and does not have the potential for ground-water contamination posed by MTBE.

## SURFACE WATER SUPPLY

Surface water sources are usually more difficult to develop than ground-water sources for small community potable-water systems. Surface waters, which are open to contamination from pollution, debris, sediment, and waterborne pathogens, usually require more extensive treatment than ground-water sources. Legal rights to ground water are normally riparian, meaning acquired with title of the land overlying the water source. The rights to surface waters are more complicated and vary widely from state to state in the United States. No development of any water source should be initiated until the rights to the water have been successfully concluded.

The advantages of surface water include the fact that it can be more abundant (for large development projects) and have less dissolved-mineral content than ground water. The permitting process, however, is long and arduous in most states for either withdrawal from free flowing streams or development of impoundments.

### Safe Yield

Once it is determined how much water will be needed for the planned development, potential sources can be evaluated for adequate production. Existing or potential lakes and reservoirs, rivers, and streams must be analyzed for safe yield. Safe yield is the amount of water that can be reasonably counted on for use by the community during drought conditions, assuming flowby requirements are met for maintaining downstream aquatic life and water-supported activities. Safe-yield analyses are complex and require specialized expertise as well as assistance from public agencies with meteorological and hydrological data.

### Reservoirs and Lakes

In order to successfully use water supply from a reservoir or lake, the following features must exist:

- The upstream watershed physical characteristics allow provision of consistent water quality and quantity.
- The watershed is protected by ownership, agreements with landowners, or by state legislation.
- Adequate storage is available to allow settling of sediments and larger micro-organisms prior to treatment.

- Adequate storage is available to allow sufficient supply during periods of drought.
- The intake design allows the highest-quality water available to be withdrawn for treatment.
- The stored water is treated to reduce the levels of turbidity, disinfection by product precursors and micro-organisms, and then is adequately disinfected.

The ideal watershed to the impoundment should be: (1) free of development and sewage systems, (2) large enough to fill the reservoir during dry years, (3) protected against erosion and drainage from livestock confinement areas, and (4) fenced for the exclusion of livestock.

Development of a reservoir often involves the construction of an embankment with an overflow spillway. The services of qualified engineering and hydrogeological consultants are necessary to design the facilities and oversee their construction. Regulatory supervision and assistance is available from the National Resources Conservation Service (NRCS) and state or county agricultural, geological, or soil conservation agencies.

The intake-facility design is vitally important to the successful operation of the water supply. If the water is drawn too close to the impoundment bottom, it may contain decaying organic matter or be too turbid, while water drawn from too near the water surface may contain algae, floating debris, or aquatic plants, resulting in taste and odor problems in the finished water. The depth of optimal intake varies with the configuration of the impoundment and the season of the year. Intake structures should be designed to provide flexibility for depth of withdrawal for optimal year-round operations.

### Rivers and Streams

Dependable year-round water supply is provided by many rivers and streams. When the only feasible source for some communities is a stream receiving runoff from a large, uncontrolled watershed, the water may be difficult and expensive to treat because of rapidly varying bacteriological and physical qualities.

As with reservoirs, the intake facilities on rivers and streams must also be designed carefully. First, they should be located upstream of any sources of contamination, such as wastewater discharges. Additionally, the intakes should be designed to exclude sediment and debris, while allowing efficient withdrawal at varying stream levels. Often, the intake is designed with the features to protect it from high-water levels, floating debris, and ice formation. On wide streams, it is often advantageous to locate the intake in the middle of the stream rather than at the bank. This design technique avoids turbid water from an upstream tributary, which typically flows along the bank of entry after a local storm event. Figure 28.8 shows a typical prefabricated intake-screen device commonly used on stream intakes.



**FIGURE 28.8** Prefabricated stream intake. (Courtesy of U.S. Filter, Warrendale, PA)

This device has an air blowback feature used to clear the screen of sediment and debris.

To add all-season reliability to the water source, it may be necessary to implement an off-stream storage reservoir. In this manner, water can be stored in times of heavy runoff for use when demands may exceed available stream flow. The off-stream storage can also be used when the stream source is of poor quality because of low flow or is contaminated because of a hazardous-material spill.

The use of an infiltration-gallery-intake arrangement can also be of great benefit, especially when the stream is subject to extreme changes in turbidity from storm events. The infiltration-gallery method of pretreatment is also referred to as riverbank filtration. Infiltration galleries are shallow, horizontal beds of natural or constructed permeable soils, with collector pipes embedded in them, that are situated adjacent to the water source. Additionally, they provide some pretreatment because of the natural filtration provided by the soil, and this improves the source-water quality. Although they are more commonly used for river and stream sources, they can be used to benefit reservoirs that are subject to events of heavy turbidity.

It should be noted that the use of an infiltration gallery to pre-filter the source water does not preclude the need for further filtration and disinfection treatment. Even the waters of crystal-clear mountain streams have been found to contain serious health-threatening organisms, such as cryptosporidium and giardia.

## WATER TREATMENT

The evolution of modern water treatment began in the early 20th century, when the practices of filtration and chlorination disinfection became common. The main goal at that time was to remove (or inactivate) pathogenic organisms to curtail the disastrous waterborne disease outbreaks that routinely plagued the country during the 19th century. This goal was achieved, as waterborne disease outbreaks were

mostly eliminated in the U.S. by the 1930s. As a result, the belief of the water utility and engineering community was, until fairly late in the century, that microbiological contamination in surface waters could be successfully addressed with appropriate pretreatment and filtration to attain clarity of 1.0 turbidity unit (ntu) or less, followed by chlorination.

During this time, ground water was considered to be already filtered, requiring only the addition of chlorine to maintain a suitable residual in the distribution system. Any additional treatment performed was done solely to change water characteristics unrelated to health, such as hardness, taste, odor, and coloration from manganese and iron. Consequently, the main challenge to water-supply engineering in the 1960s and 1970s was to attain these simple treatment goals at the lowest operation and maintenance cost. As a result, new techniques were developed during that period to clarify surface water more economically. These included new sedimentation-basin designs, tube and plate settlers, dissolved air flotation, high-rate filtration processes, and pre-engineered proprietary-package systems that integrated several basic processes into a single-treatment unit.

Potential chronic (long-term) health risks from trace organic compounds in potable water became a public concern in the 1970s and 1980s. A new wave of regulations followed to set maximum containment levels (MCLs) for trihalomethanes (THMs), volatile organic compounds (VOCs), and pesticides. In response to this public concern, which continues to the present day, water engineers have developed successfully new treatment methods to remove organic compounds from drinking water. These new approaches, such as enhanced coagulation, air stripping, and activated carbon adsorption, were the main focus of water-treatment engineering through the mid-1990s.

Microbiological contamination, however, has recently emerged (again) as the primary focus of the water-treatment industry. The main causes of this development are: (1) documented cases of drinking water supplies contaminated by chlorine resistant cysts such as cryptosporidium and giardia rather than bacteria; and (2) the promulgation by the U.S. EPA of the Total Coliform Rule and the Surface Water Treatment Rule in 1989 and subsequent enforcement and monitoring actions. To address these new challenges, there is renewed emphasis on source-water protection, treatment-plant optimization, management of plant recycle streams, and implementation of newer technologies such as ozonation and membrane treatment.

Today's treatment-system engineer not only must meet current drinking water standards but must anticipate future regulatory requirements. No more is the definition of "safe" water a constant as it was in the 1950s, 60s, and 70s. The water industry will continue to struggle with the potential health effects of traces of an ever-increasing variety of organic compounds and infectious organisms. Now, more than ever, a water-treatment system must provide flexibility for modification in order to meet potential requirements.

A process schematic of a complete, conventional surface-water-treatment system is shown in Figure 28.9. The objective of water treatment is to provide high-quality, safe, finished-water for long-term use. The processes used by treatment facilities to achieve this depend on the characteristics of the source water. As noted previously, surface water is likely to contain pathogenic organisms, organic substances, and particulate matter, while ground water may have undesirable mineral content or contaminants such as volatile organic compounds. The following processes are those normally used in water treatment and are listed more or less in sequential order of occurrence in a water plant:

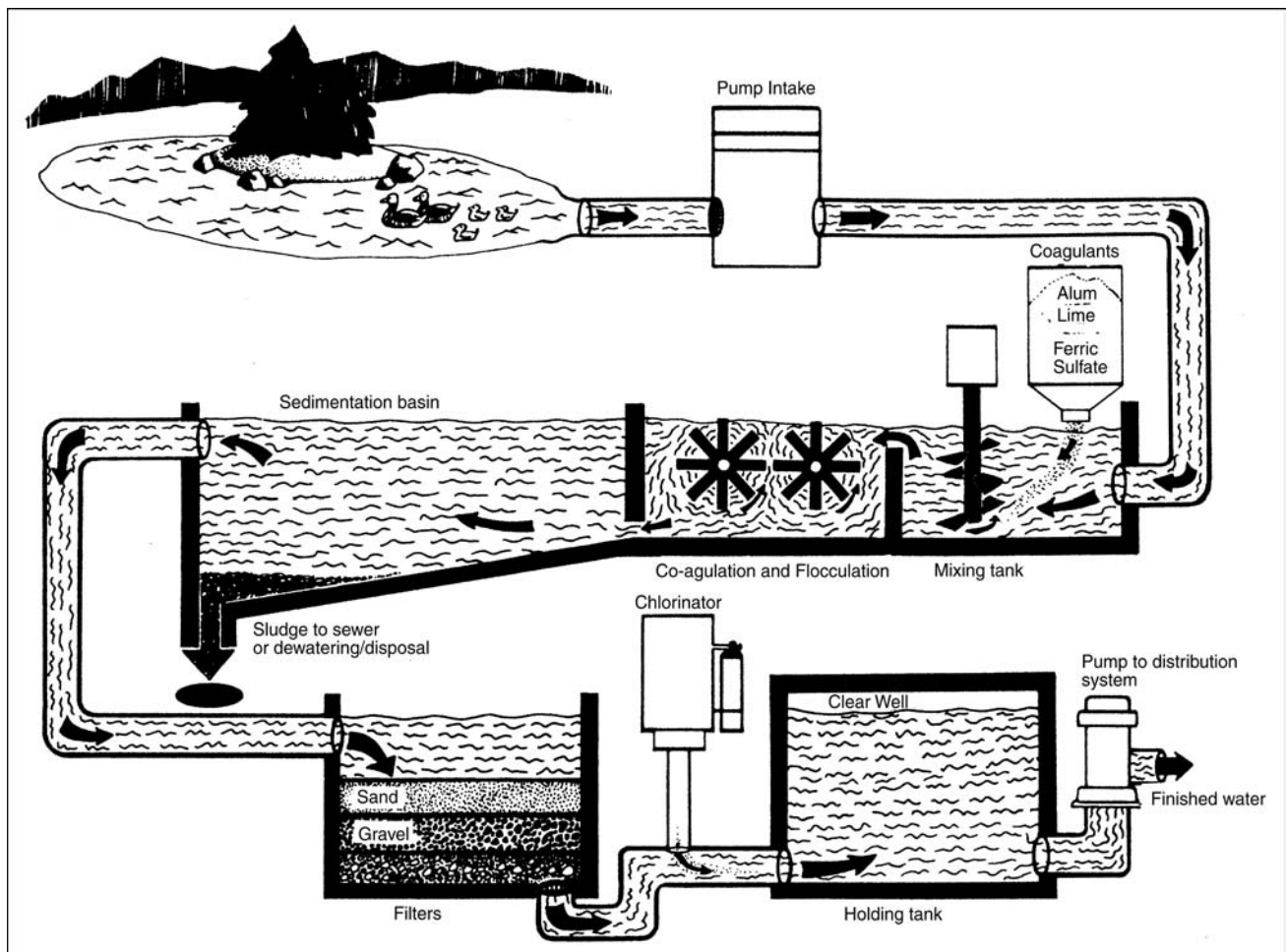
Pretreatment	Conventional filtration
Aeration	Membrane filtration
Coagulation	Adsorption
Flocculation	Disinfection
Sedimentation	Corrosion control
Softening	Fluoridation

These processes overlap and would not all be appropri-

ate for use in any single treatment plant for surface or ground water. Disinfection is the one process that would be used in all treatment systems for a community public-water supply.

**Pretreatment**

Pretreatment refers to any process that conditions the water prior to its entering the main treatment facility. For ground water, pretreatment is accomplished by natural filtering through aquifer sand and gravel layers, or through an artificially-placed gravel pack at the well screen intake. Chemical injection (usually hypochlorite) of ground water for pretreatment to control iron bacteria is sometimes necessary, while pretreatment for surface water consists of screening as a minimum. Presettling in an off-stream reservoir, for example, can provide more consistent raw-water turbidity, making treatment operations less variable and complex. Nuisance aquatic organisms such as algae, weeds, and zebra mussels can interfere with treatment processes and cause unpleasant tastes and odors in the finished water. Exclusion of sunlight from the intake structure and judicious application of copper sulfate to the impoundment



**FIGURE 28.9** Typical surface water treatment plant schematic.

have been effective in controlling algae. Physical screening and cleaning are normally the most cost-effective means for controlling aquatic weeds and animals.

### Aeration (Air-Stripping)

Uses of aeration as a means of treatment are:

- Removal of volatile organic compounds (VOCs)
- Removal of radon gas
- Removal of substances that may interfere with downstream processes (e.g. removal of carbon dioxide from water prior to lime-softening)
- Oxidation of iron and manganese so they can be removed by downstream processes
- Reduction of concentrations of substances causing undesirable tastes and odors, and for oxidation of organic matter

Aeration can be accomplished in any number of ways, including: waterfall (i.e., cascade), diffused air, mechanical aeration, and pressure aerators (adding air to water under pressure).

### Coagulation

Coagulation is the process of adding chemicals to raw water that cause an almost instantaneous reduction of the forces that tend to keep particles separated. Coagulant chemicals routinely added to destabilize particles are alum, ferric salts, lime, and cationic organic polymers. The coagulation process is also known as rapid mixing or flash mixing. The purpose of flash mixing is to provide uniform, instantaneous dispersion of the coagulant chemical throughout the influent raw water to condition it for subsequent removal of particles. Rapid mix is achieved by introducing the chemicals at a point of maximum agitation, such as at a static mixer. Enhanced coagulation is a term coined by the U.S. EPA in the Disinfectants and Disinfection By Products Rule. This rule requires that the coagulation process of some water supplies be operated in order to provide removal of a specific percentage of organic material from raw water as measured by total organic carbon (TOC). This enhanced removal of TOC is usually achieved by increasing the dosage of coagulant chemical or by adjusting the raw water pH during the coagulation reaction.

### Flocculation

Flocculation is the agglomeration of small particles and colloids to form larger particles (flocs) that can be settled or filtered. Incidental flocculation begins in the zone of decaying mixing energy of the rapid-mixing step (that destabilized the forces that keep particles separated). In some instances, this incidental flocculation may be adequate. However, a separate flocculation process is usually included in the process flow train to build floc to optimum size, density, and strength. Sometimes a separate aid, such as a

cationic, anionic, or non-ionic polymer, is added in the flocculation process to achieve greater floc density. Baffles, low-moving paddles, or gentle aeration, all of which provide velocity gradients that promote particulate contact (without floc-shearing agitation), are used in conventional plants.

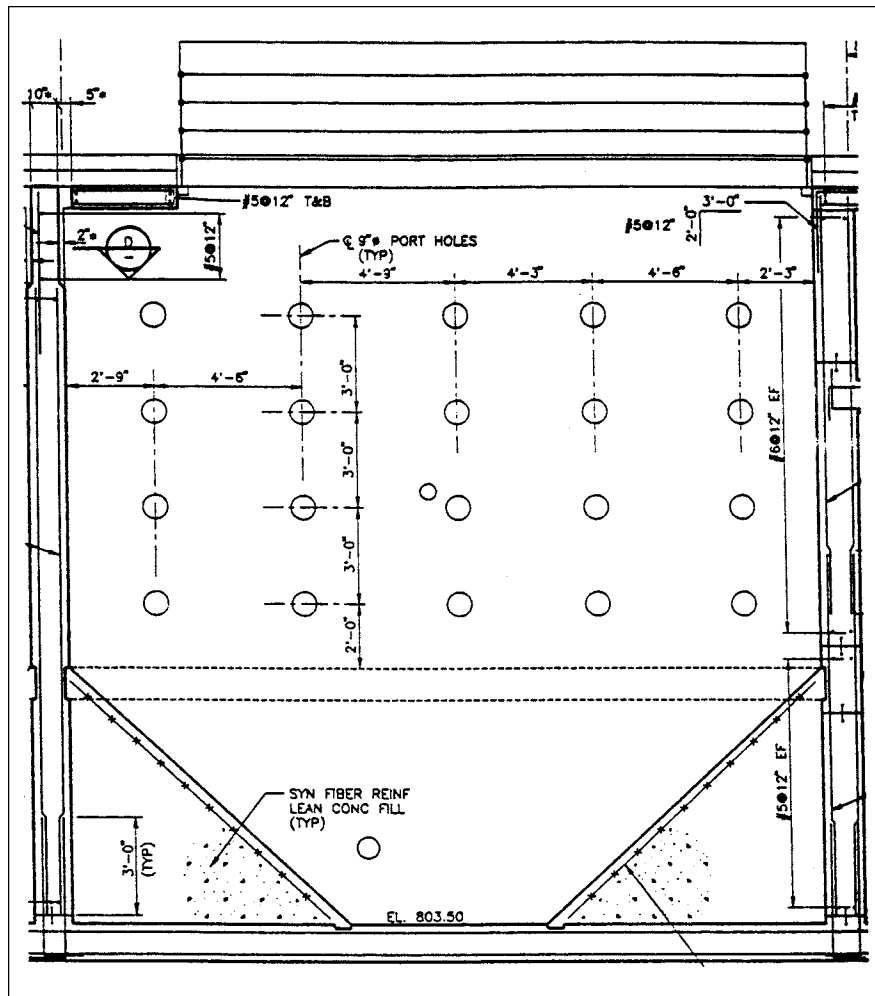
### Sedimentation

During sedimentation, suspended material begins to settle out of the water because of gravity. Some water-plant settling basins have mechanical collectors for solids removal and disposal. When there are no solids collectors, multiple basins must be provided so that each basin can periodically be taken out of service for solids removal. Detention time is not usually an important design parameter for modern sedimentation basins. It is rather more important that sedimentation basin inlets be baffled properly to prevent short-circuiting (see Figure 28.10), and that they be designed for proper surface-loading rates (see Table 28.6). The efficiency of modern sedimentation basins is often enhanced with either tube settlers or lamella plates (see Figure 28.11). These devices allow sedimentation-basin space requirements to be greatly decreased. Consequently, these devices are very popular for use in pre-engineered package-plant units, while still remaining popular for use in large water plants.

### Softening

Water softening is the process by which minerals, mainly calcium and magnesium, are removed from water. Softening also removes metals and other water constituents and is commonly used when hard-water scaling is a problem. The processes by which water is softened include lime-soda ash, ion exchange, and membrane treatment. Note: both ion-exchange and lime-soda ash processes increase the sodium content of the finished water. This fact must be considered for those customers on low-sodium diets. Lime and soda ash react with the various calcium and magnesium salts (bicarbonates) to form two insoluble precipitates—calcium carbonate and magnesium hydroxide—which are removed by sedimentation and filtration. If sodium hydroxide (caustic soda) is substituted for lime, less sludge is produced, a consideration when sludge-disposal costs are high. However, caustic soda is more expensive than lime and increases the total dissolved solids contents in the treated water. Additionally, it is also a hazardous chemical that may not be appropriate for small systems employing part-time personnel. Lime and soda ash softening, with certain adjustments at large plants, also provides removal of color and turbidity.

A more appropriate process choice for water softening in small systems is probably ion-exchange. Ion-exchange is used by small community systems with ground-water supplies and in household softeners. Softening is provided by replacing (or exchanging) hardness ions in the water with sodium ions, and this is accomplished by flowing hard water through a vessel containing polystyrene resins (see Figure 28.12). The calcium and magnesium ions, within the water, attach to the resin, which releases sodium ions. The resins must be periodically regenerated by adding a salt-



**FIGURE 28.10** Typical inlet baffle across rectangular clarifier in section. (Reprinted from *Water Treatment Plant Design*, 3rd Ed., 1990, McGraw-Hill, New York.)

water sodium chloride solution to the vessel; to replenish the sodium ions, a small wastewater stream requiring disposal is created. Ion-exchange has also become popular in recent years for small community water systems, since it also is useful in removing nitrate from ground water supplies.

Potassium chloride, as a substitute for sodium chloride in water softeners, is becoming more popular due to environmental considerations. A small amount of potassium in drinking water can be beneficial to users, while sodium can be harmful for users on salt-sensitive diets. There is also some concern that sodium damages septic tank drain fields and deteriorates the quality of ground water after recharge. The only disadvantage of potassium chloride is that it presently costs nearly twice as much as sodium chloride.

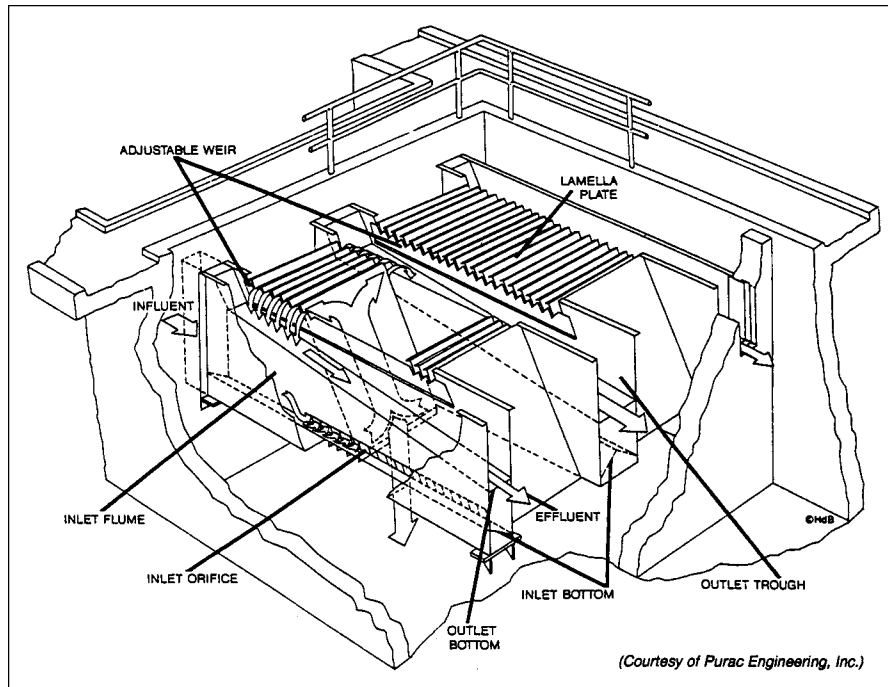
**TABLE 28.6 Typical Sedimentation Surface-loading Rates Using Alum Coagulation**

APPLICATION	gpd/ft <sup>2</sup>
Turbidity removal	800 to 1200
Color and taste removal	600 to 1000
High algae content	500 to 800

Reprinted from *Water Treatment Plant Design*, 3rd Ed., 1990, McGraw-Hill, New York.

**Conventional Filtration**

Conventional filtration, as applied to water treatment, is the passing of water through a porous medium to remove suspended solids. Filtration remains one of the fundamental processes of water treatment, with recorded use as early as 4000 B.C. The effectiveness of filtration is determined by measuring the turbidity of the filtered water. Turbidity is a light-scattering property of suspended particles in water measured in turbidity units, ntu. Since most of the turbidity within highly turbid surface-source water is removed by processes preceding filtration, the solids load on the filters



**FIGURE 28.11** Plate settlers.

is reduced, making them more cost-effective to operate. Filters must produce a high-quality effluent to ensure that disinfection of the finished water is effective. Filtration greatly improves water quality but is only one of several major barriers against contamination in finished potable water. Ground water that is low in iron and manganese and not contaminated by surface influence may not require filtration; however, all surface waters must receive filtration as required by the SDWA Surface Water Treatment Rule. The types of filters that may be used are slow-sand, rapid-sand, diatomaceous-earth, bag, and cartridge. All must be followed by effective disinfection.

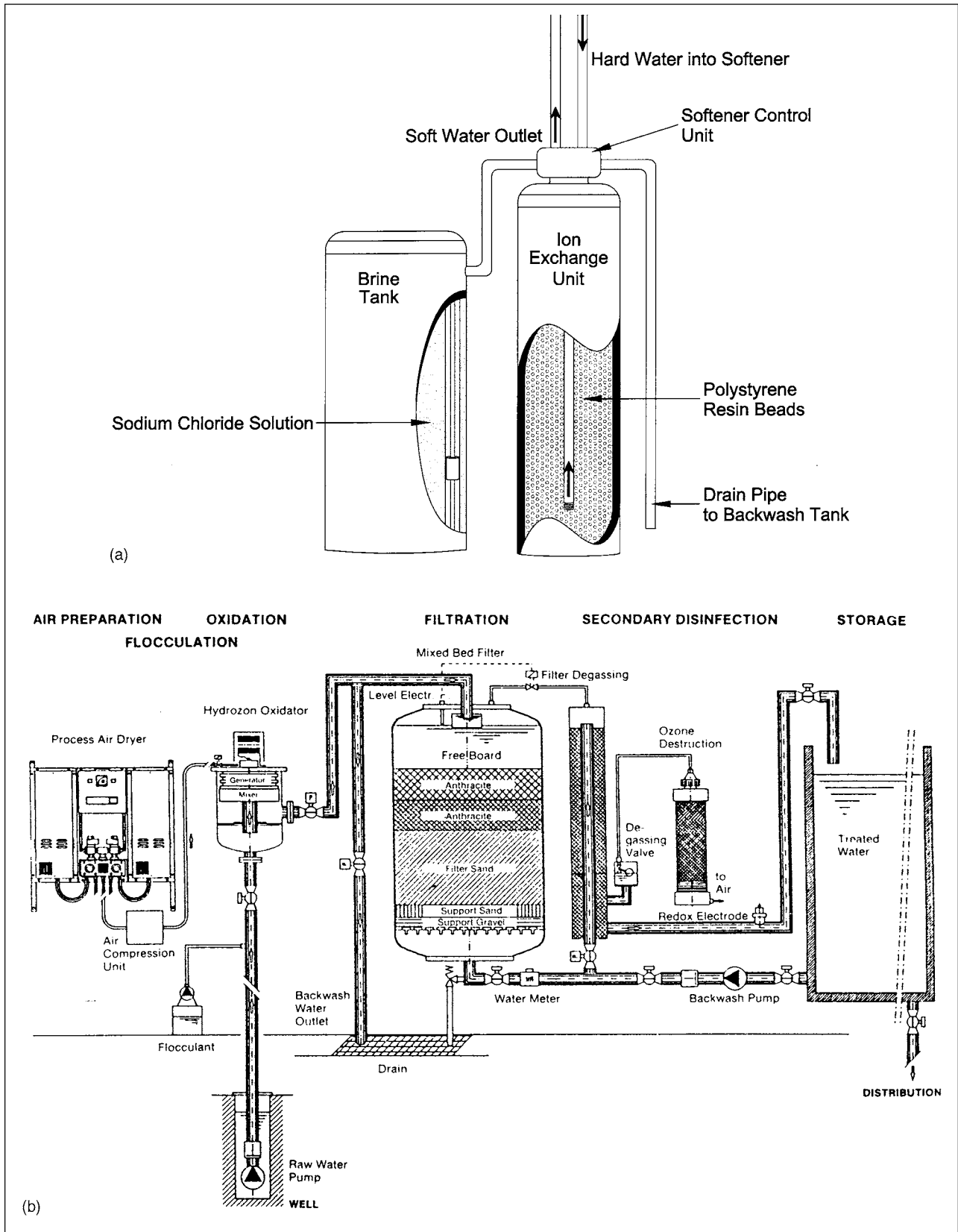
- *Slow-sand filters.* For slow-sand filtration, water is passed through beds of fine sand at light loading rates averaging  $0.05 \text{ gpm/ft}^2$ . Although much more filter-bed area is needed for slow-sand filtration than for other methods, this may not be that significant for small plants. Since the sand is not backwashed in place, the top 1 in. of its surface is periodically removed and replaced with clean sand. The removed sand is usually discarded, but it can be washed and stockpiled for replacement later. Slow-sand filtration does not require extensive pretreatment with chemicals but works best in waters that are not excessively turbid or have highly variable turbidity. It is normally preceded with presettling in an impoundment, as a minimum. Slow-sand filters periodically slough biological material, as they are biological processes. The time-period between removals of the top layer of sand depends on the turbidity of the water. Multiple beds are

needed so some may be cleaned while others remain in service. The use of this method has become less common as more mechanical-intensive means have become more reliable.

- *Rapid-sand (high-rate) filters.* This method is presently the most commonly used means of filtration for both large and small water plants in the U.S. Filtration beds of sand or sand and anthracite coal, usually supported on beds of gravel, are loaded at rates of 2 to  $10 \text{ gpm/ft}^2$ . The filters are cleaned by backwashing finished water in the reverse direction through the filters after filter runs of 12 hr to 15 days. Note: if using this technique with a surface-water source, the surface water must receive coagulation, flocculation, and clarification before high-rate filtration.

High-rate filters can either be gravity units or pressure units. For pressure units, the water is pumped through an enclosed pressure-vessel that contains the filter medium. This type of unit is often used by small utilities with ground-water sources.

- *Diatomaceous earth (DE) filters.* DE filters can either be in pressure or vacuum-vessels. The filter media is diatomaceous earth, the skeletal remains of small single-cell organisms in sea water called diatoms. DE filters are backwashed every several hours, but the backwash process removes the medium along with the accumulated solids. The filters must then be precoated with new DE prior to the next filter run. During a filter run, DE medium, is continually added to the influ-



**FIGURE 28.12** Schematics of (a) basic ion exchange unit; (b) ion exchange treatment system. (Courtesy of National Drinking Water Clearinghouse, Morgantown, WV.)



ent water to maintain the filtering surface in optimal condition. DE filters, which were used extensively for small systems from the 1970s to early 1990s, are waning in popularity because of the development of less complex package plants and membrane processes.

- *Bag and cartridge filters.* Bag and cartridge filters have been used for years in industrial water-treatment applications but are now becoming more practical for use in small municipal systems. For bag filtration, disposal bags made from materials of a specific pore size are supported in a housing as water flows through them for removal of particulate matter. Typically, bags are used in stages from larger to smaller pore size and are replaced and discarded once maximum operating pressures are reached from solids accumulation in the bags.

The arrangement and operation of cartridge filters is similar to bag filters in that they are housed in canisters in multiple stages of porosity and are disposable once excessive operating pressures are reached. The cartridges are of rigid construction, usually made of ceramics or cotton fibers.

### Membrane Technology

A semi-permeable membrane is a thin layer of material that has the capability of separating substances when a driving force is applied across it. Membrane technology was once considered viable only for desalinization. However, advances in membrane production and module design have greatly contributed to the decline in capital and operating costs of such facilities for water treatment. These installations are being increasingly employed for removal of bacteria, other micro-organisms, and particulate and natural organic matter (which causes taste and odors and reacts with disinfectants to form undesirable organic by-products). The capital cost of membrane treatment (on a dollar per volume treatment-capacity basis) does not rapidly increase as treatment capacity decreases, as it does with conventional water treatment. This allows membrane technology to be cost-effective for small systems.

Indeed, the National Research Council, in a 1997 report, concluded that membranes will be used with accelerating frequency in small systems as the complexity of conventional treatment processes increases to meet new regulatory requirements. As an example, for ground-water sources not needing pretreatment, membrane-treatment facilities consist of little else than holding tanks, membrane modules, and feed and cleaning pumps. However, wastestreams for membrane technologies are 15% or more of inflow, while conventional treatment wastestreams are 5 to 10%. This is a significant consideration where wastestream disposal is a difficult problem.

Traditional water treatment has focused on liquid-solid separation, rather than removal of dissolved contaminants

from water possible with membrane technologies. The 1996 Safe Drinking Water Act (SDWA) Amendments encouraged the use of membrane treatment alone or in combination with conventional liquid-solid separation to meet the more stringent requirements. Membrane processes used in water treatment are electrodialysis, reverse osmosis, microfiltration, ultrafiltration, and nanofiltration.

- *Electrodialysis.* This process has been widely used for desalinization of brackish water. Unlike other membrane processes that use high pressure to force water through a membrane, electrodialysis uses electrodes to pull positive and negative ions from the water through selective membranes. Since only charged ions are removed, the range of constituents removed is much more narrow than, for example, reverse osmosis. Electrodialysis is effective in removal of fluoride and nitrate and can also be used to remove selenium, barium, and cadmium. Its wastestream, however, is 20 to 90% of influent flow, and it has a relatively high capital and operating cost, since it requires a high level of pretreatment. For these reasons it is not appropriate for consideration for most small community systems, except for special applications, such as for brackish water.

- *Reverse osmosis.* Pressure, as high as 300 psi depending on the application, is used to force water through a semi-permeable membrane in the reverse osmosis (RO) process. Nearly all (96%) inorganic contaminants are left on the high-pressure side as water passes through the molecular structure of the membrane. RO is also effective for removal of natural organic substances, pesticides, radium, and microbiological contaminants. Is used in series, RO can achieve contamination concentrations of nearly zero. To prevent membrane fouling, filtration pretreatment is usually required as a minimum, while sometimes chemical additives, softening, or both are needed. Note: compact commercial-package RO units are available that include all the pretreatment equipment. These units only require minimal labor and are simple to operate, making them suitable for small systems, especially when there are extremes in seasonal water demand.

- *Nanofiltration, ultrafiltration and microfiltration.* These pressure-driven membrane technologies differ only in the pore size of the membranes and can be used alone or in combination, depending on the water quality goals. These processes are sometimes referred to collectively as membrane filtration. Microfiltration uses the largest pore size of the three for removal of particulate matter and microbes in the 0.05 to 10  $\mu\text{m}$  size range. It is not, however, able to remove soluble constituents. Ultrafiltration is also used for liquid-solids separation, but can also remove ionic organic material (based on molecular weight) and viruses since its size

operating range is 0.001 to 0.05  $\mu\text{m}$ . Both ultrafiltration and microfiltration operating pressures are from 10 to 100 psi. Nanofiltration, which uses operating pressures from 75 to 150 psi, removes particles as small as 1 nanometer. Nanofiltration is very effective in removal of color and disinfection by-product precursors, and it can remove ions contributing to hardness (such as calcium and magnesium).

Table 28.7 compares the membrane technologies for use in compliance to the Surface Water Treatment Rule.

### Adsorption

Adsorption (in water treatment) refers to a process by which the water is brought in contact with a material that attracts certain specific molecules and to which they adhere. The adsorptive material most suitable for organic containment removal, including taste and odor causing compounds, is granular-activated carbon (GAC). The EPA has designated GAC as the best available technology for synthetic organic-chemical removal (e.g., volatile organic compounds). Carbon is available in two forms: powdered-activated carbon (PAC) and granular-activated carbon (GAC).

Powdered-activated carbon can be fed, dry or in a slurry, early in the treatment process and later removed in downstream processes. Typically PAC is used in treatment plants occasionally to address taste and odor problems. GAC is used when continuous treatment is necessary. GAC is commonly used as a filter medium, either as a top-layer supplement to the filter bed or as a replacement for filter media in a normal filter bed or special contactor tank. The contactors can either be by gravity or pressure units, similar to those used in conventional filtration. The GAC must be periodically removed and replaced with new or regenerated carbon when its absorptive capacity is expended.

### Disinfection

The SDWA Surface Water Treatment Rule (SWTR) requires disinfection of finished water for public systems with surface-water sources or ground-water sources under the influence of surface water. Disinfection is arguably the most important step in making sure that water is safe to drink. Primary means of disinfection are chlorination, ultraviolet light, and ozone.

Disinfection (in the water utility industry) is referred to as primary and secondary. Primary disinfection is achieved as part of the treatment process, and secondary disinfection is done to ensure that the distribution system is protected against microbial contamination or regrowth. Chlorine and compounds of chlorine are by far the most popular means of disinfection, since they provide both primary and secondary residual disinfection. Using ultraviolet light is not as effective as chlorine (or ozone) at inactivating cryptosporidium and giardia, and no chemical disinfectant has been able to be totally reliable in inactivating cryptospori-

dium oocysts. The best-available treatment method for this pathogen is still physical removal.

- **Chlorination.** Chlorination is most commonly provided by gaseous chlorine, sodium hypochlorite or calcium hypochlorite. Chlorine is effective against almost all microbial pathogens and is appropriate as both the primary and secondary disinfectant. Figure 28.13 shows a typical arrangement for a cylinder-mounted chlorinator for a 150-lb chlorine gas cylinder typically used in small water plants. At least two of these arrangements with automatic switchover apparatus (for when one cylinder becomes empty) are usually provided. Since chlorine gas is lethal even at small concentrations in air, special housing, monitoring, safety equipment, and trained personnel are necessary.

Calcium hypochlorite is a white solid, available in tablet, granular or powder form, which dissolves easily in water. The tablets are designed for use in commercially available feeding apparatus that add tablets to plant flow based on flow rate or time. The powder and granular forms are commonly used to make a stock solution that is then metered into the plant flow at desired application points. Since calcium hypochlorite is highly corrosive, it must be kept away from organic materials such as wood or cloth, as contact may cause a heated reaction and fire. This chemical readily absorbs moisture, releasing chlorine gas. Shipping containers should be emptied completely and carefully resealed.

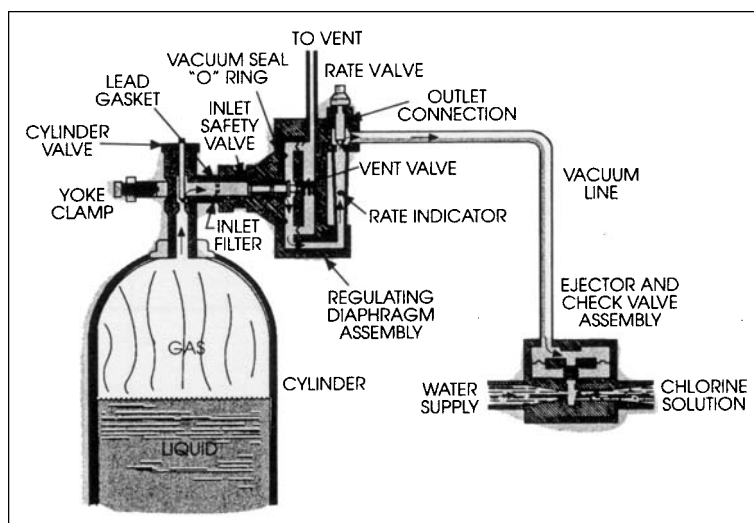
Another chemical commonly used for chlorination disinfection for small water systems is sodium hypochlorite. Although highly corrosive, sodium hypochlorite solution is less difficult to handle than either calcium hypochlorite or gaseous chlorine. Sodium hypochlorite is available in solution form in concentrations of 5 to 15% chlorine. Since sodium hypochlorite solutions deteriorate, they should not be stored more than 1 month even in an ideal cool, dry, and dark place. This chemical is more expensive than gaseous chlorine, although the cost is significantly reduced if bulk deliveries by tanker truck can be made to an on-site storage tank. This may not be practical for small systems due to the relatively short shelf-life of the solution.

Chloramine forms when ammonia is added to chlorinated water, or when water containing ammonia is chlorinated. It is a weak disinfection for use against viruses and protozoa as compared to free chlorine, but chloramination produces less disinfection byproducts. This makes chloramine appropriate for use as a secondary disinfectant to prevent bacterial regrowth in the water distribution system. Chloramine is commonly developed and applied by injecting ammonia gas (or ammonia hydroxide solution) into the flow stream just downstream of where chlorine is applied.

**T A B L E 2 8 . 7 Surface-water Treatment-compliance Technology; Membrane Filtration**

<b>UNIT TECHNOLOGIES</b>	<b>REMOVAL: LOG GIARDIA &amp; LOG VIRUS</b>	<b>RAW WATER, PRETREATMENT AND OTHER WATER QUALITY ISSUES</b>	<b>COMPLEXITY: EASE OF OPERATION (OPERATOR SKILL LEVEL)</b>	<b>SECONDARY WASTE GENERATION</b>	<b>OTHER LIMITATIONS/DRAWBACKS</b>
Microfiltration (MF)	Very effective giardia, >5–6 log; Partial removal of viruses (disinfect for virus credit).	High quality or pretreatment required. Same note regarding TOC.	Basic: increases with pre/post-treatment and membrane cleaning needs	Low-volume waste may include sand, silt, clay, cysts and algae	Disinfection required for viral inactivation
Ultrafiltration (UF)	Very effective giardia, >5–6 log; partial removal of viruses (disinfect for virus credit).	High quality or pretreatment required (e.g., MF). TOC rejection generally low, so if DBP precursors are a concern, NF may be preferable.	Basic: increases with pre/post-treatment and membrane cleaning needs.	Concentrated waste: 5 to 20% volume. Waste may include sand, silt, clays, crystals, algae, viruses and humic material.	Disinfection required for viral inactivation.
Nanofiltration (NF)	Very effective, absolute barrier (crysts and viruses)	Very high quality or pretreatment required (e.g., MF or UF to reduce fouling/extend cleaning intervals). See also RO pretreatments.	Intermediate: increases with pre/post-treatment and membrane cleaning needs.	Concentrated waste: 5 to 20% volume.	Disinfection required under regulation and recommended as a safety measure and residual protection.
Reverse Osmosis (RO)	Very effective, absolute barrier (crysts and viruses)	May require conventional or other pretreatment for surface water to protect membrane surfaces: may include turbidity or FE/MN removal; stabilization to prevent scaling; reduction of dissolved solids or hardness; pH adjustment.	Intermediate: increases with pre/post-treatment and membrane cleaning needs.	Briney waste. High volume, e.g., 25 to 50%. May be toxic to some species.	Bypassing of water (to provide blended/stabilized distributed water) cannot be practiced at risk of increasing microbial concentrations in finished water. Post-disinfection required under regulation, is recommended as a safety measure and for residual maintenance. Other post-treatments may include degassing of CO <sub>2</sub> or H <sub>2</sub> S, and pH adjustment.

Courtesy of National Drinking Water Clearinghouse, Morgantown, WV.



**FIGURE 28.13** Cylinder mounted chlorinator. (Courtesy of National Drinking Water Clearinghouse, Morgantown, WV.)

The chemical reactions of water treatment through chlorination are somewhat complex. A discussion of some of the associated terminology follows for clarification. “Chlorine dosage” or feed rate is the amount of the disinfectant fed into the treatment flow stream expressed in mg/l, or parts per million (PPM), which are equivalent. The water being treated has impurities that combine with some of the chlorine. These impurities, such as organic matter or ferrous iron, exert a “chlorine demand” that necessitates increased dosage to both satisfy this demand plus provide chlorine for disinfection. Chlorine left over for disinfection is referred to as “available.”

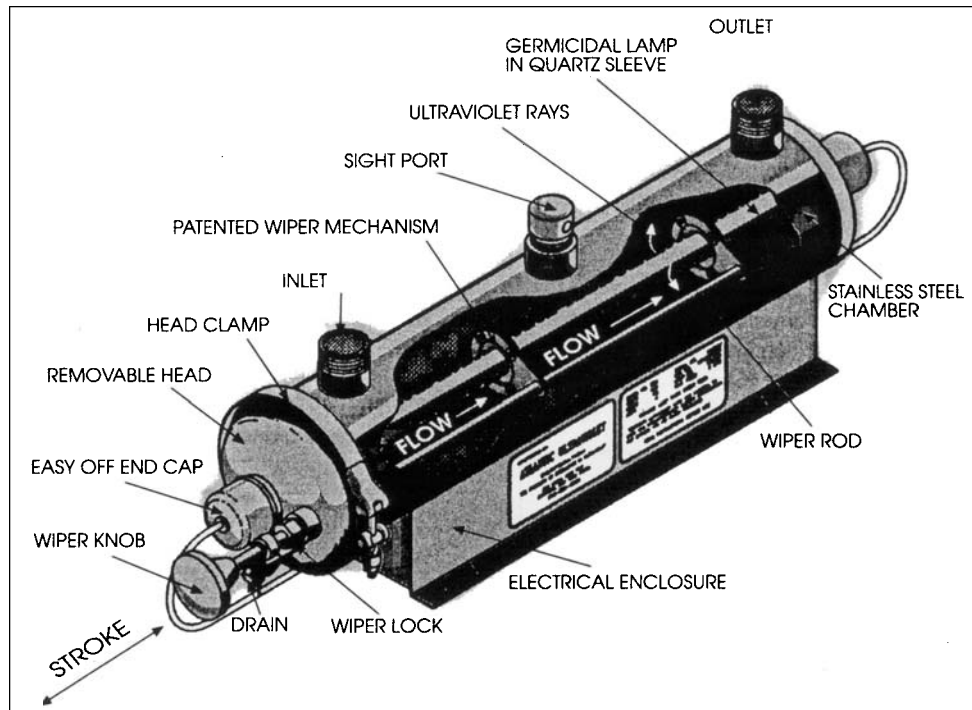
“Combined available chlorine residual” refers to compounds of chlorine that have some disinfectant properties that are formed when chlorine reacts with ammonia or other nitrogen compounds. Chloramine, previously discussed, is an example. The most effective disinfectant form of chlorine is “free available chlorine residual.” This is the uncombined chlorine that remains in the water after any combined residual has formed. “Total available chlorine residual” is the total concentration of chlorine compound available to act as a disinfectant, including both combined and free-chlorine residuals.

“Chlorine contact time” is the period of time from when chlorine is first applied and when the chlorinated water is used. The contact period required to ensure effective disinfection depends on chlorine dosage, type of chlorine residual, temperature, and pH of the water. The SDWA SWTR requires that chlorine dosage be adequate to fulfill chlorine demand and provide free available chlorine residual after chlorine contact time has been satisfied, before the water reaches a consumer.

The SWTR requires certain minimum CT values, where the residual concentration “C” of a disinfectant in mg/l is multiplied by the contact time “T” in minutes. State agencies with authority to enforce the SWDA rules can provide particular details and requirements for CT calculations. These calculations often require that additional finished-water storage tankage be installed, or that existing tankage be baffled to prevent short-circuiting. This is to insure that the disinfectant has adequate contact time before the first customer uses the water.

- *Ultraviolet light (UV).* Ultraviolet light radiation is produced by a special lamp arranged so that it irradiates water as it flows by (see Figure 28.14). It provides disinfection by penetrating the cell wall of an organism to disrupt the genetic material such that it is no longer able to reproduce. UV is very effective in destroying viruses and bacteria in water, and can be used as a primary means of disinfection. However, it provides no residual effect, and a secondary disinfectant must be used in conjunction with UV to prevent micro-organism regrowth in distribution systems. UV disinfection produces no known toxic residual and requires only a brief contact time. It is not, however, suitable for water with elevated turbidity, suspended-solids content, color, or soluble organic matter that can interfere with the passage of light. UV radiation is not reliably effective against giardia or cryptosporidium cysts, so they must be removed physically from the flow by upstream processes.

- *Ozonation.* Like UV, ozonation provides no residual, and a secondary disinfectant (such as chlorine) must be used with it to protect against pathogens in the distributions system. However, ozone is a powerful pri-



**FIGURE 28.14** Ultraviolet water purifier. (Courtesy of National Drinking Water Clearinghouse, Morgantown, WV.)

mary disinfectant that requires a shorter contact time and lower dosage than chlorine.

Ozone is generated by passing dry air through high-voltage electrodes to form a powerful disinfecting and oxidizing agent. It is unstable and, therefore, must be generated on site. It has a low solubility in water and must have an efficient contact arrangement. Even so, an excess-ozone destruction unit is necessary to prevent this toxic gas from being released from the contractor near operations personnel.

Figure 28.15 shows a schematic diagram of typical ozone treatment for a small water system. Ozone-package units are available for small water plants, but power consumption is high and operation and maintenance is fairly complicated and costly. Ozonation can produce some undesirable halogenated organic materials, but much less than disinfection by chlorination.

### Corrosion Control

Corrosive water attacks the structural integrity of the distribution system, causing leaks and adverse effects on the finished-water aesthetics. It also causes trace quantities of hazardous metals, mostly lead and copper, to be leached from both the distribution system and household fixtures into the consumable water, potentially affecting the health of consumers. The lead and copper rule of the SDWA requires a testing program, based on population, and actions to reduce corrosiveness of the water when testing indicates

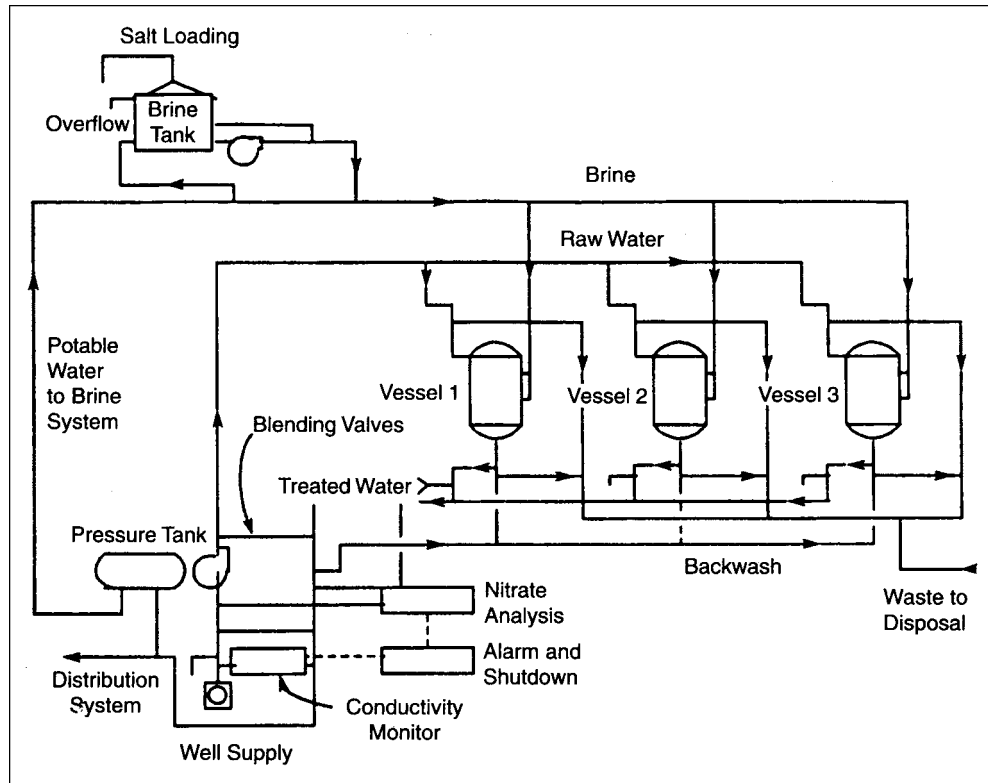
it is needed. Corrosiveness can be reduced by changing the characteristics of the water. Common methods are alkalinity and pH adjustment and lime softening. Chemical handling and feeding facilities need to be included in treatment facilities with adverse water characteristics.

### Fluoridation

Fluoridation of water to reduce tooth decay in children is a common practice in the United States. The three chemicals used for this purpose are sodium fluoride, fluorosilic acid, and sodium fluorosilicate. Fluorosilic acid is available as a corrosive liquid, while the other two are available in crystalline or granular form. These chemicals can be fed by solution or dry feeders, and the dosages must be carefully controlled. Too much fluoride can cause teeth to become pitted and mottled, while too little has no effect.

### Package Plants

Package plants offer many advantages over conventional in-ground, custom-treatment plants for small systems. The main difference between custom and package plants are that package units are factory-assembled and mounted on skids for transport to the site, while custom plants, as the name implies, are constructed at the site. Not only are the package units more economical to construct, but they are specifically designed to minimize operation attention. This makes them ideal for application at small systems where full-time, trained operators are not usually available.



**FIGURE 28.15** Schematic of a system that provides primary disinfection with ozone, filtration, secondary disinfection and excess ozone gas destruction. (Courtesy of National Drinking Water Clearinghouse, Morgantown, WV.)

Package plants are appropriate in plant sizes from 25,000 gpd to 6,000,000 gpd. They can be used to treat ground water for contaminants, such as iron and manganese, or surface water with a wide range of characteristics. The influent-water quality is the most important consideration when selecting a package unit. However, package units are designed to treat water in a certain range of parameters, and it is important to select the appropriate unit for the range of influent characteristics of the water to be treated.

The SWTR has greatly increased interest in package units for small communities. Construction and maintenance of conventional custom plants that meet the new regulations poses financial challenges that can be somewhat relieved by use of package plants instead.

Package units, which are able to compactly incorporate conventional processes, are widely used in small systems to treat surface and ground water sources. However, many of the advanced processes are also available in package units, which can be used in combination with conventional processes to provide comprehensive water treatment.

The consulting engineer responsible for designing the water-treatment facilities for land development projects would do well to strongly consider package units for best economy and results.

### Residuals Disposal

Water-treatment plants generate residual wastestreams requiring handling and disposal facilities that must be planned during the design of the facilities. Some treatment processes result in residuals that are not particularly difficult to handle. For example, debris separated by initial raw-water screening, such as leaves and limbs, can be taken to conventional solid-waste landfills. Residual sludges formed when suspended solids in the source water are coagulated and settled can be disposed of in a variety of ways. They can be discharged to a sanitary sewer, if available, dewatered by a mechanical device (i.e., a sludge press or centrifuge), or discharged to a lagoon. However, decant from sludge lagoons discharged to surface water must have a National Pollution Discharge Elimination System (NPDES) permit. The solids that are held in lagoons must eventually receive final disposal. They can either be hauled in liquid form to be applied on land or be processed at a wastewater treatment plant. Lagooned solids that have been held long enough will become de-watered sufficiently so that they can be excavated and hauled to landfills or permitted land-application sites.

As drinking-water quality becomes more stringently regulated, it is reasonable to expect that contaminant-removal levels will increase. This will result in the use of more com-

plicated treatment processes and more complex and possible hazardous residuals. For example, brines resulting from ion-exchange media regeneration and solids from plants treating water with radionuclide content may require special permits and handling methods for disposal.

## REFERENCES

- American Water Works Association, *Design and Construction of Small Water Systems*, 2nd Ed., Denver, Colo: American Water Works Association, 1999.
- Babbitt, H. E., J. J. Doland, J. L. Cleasby, *Water Supply Engineering*, 6th Ed., New York: McGraw Hill, 1962.
- Cooley, R. L., J. F. Harsh, and D. C. Lewis, *Hydrologic Engineering Methods for Water Resources Development, Vol. 10, Principles of Ground Water Hydrology*, Davis, Calif.: Hydrologic Engineering Center, 1972.
- Roscoe Moss Company, *Handbook of Ground Water Development*, New York: John Wiley & Sons, 1990.
- Virginia Department of Health, *Waterworks Regulations*, Commonwealth of Virginia, Richmond, VA., 1993.
- U.S. Environmental Protection Agency, *Drinking Water Treatment for Small Communities*, EPA/625/5-90/025, 1990.
- National Small Flows Clearinghouse, *Pipeline*, Winter 2001, Vol. 12, No. 1. West Virginia University, Morgantown, WV.
- National Drinking Water Clearinghouse, *Ion Exchange and Demineralization, Tech Brief Four*, West Virginia University, Morgantown, WV., 1997.
- National Drinking Water Clearinghouse, *Membrane Filtration. Tech Brief Ten*, West Virginia University, Morgantown, WV., 1999.
- National Drinking Water Clearinghouse, *Disinfection, Tech Brief One*, West Virginia University, Morgantown, WV., 1996.

## CHAPTER 29

# EROSION AND SEDIMENT CONTROL

*John Matusik, P.E. / Edited/Updated: Timothy C. Culleiton, P.E.*

### INTRODUCTION

Erosion is the detachment and transport of sediment particles caused by the forces of wind, rain, ice and gravity. Sediment, the by-product of the erosion process, is the solid material composed of organic and mineral particles that is in suspension or transport or has been relocated from its origin. Deposition of these particles is sedimentation.

Natural erosion is a slow, uniform process essential for maintaining a balance between plant and animal life and is responsible for the creation and evolution of natural landforms. Land-disturbing activities, such as logging, mining, farming, and construction, greatly accelerate the natural erosion process. Such activities denude large areas of vegetation, loosen and disturb the soil, increase the length and gradient of land slopes and alter the natural surface-runoff patterns. Such extreme and rapid changes in the natural landforms can increase the erosion rate from two to several tens of thousands times the natural rate existing before the activity began.

Short-term effects of erosion and sedimentation are recognizable near a denuded site immediately after a storm in the form of muddy runoff and sediment deposits on pavement. The long-term effects, such as reduced hydraulic capacity of streams and degraded water quality, take years before they are recognized. Additionally, the long-term effects of a denuded site extend downstream to the lower reaches of the drainage shed. These effects take the form of reduced hydraulic capacity in streams and culverts due to sedimentation, which creates higher flood elevations. Reduced water quality affects aquatic life and recreational activities.

Specific types of controls can be incorporated as part of any land-disturbing activity or operation to reduce exces-

sive erosion and sedimentation. Erosion controls limit the amount of eroded soil, and sediment controls prevent the release of the eroded sediment from the site into the natural environment.

A distinction should be made between the terms “silt” and “sediment.” The two words frequently are used interchangeably, although this is incorrect. In the classification of soils, silt is the combination of microscopic soil particles that consist of fine quartz grains and micaceous minerals. Unlike silt, which is a classification of particle size, sediment is a relative term. In large rivers with high energies, the particles transported may range from several inches in diameter down to several microns. In small, weak streams, the size of the particles transported may be on the order of 200 microns and smaller. Despite the significant difference in size, all of the transported particles are considered sediment.

Most of the research for cause, effects and mitigation of soil erosion focuses on erosion caused by water. However, in dry climates and during the dry season in other climates, soil erosion from wind can be significant and should not be ignored. The scope of this chapter focuses on the cause, effects, and mitigation of soil erosion as it relates to land development construction sites.

### CONSEQUENCES OF EXCESSIVE EROSION

Various types of nutrients and toxicants become attached to the surface of soil particles and, if carried into streams and lakes, their effect can be detrimental to aquatic ecosystems. Other residual effects of sediment include reduced hydraulic capacity of streams resulting in higher flood elevations, loss of storage volume in lakes, increased turbidity, and smothering of ecosystems residing on the bottom of



the channels and lakes. On an even more local scale, excessive and uncontrolled erosion causes damage to adjacent properties such as lawns and landscaped areas, trees, sidewalks, and walkways, and this in turn creates a bad relationship between residents and the developer or contractor.

### Water Quality

The highest concentration of natural soil nutrients, such as phosphorus, nitrogen, and their compounds, reside in the upper layers of soils. Clearing and grading operations expose much of these surface soils so that surface runoff carries the nutrient-laden sediment into the streams and ponds. The adverse effects of excessive sediments and adsorbed compounds can extend to the ecosystems at the water's surface, which rely on the water as part of their life support. The injection of excessive sediments and colloids into the watercourse increases the turbidity, which reduces the amount of sunlight dispersed throughout the water. Since most aquatic vegetation and organisms require nominal amounts of sunlight for energy and growth, the change in turbidity affects the type and amount of plants and organisms that can survive. This, in turn, affects other creatures higher in the food chain.

Additionally, decreased sunlight in the water creates fluctuations in the dissolved oxygen content and creates thermal stratification in the water. Aquatic life thrives within small variations of thermal conditions and requires specific amounts of dissolved oxygen. Their existence depends on a stable environment. Increased turbidity reduces the temperature of the water near the surface. The cooler, denser water sinks to the lower levels displacing the less dense, warmer layer of water. The lower layer of water carries nutrient materials and light sediments with it as it moves to the surface. The increased concentration of mineral and organic nutrients in the water coupled with the wide fluctuation of dissolved oxygen produces an aquatic environment conducive to plants rather than aquatic animals. The transformation of the water from low concentrations of vital nutrients, with stable, wide variations of life forms, to water with increased nutrient concentrations and fluctuation of dissolved oxygen, substantially reduces the types of life forms that can exist in such conditions, the consequences of which are a notable reduction in water quality.

Nitrogen, phosphorus and other biostimulants added to natural waters promote algae and bacteria growth. A chain-reaction begins with the proliferation of algae due to the abundance of phosphorus. The algae propagate, die off, and fall to the bottom of the pond. On the bottom, bacteria decompose this material consuming dissolved-oxygen in the process. Because of the reduction in dissolved-oxygen levels, aquatic organisms are either driven away or die off as the excessive suspended sediments settle out of the water and the bottom vegetation becomes buried. The adverse effects on fish include clogging of their gills and the suffering of abrasions, both of which severely reduce their ability to subsist in this environment.

### Hydraulic Capacity and Storage

Alluvial channels are those watercourses that contain clays, silts, and sands in the bed and bank linings. This includes most natural streams. The silt and sand lining is in constant motion. The stream lifts and carries the particles for short distances, less than 1 ft, or for considerably longer distances, depending on the transport energy. The distance depends on the size, shape, and density of the particles relative to the transport energy of the stream. The transport energy of the stream is, in part, associated with the channel's discharge, velocity and geometric configuration.

Despite the constant motion of the lining, many reaches of natural streams are in a state of equilibrium over a given period. That is, the channel is considered stable with no objectionable silting, scouring or sedimentation within the reach. When the water/sediment discharge balance is disturbed, the stream compensates for the imbalance by aggrading or degrading the bed and banks. If the imbalance causes the stream section to carry less sediment than naturally supplied, the bed and banks begin to erode (degradation) to make up for the deficit. Conversely, injecting the stream with more sediment than it can carry causes deposition of the sediments (aggradation). In each case, the channel's hydraulic characteristics change, and this affects the floodplain and hydraulic structures along the watercourse.

Aggrading sections tend to raise water-surface elevations, the consequences of which are most damaging during floods. As the sediments deposit and fill in the stream section, the normal water-surface levels are forced to rise to higher flood levels to obtain the required conveyance. The higher water elevations then, in turn, erode the unstable upper portions of the banks. This compounds the environmental problem by adding more sediment to the stream and drastically altering the section. Subsequently, the same volume of water from the more frequent storms begins to overflow the banks and expand the limits of the floodplain; the resulting effects from the localized aggradation and degradation are relayed upstream and downstream. Additional sediments transported downstream tend to block stream crossings and reduce the capacity of the stream itself. Upstream, increases in water surface because of the backwater effects from the floodwaters can occur.

Many land development projects incorporate lakes and ponds into the design to control post development runoff. Proper operation of these ponds depends on adequate storage volume to accommodate the excess runoff. The excess sediment accumulation displaces the water to higher levels and affects flood stage as well as the water quality. If these small ponds are not designed to accommodate excessive sediment loads, maintenance costs for dredging remediation are significantly increased.

### SOIL EROSION AND SEDIMENTATION PROCESS

The natural soil erosion and sedimentation process is an important factor in the formation of the surface features of

the earth. Features such as the Grand Canyon, badlands of the Dakotas, escarpments on mountains, fertile soils and delta areas along rivers are the result of erosion. The erosion process contributes to a variety of vegetation growth, which, in turn, supports the various animal species. These surface features, plant growth, and animal habitats are the natural result of a slow, uniform process occurring over geologic time. Because of the long length of time for this change to occur, the ecosystems are able to adapt gradually to the changes. Uncontrolled accelerated erosion has damaging effects on the environment and compromises aesthetic values important to development.

The National Pollution Discharge Elimination System (NPDES) discharge permit requirements that evolved from the Clean Water Act require a sediment and erosion control plan as part of the design process. Design of a soil erosion and sedimentation plan for a development project requires a fundamental knowledge of the process itself. With a basic understanding of the principles of soil erosion and sedimentation mechanics, the engineer can then plan and design efficient methods for erosion and sediment control.

Soil erosion and sedimentation is a three-step process. First, the soil particles must be detached from the soil mass. Water (or wind) must then contain sufficient energy for entraining and transporting the particles. Finally, deposition occurs when the energy of the water (wind) is less than the fluid shear resistance and gravitational forces acting on the sediment particles. The key to erosion and sediment control is to prevent detachment and reduce the velocity of the runoff.

### Detachment and Transport

Detachment of soil particles from the soil mass often begins when the energy of a raindrop hitting the ground is sufficient to break down the soil aggregates and project them into the air. Upon impact, these ejected particles then contribute to the detachment of even more soil particles. Soil particles can be ejected 2 ft into the air and may be displaced 5 ft in the horizontal direction. The detachment rate of the soil particles is a function of both the kinetic energy of the raindrop and the force holding the soil particle in place. Detachment occurs when the kinetic energy of the raindrop is greater than the total energy that holds the particle in place. The basic components of the particle energy are the intragranular-frictional-resistance, gravity, and chemical bonding. The most easily detached particles are the medium and coarse non-cohesive particles. Cohesive clay particles have the highest resistance to detachment because of the strong, intraparticle ionic bonds. The raindrop must carry enough kinetic energy to overcome this intraparticle bond for detachment to occur.

Raindrops vary in size, with the largest diameters on the order of  $\frac{3}{16}$ -in. (5 mm). The velocity of the raindrop depends on its size. As the diameter increases, the terminal velocity increases up to a maximum of 30-mph (9 m/s) for the larger raindrop sizes. Velocity and size are related to the

storm intensity. Raindrops increase in size as storm intensities increase from low to moderate intensities. However, for higher-intensity storms, raindrop sizes decrease.

The transport process begins with entrainment of the soil particle. That is, the soil particle is “captured” and pulled along by the combined effects of turbulence and momentum of the fluid. Heavier-sediment particles near the bottom of the flow mass move by bouncing, sliding or rolling. Flow turbulence lifts the lighter sediment particles and transports them in suspension. Whether the particle moves in suspension or along the bottom of the flow mass depends on the velocity and turbulence of the water and the physical characteristics of the particles themselves.

Rainfall not only acts as a detaching agent at impact, but also as the transporting agent as runoff in the erosion process. The common factor in detachment and transport is the energy of the agent. At impact, the kinetic energy of the raindrop depends on its velocity and size. These parameters are interrelated with the volume and intensity of the storm.

The following summarizes the relationship between raindrop diameter, rainfall intensity, and velocity:

- As raindrop diameter increases, rainfall volume increases.
- As rainfall intensity increases, the average raindrop diameter increases for lower-intensity storms and decreases for higher-intensity storms.
- As raindrop diameter increases, the terminal velocity increases up to the limiting diameter of 6 mm.

### Sheet Flow

After the soil has been fully saturated, the rain accumulates into small depressions. These depressions are on the order of only several raindrop diameters deep. As the depression capacity is exceeded, the first phase of runoff, referred to as sheet flow, begins. On a macroscopic scale, sheet flow appears as a very shallow uniform depth flowing over the area. On a scale of a raindrop, it is a series of braided watercourses flowing in very small, unapparent channels. The depth of flow is slightly larger than several raindrop diameters. This type of flow usually lasts no more than several feet and only during the early part of the rainstorm. Typical velocity for this sheet flow is around 1 fps.

### Rill Flow

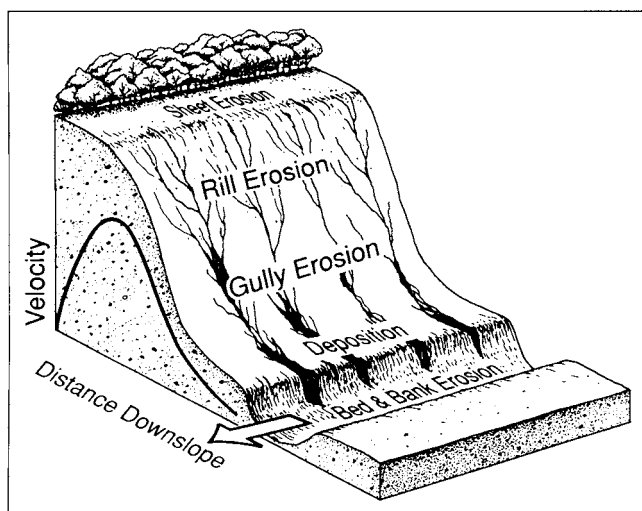
Water transforms from sheet flow into rill flow as it moves down-slope. As sheet flow depth increases beyond several raindrop diameters, the runoff begins flowing in more defined watercourses several inches deep. This concentration of flow carries more momentum than sheet flow. Consequently, the erosive power and turbulence of rill flow is much higher than sheet flow. The relationship between velocity, erosive power and transport capacity of flowing water is such that when the velocity doubles, the water breaks loose four times more soil and transports 32 times more

soil. Much of the soil erosion and transport of sediments begins during the rill flow.

### Gullies

The next phase in the erosion process is the formation of gullies. Gullies are significantly larger than rills. These steep-sided watercourses with near vertical scarps at the head may be several feet to 15 ft (and larger) deep and twice as wide. They may be hundreds of feet long. Gully formation comes about from the merging of many smaller depressions. Overland flow and rain splash and soak the soil to the point where the sides of these depressions erode away and slough to the bottom of the depression. As the sides of adjacent depressions wear away, the depressions combine as a larger one. The gully enlarges as the up-slope water flows through the gully and further deepens it. Flows entering the head of the gully have very high erosive energies. Erosion in this part of the gully causes the movement of the gully to progress further up-slope. Figure 29.1 shows the relative velocity during the various stages of the erosion process.

In land development projects, continuous grading and construction operations prevent gully erosion, that is, unless the site sits unprotected and remains unvegetated for very long. Typically, this is not the case if judicious land planning management is exercised during construction. Most eroded soil at construction sites occurs from sheet and rill erosion. Studies have shown that construction sites can produce 1 yd<sup>3</sup> of sediment per 100 ft<sup>2</sup> of cleared areas during a moderate-storm event. Estimates of soil erosion rates for a highway construction project in Fairfax County, Virginia range from 126 to 151 tons/acre/year. This compares to estimates of a natural erosion rate of 0.18 tons/acre/year in forestland, and estimates of agricultural erosion rates range from 0.3 to 6 tons/acre/year.



**FIGURE 29.1** Relationship between runoff velocity and stages of the erosion process.

Soil erosion and the transporting of soil particles is a progressive yet discontinuous process. The soil particles are transported a fixed distance based on the energy of the flowing water. Eventually, the particles will again be entrained and transported further downstream when higher erosive velocities recur.

The following statements summarize the relationship of soil erosion with rainfall characteristics:

- Average soil loss per rain event increases with the intensity of the storm.
- Most erosion occurs from moderate-storm events producing 1 to 2½ in. of rain.
- The erosive power of water is related to the volume and velocity created by the storm. Any storm that effectively causes an increase of either of these two parameters will pose a high erodibility risk. Erodiability is the susceptibility of a soil to erosive forces.

### FACTORS INFLUENCING SOIL EROSION

A soil's propensity to erode not only depends on its characteristics but also on the type of vegetation, topography, and hydrologic conditions in a soil's region. The variability and non-deterministic nature of these factors adds to the difficulty in quantifying soil erosion to a high degree of certainty.

#### Soil Characteristics

Rainfall and flowing water erode the soil only if the energy is high enough to dislodge and entrain the soil particles. The energies that hold the soil particles in place depend, in part, on the soil texture and soil structure. Soil structure is the geometric arrangement of soil particles with respect to each other, that is, how the soil particles interlock with each other. Factors that affect the soil structure are the size, shape, mineralogical composition and moisture content of the particles. Soil texture, in the engineering sense, is a measure of the particle size and gradation. It is the proportion of the sand, silt and clay content within the soil. Different organizations have developed different methods to classify soils and particle sizes. Soil grain sizes as per various organizations are shown in Table 39.12.

Energy from rainfall impact and runoff must overcome the frictional resistance and any chemical bonding between the particles. Once this threshold energy is reached, the minimum energy to keep the particle in suspension or moving is considerably less. For those particles that move through rolling-bouncing-sliding motions, the more round the particle the less energy required. The minimum threshold energy required to start a particle in motion depends on the physical and chemical characteristics of the particle, and these are directly related to the soil structure and texture. Non-cohesive granular soil particles (e.g., sand) require the least energy for incipient motion. Clays and silts are platy and have strong chemical bonds. Thus, they have a higher threshold energy. Although clays have the highest

threshold energy for incipient motion, their size (2 microns) makes them easily transportable once they are in motion.

Two other factors that directly affect the erodibility of the soil are infiltration and permeability rate. Generally, those soils that have high infiltration capacities and permeability rates are less erodible. Since these soils have high infiltration and permeability rates, the amount of runoff is reduced, as is the available energy to the surface soil particles. However, storms of sufficient duration will eventually exceed the soil's infiltration capacity. Once this happens, the soil becomes saturated and the erosion-resistance of the high infiltration and permeable soil is temporarily lost.

The following statements summarize the relationship between erodibility and soil texture:

- Soils containing high percentages of silts and sands are the most erodible. Their erodibility is magnified when these soils appear on steep slopes.
- Clean, well-drained, well-graded gravels are the least erodible. These soils have a high resistance to erosion because of their combination of cohesive and intergranular strength.
- Higher-clay-content soils are less erodible except the unstable clays, such as illite and montmorillonite, which have low infiltration capacities and are highly susceptible to erosion.
- Soils with high organics are more permeable, which allows for infiltration of higher amounts of rain water, and are more resistant to detachment.

### Vegetation

Vegetation is perhaps the most important factor in reducing soil erosion. Leaves, branches, and stems reduce the kinetic energy of the raindrop and thus reduce detachment. Some forms of vegetation are more effective at dampening the kinetic energy than others. The structure of the vegetation must be sturdy enough to absorb the kinetic energy of the raindrop.

Not all vegetation is beneficial in reducing soil detachment. Droplets of rain falling from saturated foliage also contribute to detachment. Although the foliage intercepts the initial kinetic energy from the precipitation, the subsequent kinetic energy of the droplets from the foliage often times is larger. Normally, raindrops falling from saturated foliage are larger than the initial raindrops due to cumulative effects of the water collecting on the foliage. Hence, the kinetic energy of these drops could be larger than the initial raindrops, assuming terminal velocity is attained in both cases. Raindrops falling from heights of about 20 ft attain terminal velocity. Therefore, any vegetation that initially absorbs the kinetic energy of the raindrop and then imparts the same kinetic energy to a raindrop is not very effective in reducing detachment. Ideally, the most effective vegetation would be strong enough to absorb the initial kinetic

energy and close enough to the ground such that the subsequent kinetic energy of the collected secondary droplets is minimal.

At the ground surface, the organic litter and other plant debris reduce erosion in two distinct ways: they act as an additional energy-absorbing barrier, and the layer and the shade from the foliage canopy reduce soil moisture evaporation. Moderate amounts of soil moisture enhance the stability of the soil aggregates.

Below the ground surface, a root system acts as a deterrent for erosion. Nearly 75% of a root system is located in the upper 12 in. of the soil. A root system, along with surface debris, roughens the ground surface. These ground surface irregularities reduce flow velocity and dissipate flow energy, the consequences of which are a decrease in erosivity, and the deposition of entrained particles. The increase in depression storage from the irregularities reduces runoff volume, and this reduces erosion potential.

### Soil Erosion on Slopes

Soil erosion rates are higher on slopes, with the highest erosion intensities occurring on convex slopes. Factors such as slope length, gradient, and geometric configuration all affect a slope's erosion potential. Because of the slope, rain splash dispenses more soil particles down-slope than up-slope. The amount dispensed down-slope increases with slope gradient. For slopes with a uniform gradient, the volume and velocity of the runoff increase as slope-gradient and slope-length increase. The tangential stress of the flowing water combined with the gravity component of the weight of the water in the direction of flow greatly increases erosion potential.

### Soil Erosion from Snowmelt

Although the impact of falling snow does not contribute to the detachment of soil particles, the contribution to erosion results from the snowmelt and the freeze-thaw cycles at certain times of the year. As snow melts, it saturates the surface layer of the soil. If the immediate sublayers of the soil are nearly frozen, permeability and infiltration capacities are significantly reduced. During a thaw, the top layer of the soil becomes heavily saturated from the snow melt, and the highly saturated soil slurry easily washes down-slope with the snowmelt. At the next freeze-thaw cycle, the process repeats itself with the next available soil layer.

### FACTORS INFLUENCING COASTAL EROSION

Nature's ability to shape the landscape is most apparent on the coastline. The forces of waves, tides, and currents are constantly and sometimes dramatically altering the shoreline. Variables that influence the erosion process include the frequency and intensity of storms, the geologic condition of the coast, and the characteristics of the near-shore sea bottom. Coastal erosion, like soil erosion and the sedimentation process, is a natural process that shapes the features of the earth. As discussed previously, land development can

accelerate the process of soil erosion, and this can lead to a degradation in water quality, an increase in flood hazards, and a reduction in hydraulic capacity and storage. Likewise, development can accelerate the coastal erosion process by increasing flood hazards and the potential for property damage. Despite the complex dynamics of the erosion process along the coastline and the development hazards it creates, the demand for view and waterfront property creates strong pressure for development.

The United States' 30 coastal states contain approximately 85% of the country's population, and over 50% of the population reside within 50 miles of the coast. According to a study published by the Federal Emergency Management Agency in June, 2000, approximately 25% of homes and other structures within 500 ft of the U.S. coastline and the shores of the Great Lakes will fall victim to erosion in the next 60 years. Property owners within the first few hundred feet of the nation's coasts face as large a risk of damage from erosion as they do from flooding. Development in several high-risk coastal areas studied has increased 60% over the last 20 years. This population explosion has resulted in development of property once considered to be swampland, or too costly to develop, while the type of development has evolved from rustic beachfront cottages to multi-story, million dollar residences.

Coastal environments typically undergo continual natural change, and coastlines routinely move back and forth as a result of natural processes. If erosion is not balanced by accretion, the shore will be washed away. Coastal lands may experience long-term erosion under some conditions. Rising sea levels cause a beach to migrate landward or the beach will drown. This causes the land behind the beach to erode. If the amount of sand from the seaward side is reduced, a beach will erode the land behind it to maintain a constant supply of sand. Many scientists believe global warming is causing sea-level to rise. While these projections are uncertain, a rapid rise in sea level would result in substantial increases in coastal erosion.

## Waves

Wave motion is the most important force in the natural process of accretion and erosion of a shoreline. Waves form as a result of deep ocean currents, winds and the rotation of the earth. Waves carry their energy to the shore where they break and release their energy at the interface of land and water. The characteristics of a wave depend on the speed of the wind, its duration, and the unobstructed water distance (or "fetch") it blows over. As waves break, run up the shore and return, they carry sedimentary material, or littoral drift, on-shore and off-shore. Most waves arrive at an angle to the shore and setup a longshore current, moving littoral drift in a series of zigzags as successive waves advance and retreat.

High waves, caused by storms, will cause a beach to change its shape. To absorb the additional energy of the high waves, beaches and dunes will give up sand to the waves, which carry it seaward and drop it on the bottom.

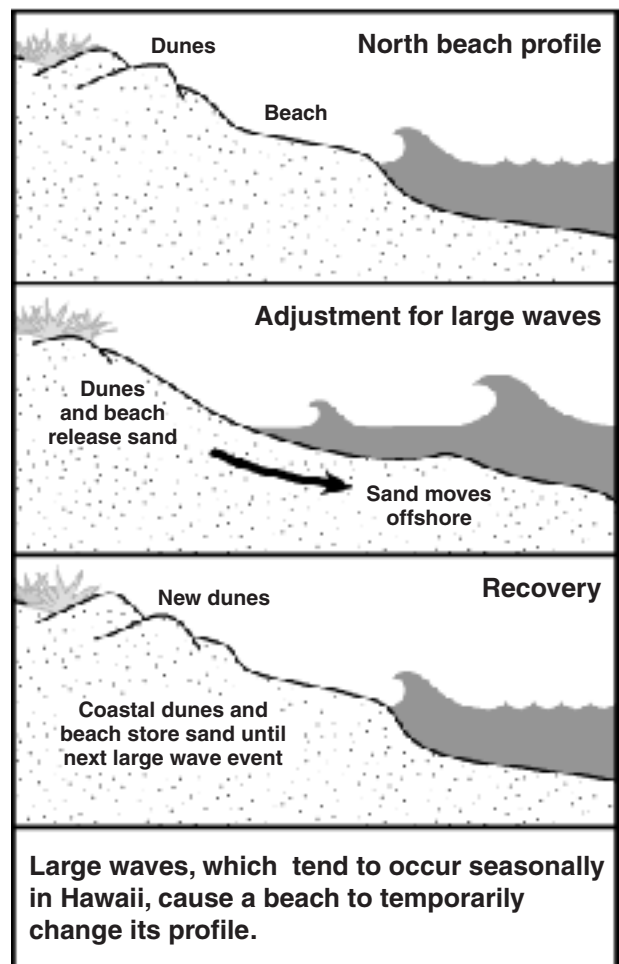
This raises the seafloor and flattens the overall profile of the beach. Waves then shoal and break further offshore, minimizing their erosive effects. Over time, after the storm, normal wave action reshapes the beach, re-building the dunes (see Figure 29.2).

## Tides

The gravitational effects of the sun and the moon in relation to the earth cause tides. The ebb and flow of tides produces regular changes in the level of the sea along the coast and generates tidal currents. The influence of tides on beaches depends on tide range, which determines the zone over which wave action can operate.

## Currents

Currents, the movement of the oceans because of specific forces acting on the water, can carry large amounts of sedimentary material. Currents may or may not play a part in beach erosion, depending on the type of current and its location. Tidal and wave currents provide the greatest impacts; however, coastal currents when close to shore may contribute to beach erosion. Currents close to shore are



**FIGURE 29.2** Seasonal beach profile adjustments. (Source: State of Hawaii, Department of Land and Natural Resources' Website)

critical to beach drift, which is the lateral transport of beach sediments.

### Human-induced Factors

Erosion can be accelerated by human alterations to the coastline. Hardening a coastline can interfere with the necessary beach profile adjustments because the dune can no longer share its sand with the beach. As a retreating beach encounters a seawall or revetment it can no longer draw upon the landward sand supply and it begins to erode. Often, attempts to reduce erosion and build up one area of the shoreline will result in reduced accretion elsewhere, reducing another shoreline. Human-induced impacts to the natural vegetation on a dune system reduces the roots and stems of the vegetation to trap fine soil and sand particles and decreases the stability of the dune.

### EROSION-CONTROL BEST MANAGEMENT PRACTICES<sup>1</sup>

Erosion-control best management practices limit the amount of soil eroded from a site and indirectly control the sedimentation process as well. The objective is to enhance the site in ways to control the energy, velocity, and volume of runoff.

Erosion control best management practices fall into three general categories:

1. *Site management measures* limit the amount of erosion by controlling the construction activities through judicious site planning and phasing of construction operations.
2. *Soil stabilization measures*, either vegetative or non-vegetative, decrease the erosion potential of the soil.
3. *Structural measures*, which take on various forms of devices, intercept and divert the runoff to stabilized collection areas or dissipate runoff energy.

Each of the above measures can be divided into temporary and permanent measures. Temporary measures function for durations no longer than the construction period. Some erosion-control best management practices are specifically designed to function for periods less than the entire duration of the project. Their purpose is to control erosion only until the immediate area is stabilized before they are removed. Although these temporary measures usually have low initial costs for installation, they have high maintenance costs due to the non-durable materials used.

Permanent measures, which have higher initial costs for construction, are justified on the basis of reduced maintenance costs and the longevity of the structure. The higher costs are attributable to the more durable materials and the

detail involved in the design and construction of the device. The type of control selected to serve a function is commensurate with the type of construction and the physical characteristics of the site.

### Site-Management Measures

Site management pertains to the general planning and scheduling of construction operations and sequencing land disturbance areas to reduce the site's erosion potential for the entire term of the project. Site management also includes identifying specific times for stabilizing areas and maintenance of the controls. Typically, this aspect is done in note form on one of the erosion and sediment control-plan sheets.

The following are typical topics addressed in site-management measures.

Plan site development to avoid highly erodible areas and to preserve natural drainage features and environmentally sensitive site features.

- Recommend a limit to the amount of land area to be denuded at any one time.
- Recommend the specific sequencing of disturbed areas. (This is typically applicable to larger projects.)
- Recommend a phasing of construction operations incorporating the previous two recommendations.
- Cite specific areas that should be immediately stabilized because of high erosion potential or off-site damage potential. For example: steep slopes; long, moderately steep slopes; areas near streams and areas near the perimeter of the property.
- Specify a maintenance schedule for the inspection of devices and clean out of sediment traps and basins.

**Site Planning.** During site development, measures can be implemented to reduce the erosion potential of a site both during and after construction. However, the first and by far the most effective measure is reducing the erosion potential of a site. Careful site planning can eliminate many potential erosion and sedimentation problems by preventing them from occurring in the first place. Erosion rates can be controlled by limiting impacts to steep slopes (slopes greater than 15%) and highly erodible soils. Consider the use of retaining walls to reduce the development footprint and to avoid impacts to these areas. Preserve natural drainage features and environmentally sensitive areas, thus preserving natural vegetation and other areas that filter runoff and reduce erosion potential. Create and retain buffer zones to the environmentally sensitive areas to protect these areas from the adverse impacts of development; prohibit clearing and grading within post-development buffer zones.

Clustering of units in a smaller land area is a planning tool that allows for the implementation of more effective perimeter-erosion and sediment controls during construction, preservation of natural drainage features and environ-

<sup>1</sup> Some localities refer to best management practices when describing measures taken to remove soluble chemicals from stormwater runoff. In a broader sense, many other localities and the Environmental Protection Agency refer to all measures designed to improve water quality as best management practices, including those with the primary purpose of removing suspended sediment from stormwater runoff.

mentally sensitive areas, and the reduction of impervious area with reduced road lengths and parking areas. Reductions in the amount of impervious area protect the site's natural ability to filter and convey storm runoff. The use of porous paving materials and the reduction of building setbacks, which reduces access road lengths, are additional measures of controlling impervious area and reducing impacts. The objective is to disturb the smallest area possible in developing the site to reduce erosion and off-site transport of sediment.

**Limiting Denuded Area.** Staged construction avoids area-wide clearance of a construction site by planning land-disturbing activities so that only the area under construction is exposed. The amount of disturbed land, to a limited extent, depends on how much land the builder needs to disturb to perform his task. An upper limit may be set by local design criteria. On one extreme, it is impractical for a builder to denude a 100-acre parcel if he can work effectively in only 5 to 10-acre sections at a time. The other extreme is witnessed when the amount of disturbed land is limited such that the builder is restricted so much that the ability to perform the task substantially increases construction cost. The amount of land that is workable on a steady basis by the builder varies from project to project and from builder to builder. Limiting the amount of denuded area at one time is practical for projects that have sufficient area to allow for such phasing.

Consider the 5-acre commercial project where the building and parking occupy most of the site. From the developer's perspective, the sooner the site is complete, the sooner the returns on investment begin. If the developer has to build the site in small phases, the additional development cost reduces the return on the investment. It is difficult to recommend a rule-of-thumb to use for the maximum amount of area allowed to be denuded. Most localities have a set limit, by ordinance, on the amount of area that may be denuded at any one time. Other localities limit the extent of time during which a disturbed area can sit unworked without stabilizers such as mulch or temporary vegetation. This limit is often set at 14 days, with reduced time limits (7 days) in critical areas, such as steep slopes. In those instances where there is no local criteria or where the criteria do not apply, the engineer must rely on experience and judgment. The engineer must be sensitive to both the environmental consequences and the cost-effectiveness of construction.

**Sequencing of Disturbed Areas.** On large or complex projects, an effective erosion and sediment-control plan may require a specific sequence for disturbed areas, in addition to limiting the amount of land to be disturbed. This requires the engineer to be familiar with the developer's program in order to coordinate the land-disturbing sequence with the program. In such cases, the engineer must be careful to detail a sequence that is most cost-effective with the client/builder's program. Typically, the sequence is such that an area is stabilized or other control measures are implemented before the next phase is begun.

Builders set their construction schedules based on a logical sequence of events; for instance, street pavement is not placed until the utilities are in and subbase prepared, and the walls of a building cannot go up until the foundation is ready. The schedule is subject to weather conditions and availability of materials and subcontractors. Most design engineers are not truly versed in the intricacies of this aspect of construction. Subsequently, it is unwise for such an engineer to try to tell the builder specifically how to perform the work. For this reason, the engineer should *only* recommend construction phasing/sequencing on the erosion and sediment control plans *or* meet with the builder/developer (and/or the contractor) to establish construction phasing/sequencing. For complex situations, the engineer may decide to meet with the developer before specifying construction staging or land phasing. During this meeting, the engineer can acquaint himself with the builder's or developer's approach for staging and phasing of the particular project. He can then incorporate the approach into the site-management measures, if and when practical or possible. However, this may not always be possible if the developer is not familiar with the construction techniques or if the builder has not been selected.

The erosion and sediment-control plan is subject to a public agency review-and-approval process. Once the plan is approved, it is costly to revise it. Revisions to approved plans must repeat the review-and-approval process. The public agency expects the erosion and sediment-control plans to be carried through as shown on the plan. Therefore, it is advantageous for the engineer and builder/developer to design an effective, workable site-management procedure at the outset.

### Soil-Stabilization Measures

Soil stabilization measures prevent erosion by absorbing rainfall energy, reducing overland flow velocity, increasing infiltration, and helping to retain soil moisture. Such measures are even used to bind the soil particles together to stabilize the soil.

**Ground Covers.** Ground covers include various vegetative covers such as grass, vines, straw, wood chips and shredded bark. Nets and mats are also a form of ground cover. Nets are a heavy, uniformly woven cloth, most notably jute yarn. Mats are porous, degradable material that roll out onto the surface. To be effective, nets and mats should be in contact with the soil surface and anchored securely.

The most favored form of vegetative ground cover, where sufficient rainfall is available, is grass. It is the easiest and fastest to install, and, relative to other forms of vegetative covers, it has low maintenance costs. Although the easiest way to generate grass is seeding, sprigging and sodding are other methods used to establish grass cover.

Of primary importance when installing the grass seed is the need to keep the seed from washing or blowing away. This can be ensured by proper soil preparation with fertilizers and roughening the soil surface to promote germination. Mulches are then added, as a temporary cover, to

protect the grass seed while it is germinating. Additionally, mulches provide temporary surface-soil stabilization until the grass can grow and stabilize the area. The most commonly used mulches are hay and straw. For maximum effectiveness, these mulches must be tacked, usually with hydromulch, to prevent the wind from blowing them away.

Grasses can be permanent or temporary. When used as permanent control measures, maintenance must be considered. Maintenance equipment (mowers, tractors) is difficult to operate on slopes steeper than 4H:1V. Although steep slopes optimize land use they also make maintenance difficult. In general, the steeper the slopes, the less frequent maintenance operations should be expected. Installing a 2H:1V slope instead of a 5H:1V slope may result in more usable land. (The steeper slope requires less distance to tie-out the proposed grade to the existing grade, thereby permitting a larger portion of the site to be a flatter grade for use as buildable or usable land). In areas of high land values, this may generate a substantial sum of money for the developer. On long, steep slopes other forms of vegetative practice should be considered.

Grass-seed mixtures are regionally developed to suit climate and soil conditions. Some species of grass are not

recommended for steep slopes or other special conditions. Local design manuals, soil conservation district offices, and the extension service of a local university provide planting information. In areas where vegetation is not practical, mulches can be used as a substitute.

**Soil Binders.** Soil binders are chemicals and synthetic materials sprayed onto the soil surface in order to bind the soil particles together or to protect seeding and sprigs from washing away. These are emulsions or dispersions of vinyl compounds, rubber or other substances that are mixed with water and applied to the soil. Soil binders are most effective when sprayed over an organic mulch to keep it from blowing away. Alone, soil binders do not have the capability to insulate or retain moisture and are easily damaged by traffic. The soil binders decay within 60 to 90 days of application. Table 29.1 lists selected mulches and the recommended rates of application.

**Soil Bioengineering.** Historically, methods of controlling streamflow and wave-induced erosion have included structural practices like riprap, sheet piles, and retaining walls. These traditional approaches are often not effective in environmentally sensitive areas when the objective is to limit disturbance and preserve the aesthetic quality of the natural

**TABLE 29.1 Organic Mulch Materials and Application Rates**

MULCHES	RATES		NOTES
	PER ACRE	PER 1000 SQ FT	
Straw or hay	1½–2 tons (Minimum 2 tons for winter cover)	70–90 lb	Free from weeds and coarse matter. Must be anchored. Spread with mulch blower or by hand.
Fiber mulch	Minimum 1500 lb	35 lb	Do not use as mulch for winter cover or during hot, dry periods.* Apply as slurry.
Corn stalks	4–6 tons	185–275 lbs	Cut or shredded in 4–6 in. lengths. Air-dried. Do not use in fine turf areas. Apply with mulch blower or by hand.
Wood chips	4–6 tons	185–275 lbs	Free of coarse matter. Air-dried. Treat with 12 lb nitrogen per ton. Do not use in fine turf areas. Apply with mulch blower, chip handler, or by hand.
Bark chips or shredded bark	50–70 cu yds	1–2 yd³	Free of coarse matter. Air-dried. Do not use in fine turf areas. Apply with mulch blower, chip handler, or by hand.

\* When fiber mulch is the only available mulch during periods when straw should be used, apply at a minimum arate of 2000 lbs/ac or 45 lbs/1000 sq ft



setting. Soil bioengineering techniques use combinations of live vegetation, with and without common structural practices, to produce erosion protection. The advantages of these bioengineering solutions include: the environmental benefits of wildlife habitat, water-quality improvement, and aesthetics; improved strength and stability over time as vegetation becomes established; reduced construction-access requirements; and limited maintenance costs. Soil bioengineering is a diverse, multi-disciplinary, and rapidly growing field requiring the knowledge of engineers, botanists, horticulturalists, hydrologists, soil scientists, and construction contractors. Soil bioengineering yields the following advantages: temporary and permanent vegetation to reduce erosion; off-site sedimentation; runoff; and velocities; increased consumption of internal soil moisture; and increased infiltration.

Examples of soil bioengineering techniques (Figure 29.3) include contour wattling, brush layering, brush matting, and coir fascines.

- *Contour wattling* is used to break long slopes into shorter slopes by planting bundles of branch cuttings (called wattles or fascines) in shallow trenches at regular contour intervals along the slope. After staking the wattles in place, only the top is exposed. When installed properly and cut from native plants, they will root and immediately begin to stabilize the slope.
- *Brush layering* is a method of restoring slopes by constructing a fill slope of alternating layers of live branches and soil, creating a series of reinforced benches.
- *Brush matting* is used to protect streambanks from erosion by placing a mattress-like layer of branches over the bank to protect its soil and slow-water velocity. The usually dead branches are secured with stakes or live branches to collect sediment and enable the establishment of vegetation on the banks.
- *Coir fascines* are wattles made from coconut fiber and are used to restore streambanks and wetlands where a natural look is desired. The coir accumulates sediment and biodegrades as plant roots develop, thus becoming a stabilizing system.

Riprap is often used to protect streambanks and lake-shores. Live cuttings can be interplanted in riprap to provide additional slope stability. Root growth below the riprap will improve the soil strength and hide the rocks, presenting a more natural look. Similarly, the use of live, woody cuttings in combination with porous, inert structural materials help create live crib walls, vegetative rock gabions, and joint plantings that stabilize slopes and improve erosion control.

Soil bioengineering measures are not appropriate for all slope failure and slope erosion problems. In some cases, grass seeding and hydromulching may be satisfactory and less costly. In other cases, it may be more effective to use

structural retaining systems alone. There are several limitations to bioengineering, as follows: the fact that the installation season is often limited to the plant-dormant season, when site access may be limited; the availability of locally adopted plants may be limited; the availability of adequate sunlight because of existing tree canopy may be limited; experienced labor may not be available; installers may not be familiar with bioengineering principles and designs and may require training; alternative practices may be more acceptable to property owners and contractors.

### Structural-Control Measures

There are practical and economical limits to the effectiveness of the erosion-control measures incorporated into a site. Usually, soil-stabilization and site-management-erosion controls supplement structural controls. Structural controls intercept runoff before it flows over denuded areas or divert it to a control area after it flows over the denuded area. Other types of structural devices obstruct the runoff to reduce the velocity or detain it to allow sediments to settle out.

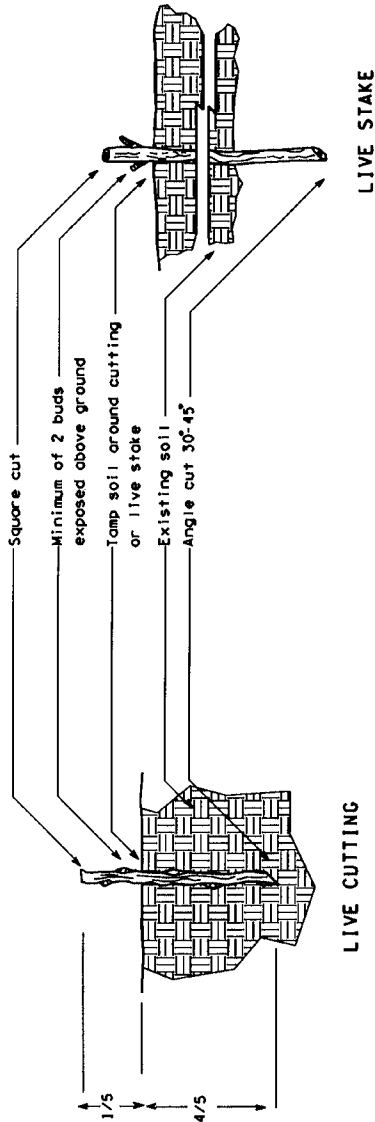
**Diversion Berms.** Compacted earthen berms (Figure 29.4), stabilized with vegetation or mulch, divert or intercept runoff. These berms are placed on natural grade around the perimeter of the denuded area. On the high side of the denuded area (up-slope), the berms (sometimes referred to as perimeter dikes) divert runoff around the denuded area to limit the amount of runoff flowing over it (see Figure 29.5). Runoff from rain, falling directly on the denuded area, is heavily laden with sediment. Berms located along the perimeter on the low side of the denuded area direct the sediment-laden runoff to a sediment trapping area, which filters out the sediments or allows the eroded sediments to settle out before runoff is released into natural watercourses.

All berms are compacted to increase their resistance to erosion and for stability purposes. The compaction typically is done by driving the earth-moving equipment along the sides and ridge of the berm. Depending on the soil used for the berm and the height of the berm, the compaction operation may have to be performed in approximately 1-ft lifts. Berms intended to function for extended periods (>30 days) should also be stabilized with vegetation to ensure their durability.

Berms are intended to collect and redirect sheet flows. They do not have the structural strength to withstand the momentum of concentrated flow. Accordingly, they should not be installed across swales or ditches that convey appreciable amounts of concentrated flows. The longitudinal grade (i.e., parallel to the berm) on the side of the flow should be sufficient to ensure positive drainage to the outlet. Minimum grades of 1% suffice in most cases. Construction of grades less than 1% in bare soil is difficult and may not be cost-effective. Steep, longitudinal slopes will result in excessive velocities and may erode the toe of the berm. Longitudinal slopes should keep velocities less than 3 fps.

**GENERAL NOTES:**

1. Live stake or cutting planted perpendicular to ground surface.
2. See plant material list for species length and spacing.
3. Live stakes - 50mm diameter minimum.
4. Install fascine parallel to contours unless specified differently.
5. Fascines constructed of a minimum 35% live, quick rooting material. Live material shall be placed on the bottom of the bundle.
6. Branches for fascine construction shall be 10 - 50mm in diameter.



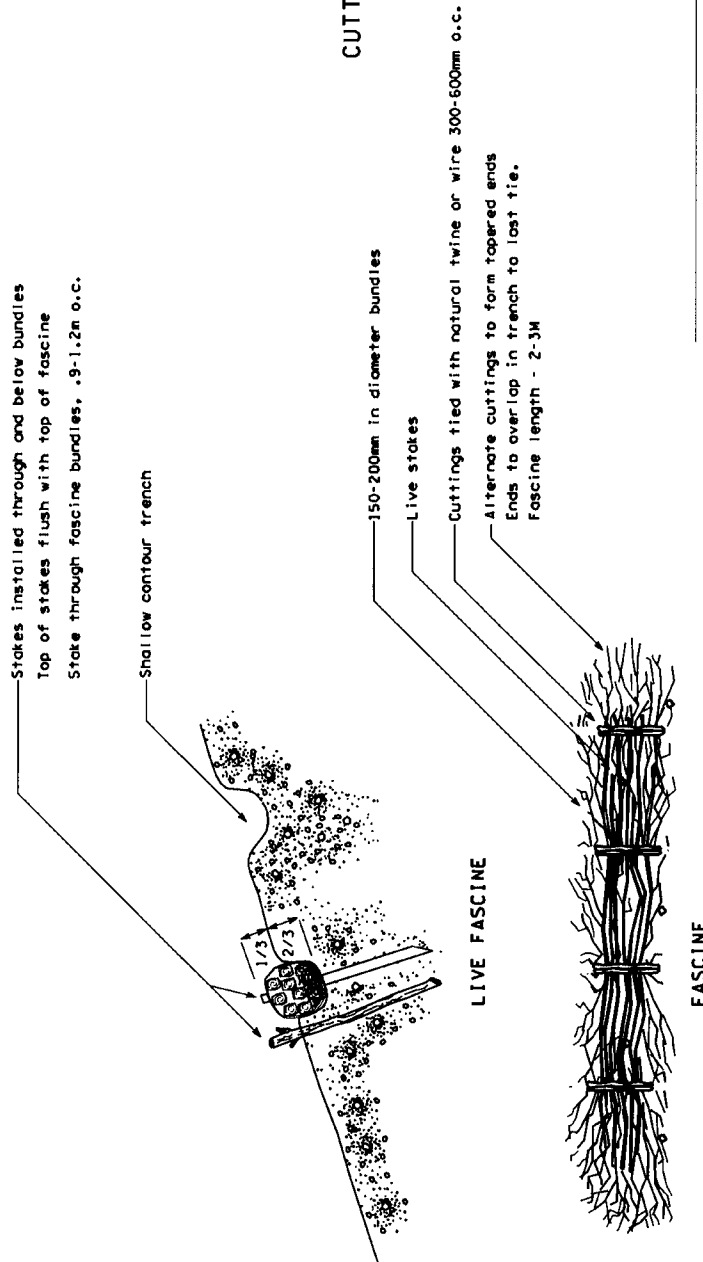
Stakes installed through and below bundles  
Top of stakes flush with top of fascine  
Stake through fascine bundles, .9-1.2m o.c.

Shallow contour trench

**CUTTING, LIVE STAKE AND FASCINE  
BIOENGINEERING DETAILS**

ALL DIMENSIONS ARE IN MILLIMETERS  
UNLESS OTHERWISE NOTED.

Not To Scale



DATE	REVISION	BY	APPR'D

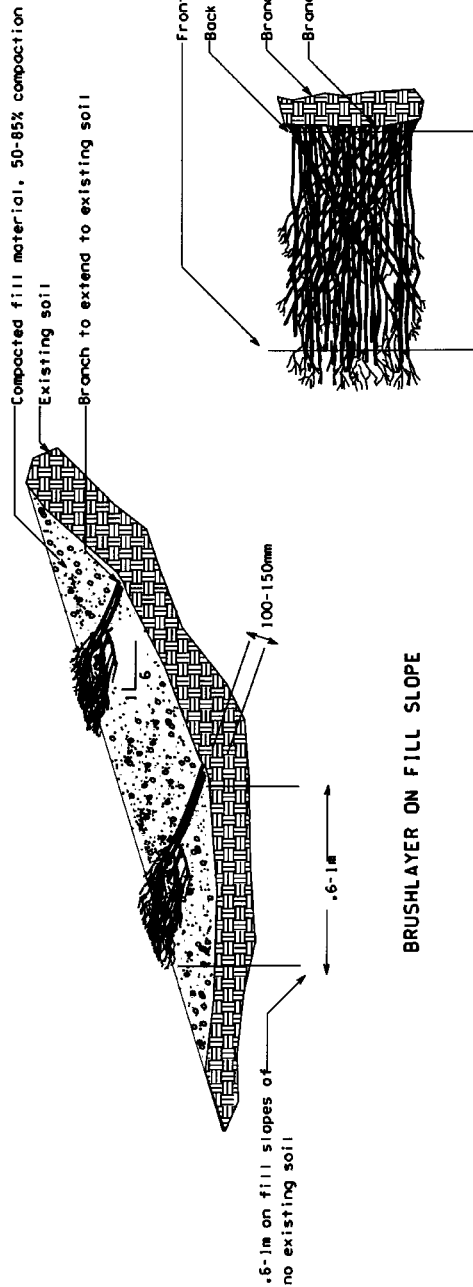
**STANDARD PLAN**  
APPROVED FOR PUBLICATION

STATE DESIGN LANDSCAPE ARCHITECT \_\_\_\_\_ DATE \_\_\_\_\_  
WASHINGTON STATE DEPARTMENT OF TRANSPORTATION  
OLYMPIA, WASHINGTON  
Sheet X of X Sheets

**FIGURE 29.3** Soil-bioengineering techniques.

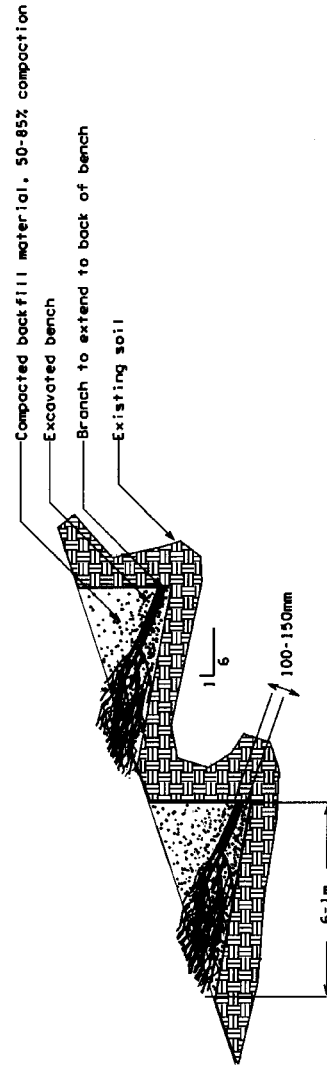
**GENERAL NOTES:**

1. Place cuttings perpendicular to slope.
2. See plans for spacing between rows.
3. Branches/cuttings to protrude from slope approximately 1/4 of its length max.
4. See Plant Material List for species and length.



**BRUSHLAYER ON FILL SLOPE**

**BRUSHLAYER ON FILL SLOPE  
PLAN VIEW**



**BRUSHLAYER ON CUT SLOPE**

ALL DIMENSIONS ARE IN MILLIMETERS  
UNLESS OTHERWISE NOTED.

**BIOENGINEERING  
BRUSHLAYER DETAILS**  
Not To Scale

**STANDARD PLAN**

APPROVED FOR PUBLICATION

STATE DESIGN LANDSCAPE ARCHITECT DATE  
WASHINGTON STATE DEPARTMENT OF TRANSPORTATION  
OLYMPIA, WASHINGTON  
Sheet X of X Sheets

**FIGURE 29.3** (Continued) Soil-bioengineering techniques.

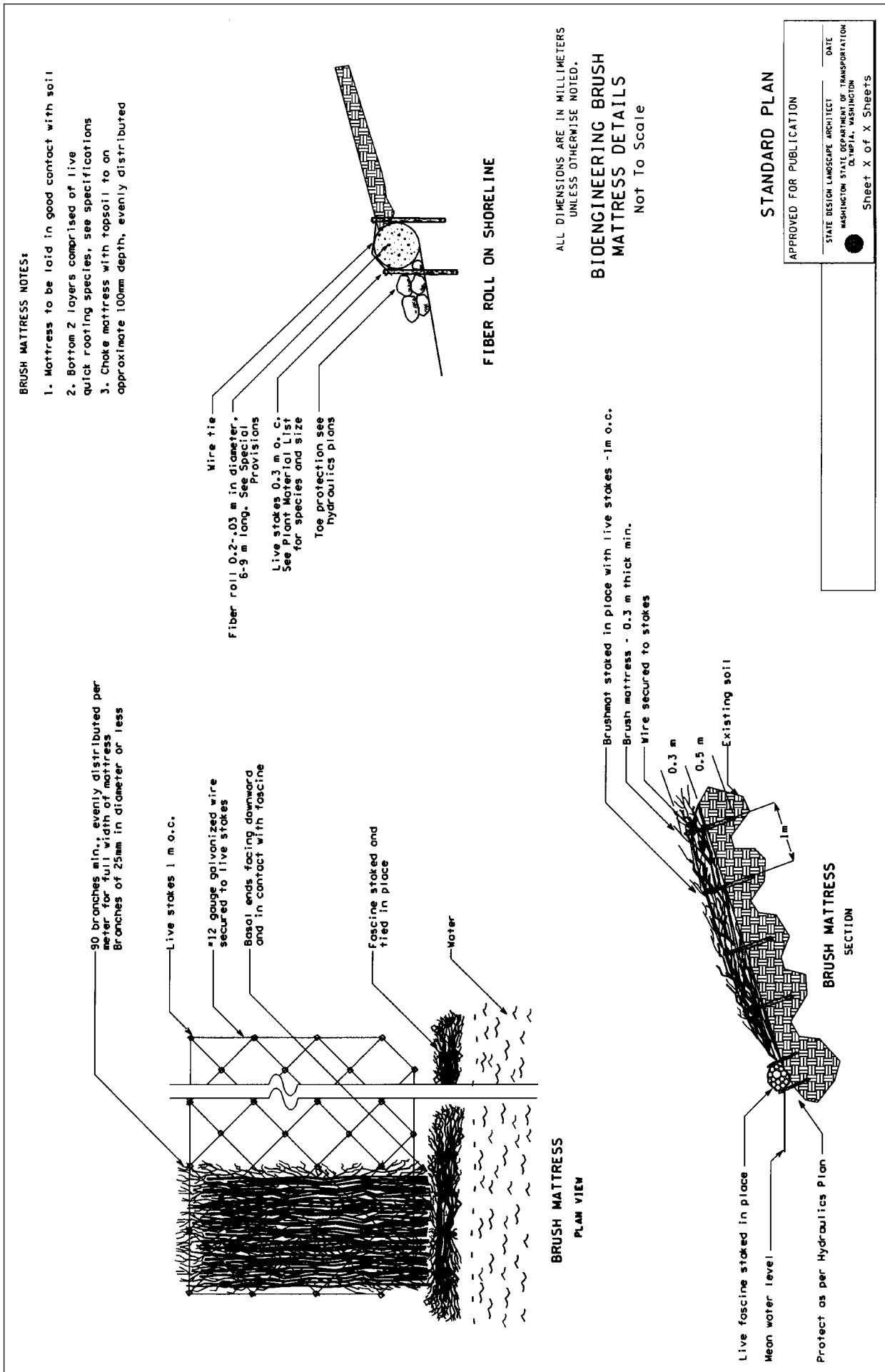
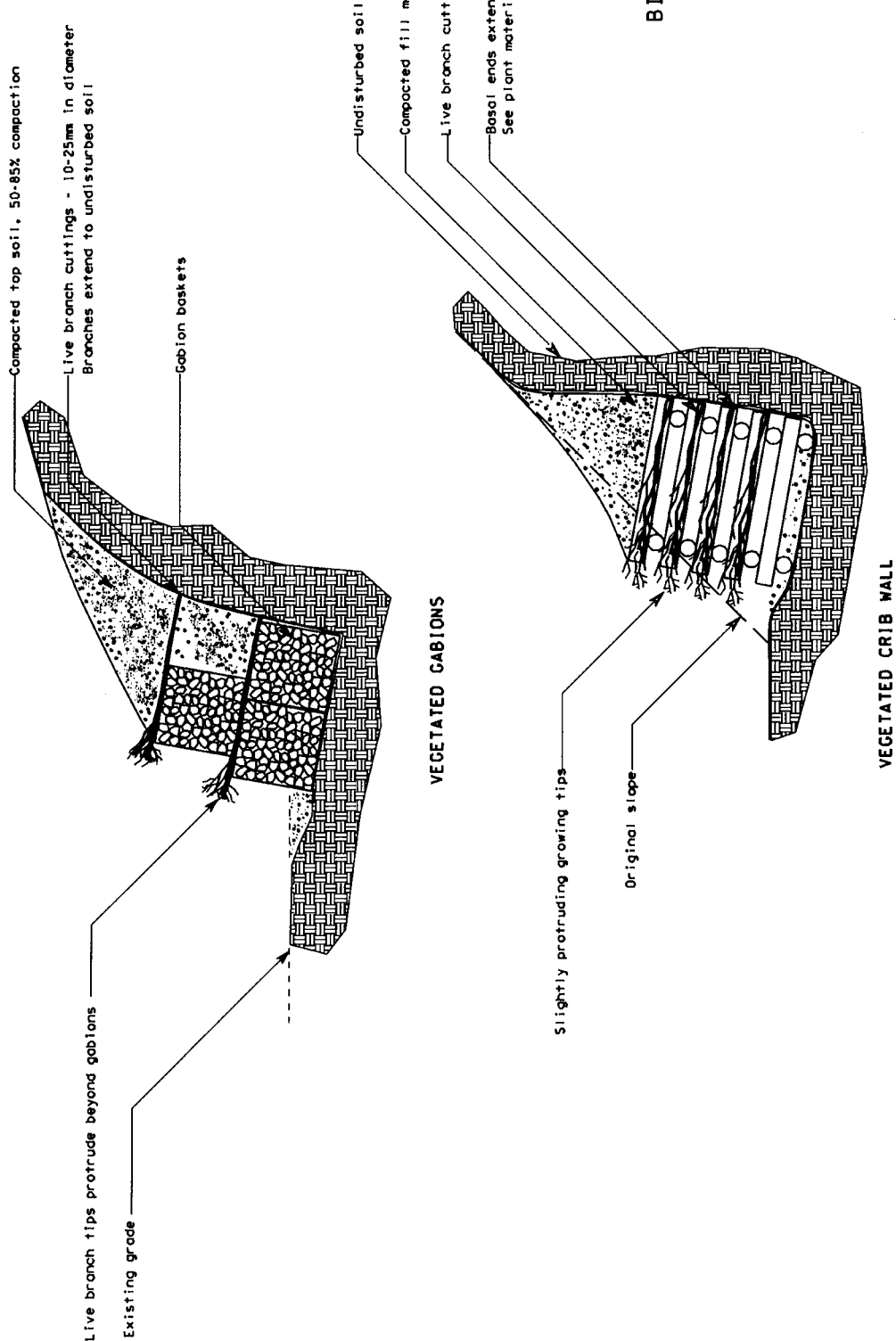


FIGURE 29.3 (Continued) Soil-bioengineering techniques.

GENERAL NOTES:

1. See Plans and Specifications for wall design and placement.

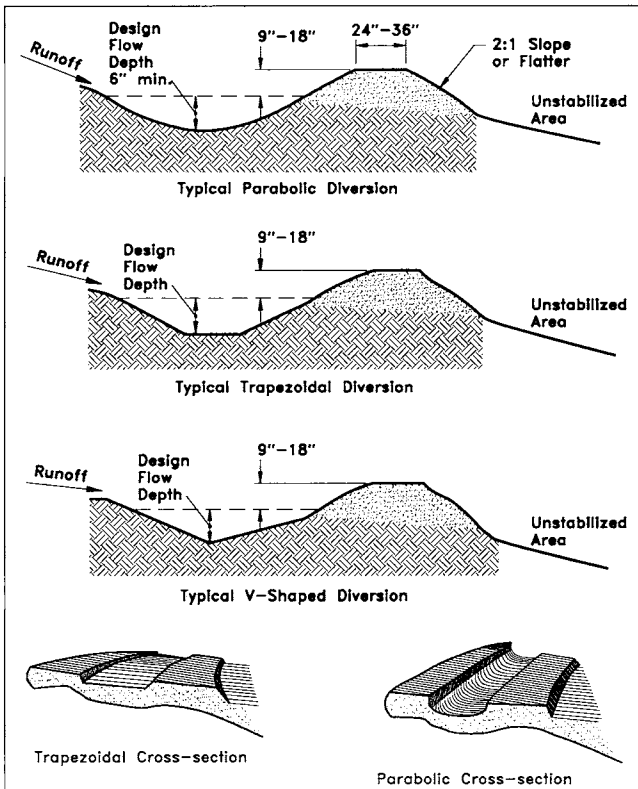


VEGETATED WALL  
BIOENGINEERING DETAILS  
Not To Scale

STANDARD PLAN C-2C

APPROVED FOR PUBLICATION	
STATE DESIGN LANDSCAPE ARCHITECT	DATE
WASHINGTON STATE DEPARTMENT OF TRANSPORTATION	OLYMPIA, WASHINGTON
Sheet X of X Sheets	

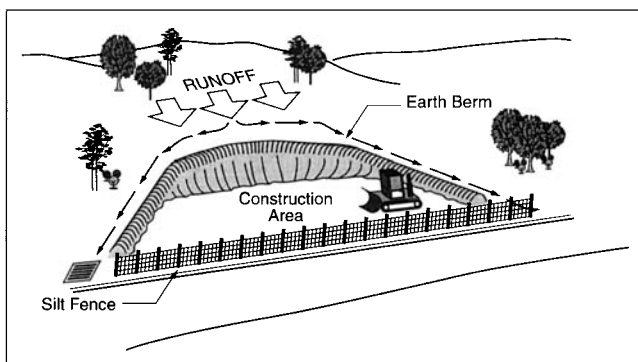
FIGURE 29.3 (Continued) Soil-bioengineering techniques.



**FIGURE 29.4** Typical diversion berms. Adapted from ULI, ASCE, and NAHB; *Residential Erosion and Sediment Control*, 1978. (Reproduced by permission of ASCE.)

If velocities approach 5 fps, special precautionary construction measures (e.g., special soils or armoring) should be used to reinforce the berm.

**Channel Conveyance.** A shallow ditch ensures positive drainage for small quantities of concentrated flow. As the flows increase, a more refined ditch design is necessary. A ditch of prismatic cross section (e.g., trapezoidal, parabolic, rectangular) where the design velocity coincides with the type of lining in the channel is required. Channels excavated in earth are most effective with a trapezoidal cross



**FIGURE 29.5** Diverting runoff around the construction site. (From Goldman S, et al: *Erosion and Sediment Control Handbook*, 1986. Reproduced with permission of McGraw-Hill.)

section and the side slopes less than the angle of repose of the saturated bank soil. Angles of repose vary from less than 15° for soft clays to over 40° for large, angular non-cohesive gravels. For normal design practice, maximum side slopes of 2H:1V function adequately over the range of these angles of repose.

Tables 29.2 and 29.3 identify various linings and the recommended velocities for grass and earth-lined channels. Use Manning's equation to check the velocities in the channel and compare to the velocities given in Table 29.2 or Table 29.3. When selecting the lining, consider whether the channel will be temporary or permanent. Installation costs should be kept to a minimum for temporary channels.

**Slope Drains.** If the flow behind a diversion berm generates highly erosive velocities, another method for conveying the water over the slope should be used. As shown in Figure 29.6, temporary slope drains or other types of flumes can convey the flow over the face of a slope without causing erosive damage.

Temporary slope drains consist of plastic pipe (flexible or rigid) or geotextile fabric. The drains are installed on the face of the slope and anchored securely. At the entrance of the slope drain, a berm diverts the up-slope-runoff into the slope drain and keeps the flow from sheeting over the exposed face of the unstable slope. The force of the water

**TABLE 29.2 Permissible Velocities for Earth-Lined Channels**

SOIL TYPES	PERMISSIBLE VELOCITIES
Fine sand (noncolloidal)	2.5 fps
Sandy loam (noncolloidal)	2.5 fps
Silt loam (noncolloidal)	3.0 fps
Ordinary firm loam	3.5 fps
Fine gravel	5.0 fps
Stiff clay (very colloidal)	5.0 fps
Graded, loam to cobbles (noncolloidal)	5.0 fps
Graded, silt to cobbles (colloidal)	5.5 fps
Alluvial silts (noncolloidal)	5.0 fps
Alluvial silts (colloidal)	5.0 fps
Coarse gravel (noncolloidal)	6.0 fps
Cobbles and shingles	5.5 fps
Shales and hard pans	6.0 fps

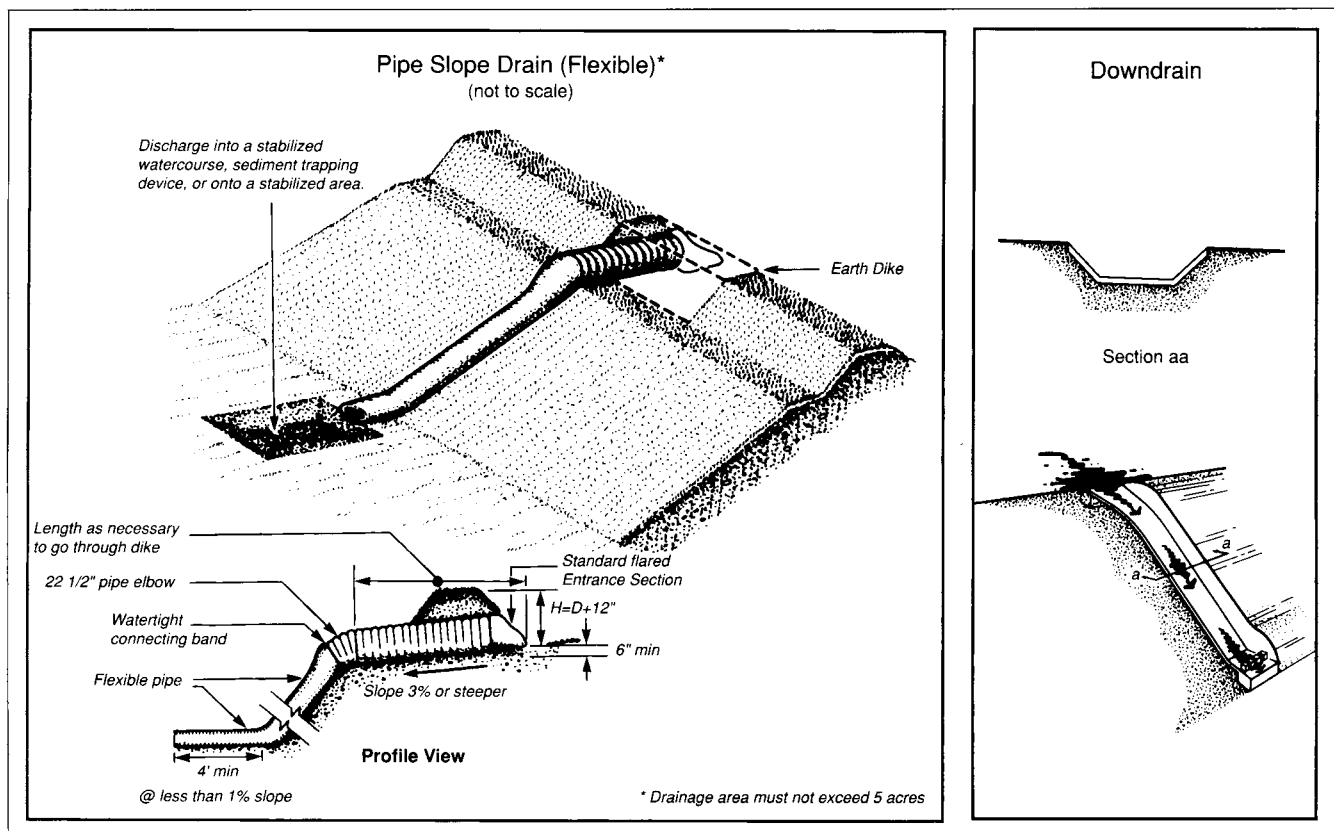
(Source: *Virginia Erosion and Sediment Control Handbook*, 1992.)

**TABLE 29.3 Permissible Velocities for Grass-Lined Channels**

CHANNEL SLOPE	LINING	PERMISSIBLE VELOCITY
0%–5%	Bermuda grass	6 fps
	Reed canarygrass, tall fescue	
	Kentucky bluegrass	5 fps
	Grass-legume mix	
	Red fescue, redtop, sericea	2.5 fps
	lespedeza, annual lespedeza,	
small grains (temporary)		
5%–10%	Bermuda grass	5 fps
	Reed canarygrass, tall fescue,	4 fps
	Kentucky bluegrass	
	Grass-legume mix	3 fps
>10%	Bermuda grass	4 fps
	Reed kanarygrass, tall fescue,	3 fps
	Kentucky bluegrass	

(For highly erodible soils, decrease the above velocities by 25%)

(Source: Virginia Erosion and Sediment Control Handbook, 1992.)



**FIGURE 29.6** Slope drains. (Source: USDA, Soil Conservation Service.)

flowing through the slope drain, along with the vertical component of gravity acting on the drain itself, puts tension at the drain's section joints. If the joints are not connected and anchored securely, they separate. Extensive erosive damage to the slope occurs when the slope drain fails in this manner, resulting in costly repairs.

**Outlet Protection.** Outlet protection is used at the discharge end of a slope drain, paved flume, and at outlets of pipes and channels. It consists of a structurally-lined apron or other type of device, which reduces the erosion potential downstream by dissipating the flow energy and allowing flow to transition back to the natural channel. In addition, it prevents scouring at the base of the slope and underneath the discharge end of the outlet pipe.

Depending on the flow energy and available materials, the apron may consist of riprap, grouted riprap, gabion baskets, concrete, or another type of structurally-sound material adequate for the hydraulic conditions. Paved channel outlets and vegetal linings are other alternatives for apron material. Figure 29.7 shows two methods for pipe outlet protection depending on whether the structure discharges into a well-defined channel or a flat area. Figures 29.8 and 29.9 provide the recommended apron length and stone size for a given pipe diameter flowing full. Figure 29.8 is used to size riprap outlet protection stone when the prevailing tailwater depth at the discharge end is less than one-half the diameter of the pipe. Figure 29.9 is used when the tailwater depth is greater than or equal to one-half the pipe diameter.

**EXAMPLE 1:** Given a pipe diameter ( $d$ ) of 24 in., full-flow conditions with  $Q = 50$  cfs and  $TW < 0.5*d$ , find length and width of outlet protection apron. From Figure 29.8 for  $Q = 50$  cfs and  $d = 24$ , read  $d_{50} = 1$  ft. On the upper pipe diameter scale, read  $La = 28$  ft for  $d = 24$ . Then  $W = 2 + 28 = 30$  ft.

The length of protection can be judged based on the magnitude of the exit velocity compared with the natural-channel velocity. The greater the difference, the longer the length of riprap-outlet protection. When outlet protection lengths are not economical, or the length of placement is limited because of property constraints, other methods of dissipating the energy should be considered. *The Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14*, published by the U.S. Department of Transportation, Federal Highway Administration, provides design information for various type of energy dissipators.

### Check Dams

Check dams are used in conveyance channels where permanent linings, such as sod, riprap, asphalt, or concrete, are not cost-effective as a temporary measure to reduce erosion potential while vegetation is germinating. Check dams are also used in temporary channels and ditches where the

length of service precludes the use of other linings. Check dams are typically stone structures placed across the flow path in channels and ditches—not live streams. Check dams act as energy dissipators, creating more resistance in the watercourse, thereby reducing the water's erosivity. Although some sediment removal is expected through filtering, the use of check dams solely as filtering devices is not recommended. Their use is limited to channels that convey runoff from 10 acres or less.

Check dams can be constructed of stone aggregate  $\frac{3}{4}$  to  $2\frac{1}{2}$  in. in diameter for smaller drainage areas, with 6 to 12-in. stone riprap on the downstream side of the check dam for larger drainage areas. The material is placed across the channel from top-of-bank to top-of-bank, with the center of the dam 6 in. lower than the top of bank. The upstream and downstream faces of the stone check dams have slope gradients of 2H:1V. Spacing of the check dams is such that the toe of the preceding upstream dam is at the same elevation as the top of the subsequent downstream dam. Figure 29.10 shows the typical layout of check dams.

During high flows, water can pond behind the check dam and flow around it over the banks of the channel. This causes erosion of the banks at each end of the check dam. After heavy rainstorms, the channel should be inspected for any erosive damage, either in the vicinity of the check dams or in the channel itself.

**Level Spreader.** The level spreader is a device that dissipates the energy of the concentrated flow by converting it into sheet flow. The level spreader is an excavated depression with a level grade across the face of a stabilized slope. Concentrated flow enters the level spreader, and when the capacity of the depression is exceeded, the flow uniformly spills over the lip and down the slope as sheet flow.

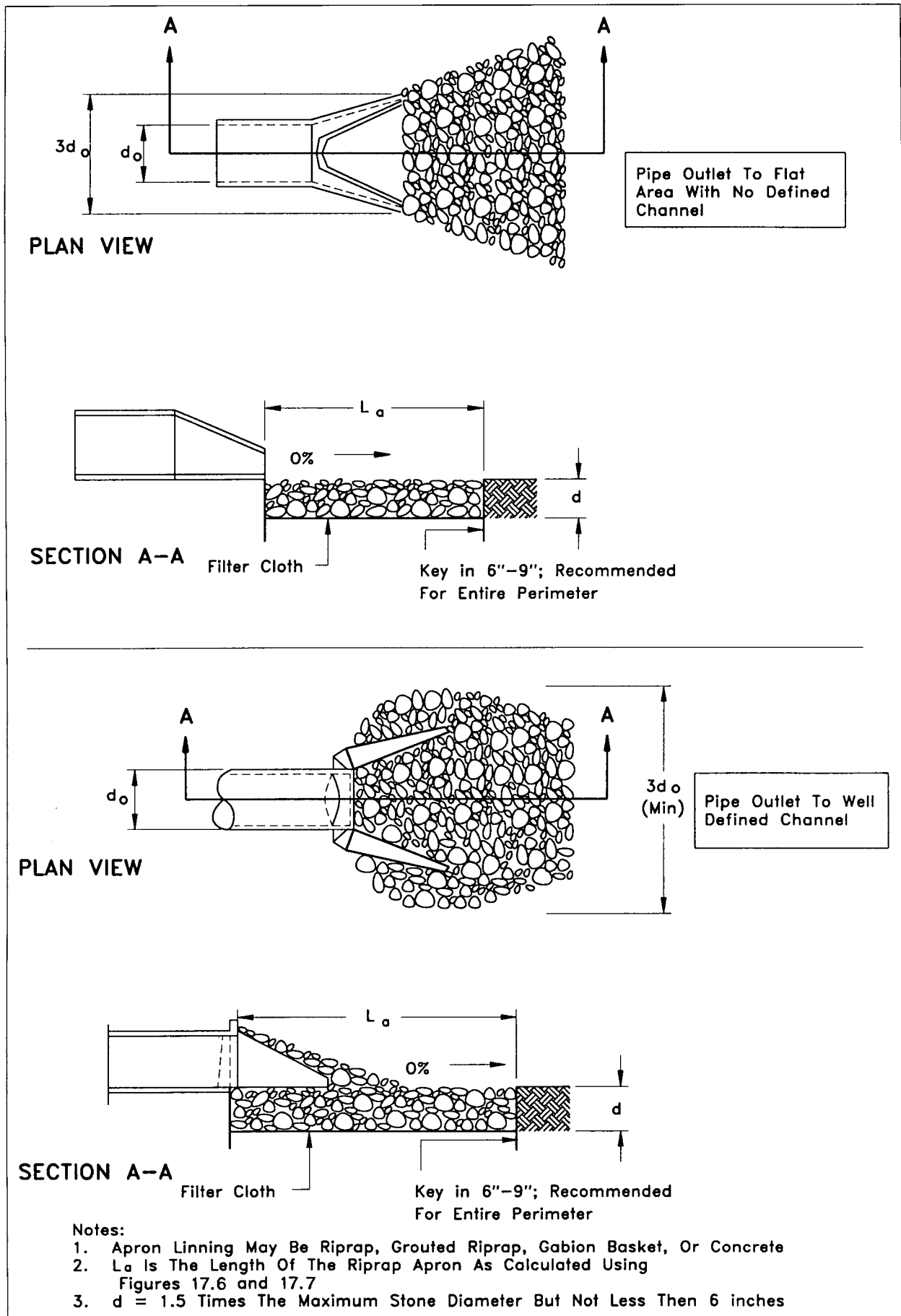
The effectiveness of the level spreader depends on strict adherence to construction specifications. The minimum depth of the level spreader is 6 in. and is uniform across its length. Limiting the maximum velocity of the water entering the level spreader to 3 fps prevents scouring within the spreader. The grade across the spreader is flat and should remain flat during its term of operation. The slope of the ground below the level spreader should be 2H:1V or flatter to keep the sheet flow velocities from becoming too erosive. Steep slopes below the level spreader promote erosion as the sheet flow accelerates down-slope. The spreader length will be determined by estimating the flow expected from the design storm and selecting the appropriate length from Figure 29.11.

Production of uniform sheet flow requires uniform discharge over the lip. Any trash, debris, or sediment accumulation within the level spreader will disturb the production of uniform flow. Frequent inspection and clean-out is necessary for the level spreader to operate effectively (see Table 29.4).

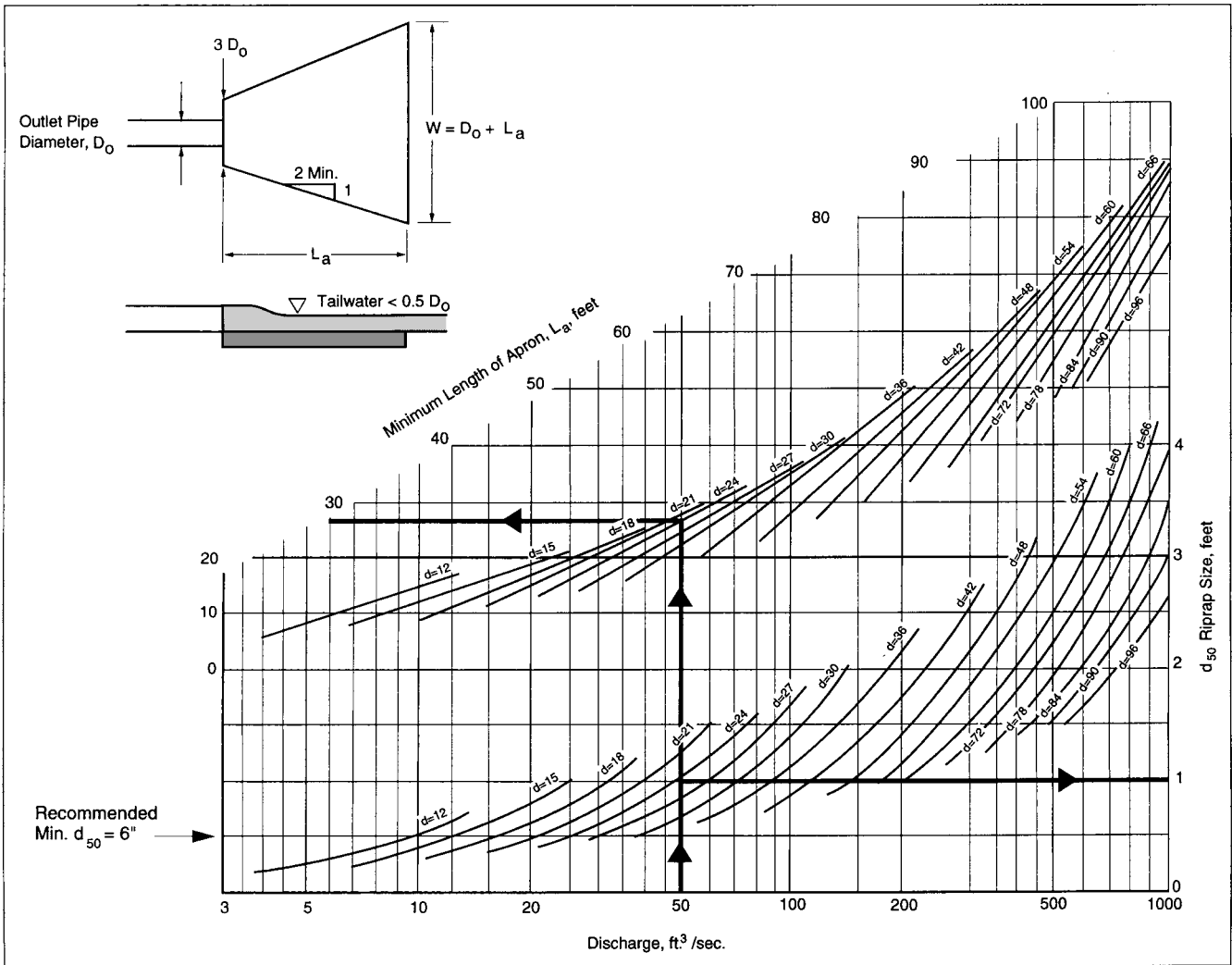
### Coastal-Erosion-Control Measures

Losses due to erosion are anticipated to grow over the next few decades as coastal areas continue to erode into areas of





**FIGURE 29.7** Pipe outlet conditions. (Source: Virginia Erosion and Sediment Control Handbook, 1992.)



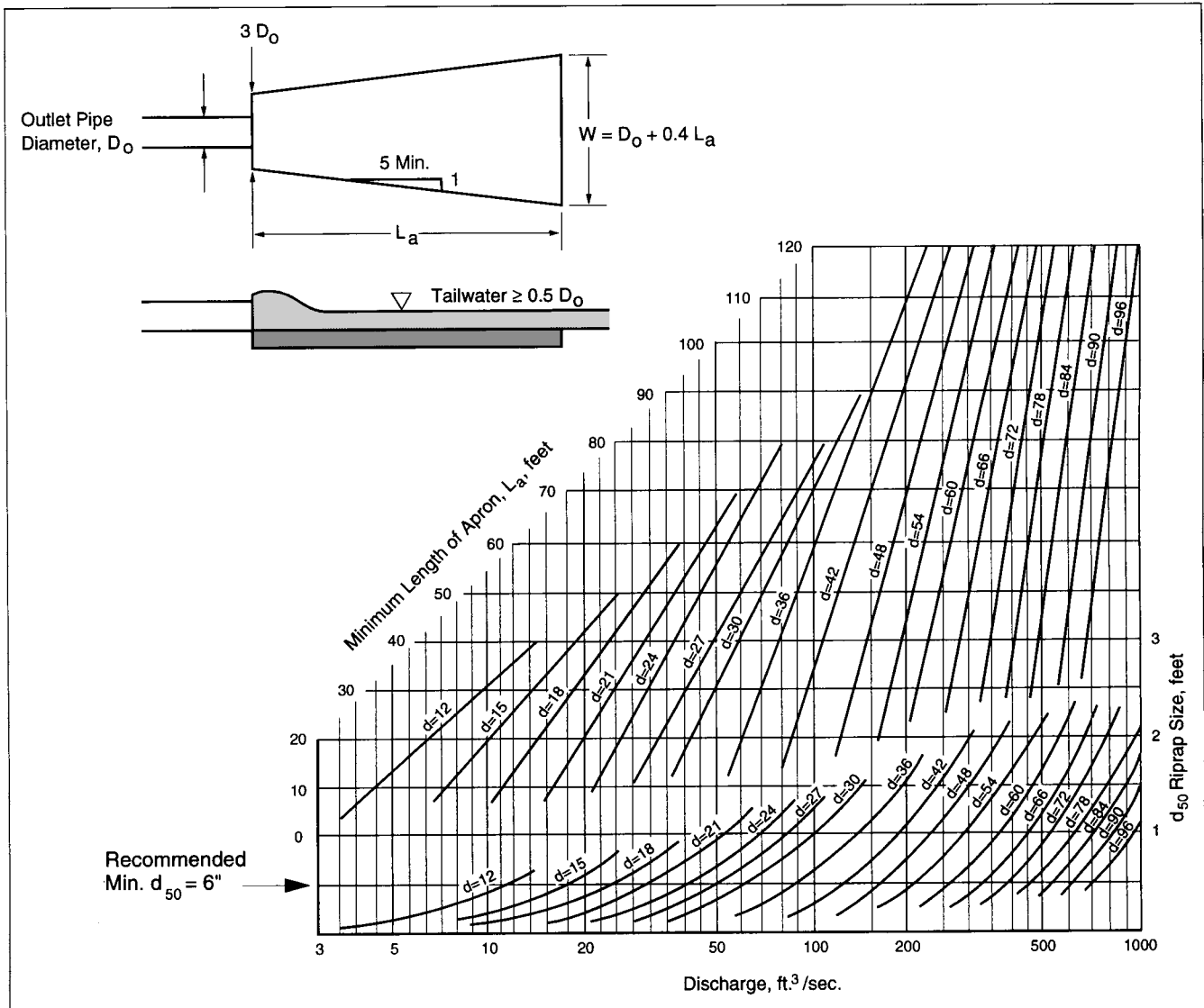
**FIGURE 29.8** Design of outlet protection for round pipe flowing full ( $T_w < 0.5 \cdot d$ ). (Source: *Virginia Erosion and Sediment Control Handbook*, 1992.)

denser development. Site management and planning development to avoid sensitive coastal land are the most effective coastal-erosion-control measures. Many states regulate land use along the coast through setbacks, control zones, or a combination of the two. Setback requirements limit building within a specified distance of the shoreline. Control zones regulate a range of activities, including construction or repair of shoreline stabilization structures, public access, and the use or alteration of a dune and other shore-line habitat areas. Acquisition of privately owned lands by the government or non-profit groups is another means by which to control development in erosion-prone areas. Acquisition can also be used to achieve important community goals like environmental protection and public-beach access. However, acquisition has not been widely used due to the high costs of coastal property.

In an attempt to protect property from erosion, a variety of structural and non-structural measures has been utilized. Many of these measures provide only temporary protection

from erosion and coastal flooding and require ongoing maintenance or repair. Methods of protecting beaches and shorelines from erosion have depended upon efforts to overcome the natural forces at work near the shoreline. Structures such as seawalls, bulkheads, breakwaters, revetments, and groins have been created as barriers against the natural effects of waves, tides, and currents. These structures inhibit the natural-functioning beach/dune system. In doing so, they interfere with the on-shore—off-shore movement of sand and cause increased erosion on adjacent shores by modifying sediment supply or changing wave-refraction patterns. Today, traditional structural measures are generally limited to the protection of existing non-conforming structures and include the following.

**Seawalls and Bulkheads.** Seawalls and bulkheads have been used traditionally to protect development from storm surges and further recession of an eroding shoreline. Seawalls are used primarily to resist wave action. Bulkheads act as retaining walls, keeping the earth behind them stable.



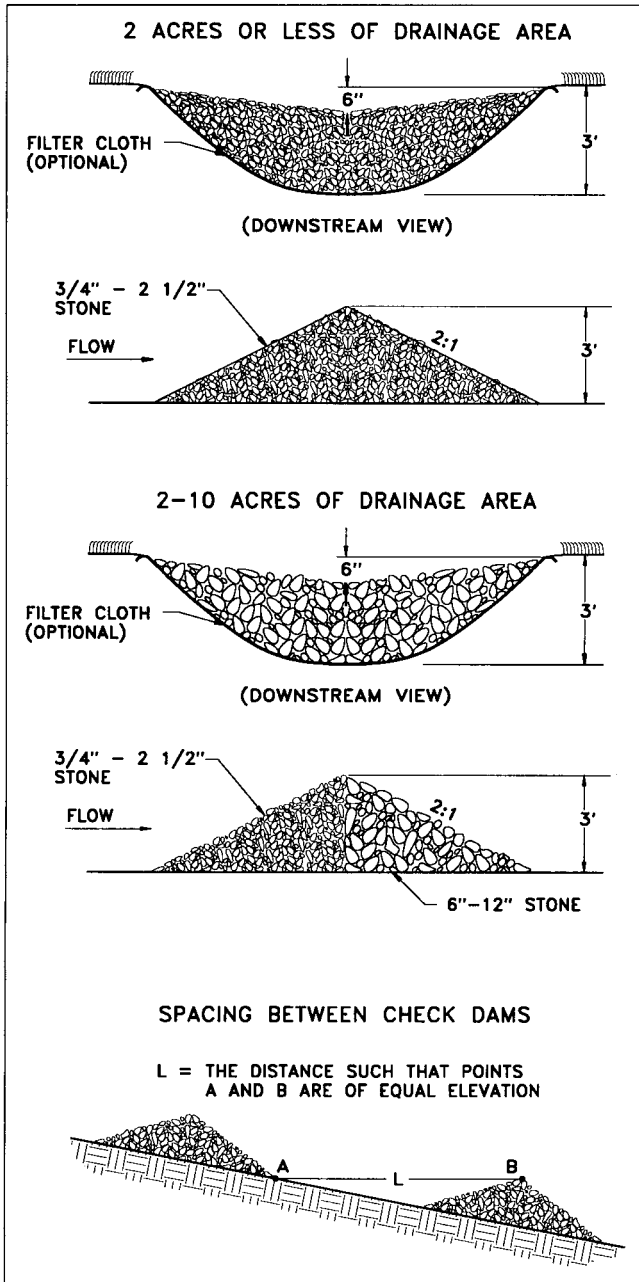
**FIGURE 29.9** Design of outlet protection for round pipe flowing full ( $T_w \geq 0.5d$ ). (Source: Virginia Erosion and Sediment Control Handbook, 1992.)

While these can prevent the recession of a shoreline, they cannot prevent the erosion of beach areas. Hardening of a shoreline can interfere with the necessary profile adjustments of a beach. A dune can no longer share its sand with a beach. As a retreating beach encounters a seawall or bulkhead it can no longer draw upon the landward sand supply and it begins to erode.

Seawalls and bulkheads can be built in three basic types: interlocking sheetpiles driven into the ground; individual piles used to support an above-ground structure; or a massive gravity-construction resting on the shore-bottom supported by its own weight. Bulkheads and seawalls must be of sufficient strength to withstand the forces of waves and

wave-carried debris, both in front of the wall and behind it. Water flowing around the wall can cause excessive erosion, so the flanks of the wall need to be protected.

**Breakwaters.** Breakwaters provide an off-shore impediment to the high-energy waves that are responsible for a large part of the erosion of beach areas. Breakwaters create a physical barrier to the beach and shore in order to disrupt the scouring of the waves. While breakwaters can reduce the erosive impact of waves, they can cause localized accretion in the lee of the breakwater interfering with long-shore currents. Longshore currents are valuable for clearing debris from beaches. Breakwaters, including a number of patented breakwater-systems, can be constructed from many different materials.



**FIGURE 29.10** Construction details for check dams. (Source: Virginia Erosion and Sediment Control Handbook, 1992.)

**Revetments.** Revetments are structures placed on the banks or bluffs in such a way as to absorb the energy of incoming waves. They are usually built to preserve the existing uses of the shoreline and to protect the slope. Where available, rubble or quarry stone makes the most reliable and economical revetments. Other alternatives include bags filled with a wet sand—cement mixture, concrete blocks or slabs, and gabions. Most revetments do not significantly interfere with littoral drift. They do not redirect wave energy to vulnerable unprotected areas, although beaches in front

of steep revetments are prone to erosion. In some locales, bank sloping in combination with soil-bioengineering techniques is favored over structural revetments.

**Groins.** Groins are used to manage long-shore transport of littoral drift to control erosion. Groins are structures that extend perpendicularly from the coastline and are constructed in groups called fields. They are most effective where the longshore transport is predominantly in one direction, or where their construction will not cause unacceptable erosion of the downdrift shore. Groyne fields must be designed carefully with respect to height, spacing, extension, and porosity. Groins must extend far enough in the water to retain adequate amounts of sand, but not so long that rip currents develop along them. Rip currents carry sand off-shore into deep water where waves cannot return it to the beach.

Non-structural measures to protect coastal property from erosion include vegetation and replenishment. Natural, salt-resistant vegetation provides good protection for the dune system, inhibits both wind and sand erosion and allows the system to adjust to natural stresses. Removal of existing salt-resistant vegetation from dunes usually is discouraged or prohibited. Adding fill to beach, either to replace the lost beach materials or to increase the size of an existing beach, can be economical and effective. The beach-replenishment material should resemble the original beach material and be placed to match the natural slope as closely as possible.

**THE UNIVERSAL SOIL LOSS EQUATION**

One of the most widely used and accepted equations for estimating soil erosion is the Universal Soil Loss Equation (USLE), an empirical equation developed by the U.S. Department of Agriculture. It was initially developed for agricultural purposes and since has been adapted to apply to construction sites. The USLE estimates the annual tonnage of soil eroded from the site attributable only to sheet and rill erosion. This easily converts into a volume, assuming a unit weight of soil.

Three common terms used in context with quantifying erosion are “soil erosion,” “soil loss,” and “sediment yield.” Soil erosion is the amount of soil moved by natural forces. Soil loss is the amount of soil moved off a particular area. Sediment yield is the soil loss delivered to a specific point. Sediment yield and soil loss are not necessarily equivalent. The amount of soil loss from a slope will be less than the sediment yield farther down slope because of deposition that occurs between the point in question and the base of the slope. Not all eroded soil qualifies as soil loss either. Eroded soil may be redeposited before it leaves the slope and, therefore, does not factor into soil-loss quantity. The USLE is

$$A = R \times K \times (LS) \times C \times P \tag{29.1}$$

where A is the average annual soil loss from sheet and rill

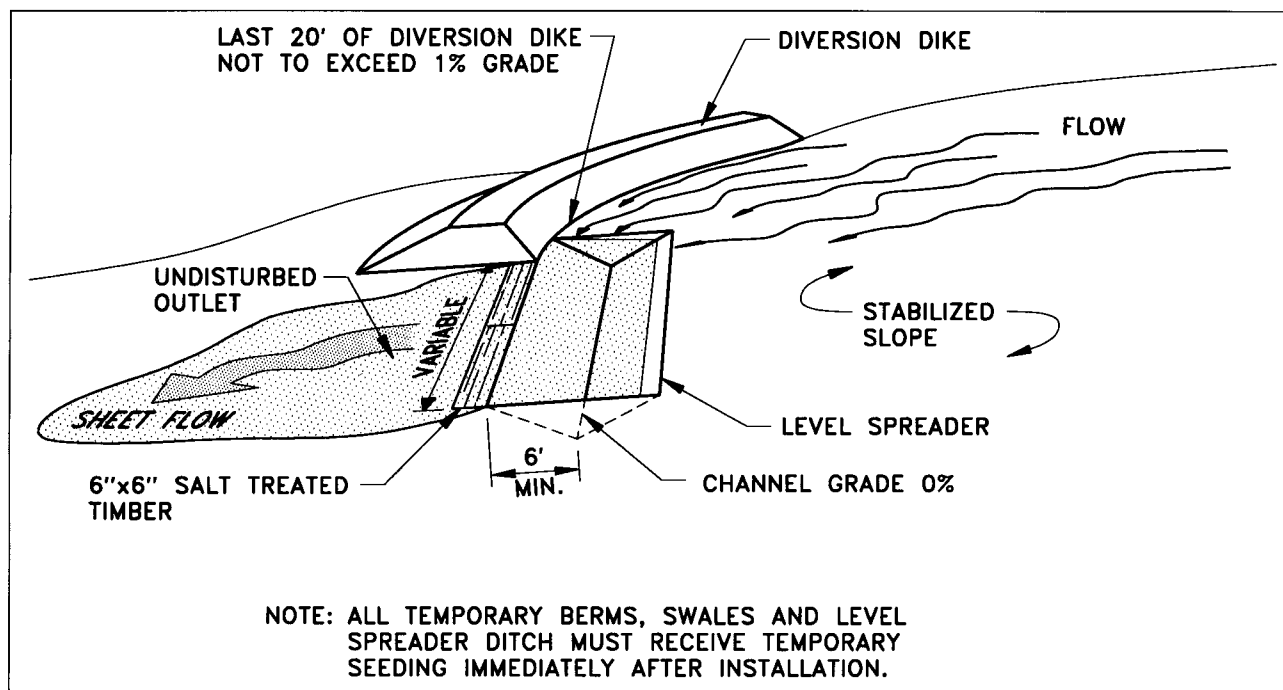


FIGURE 29.11 Level spreader. (Source: Virginia Erosion and Sediment Control Handbook, 1992.)

erosion measured in tons/acre. The remaining terms of the equation are discussed next.

**Rainfall-Erosion Index (R)**

The rainfall-erosion index is the product of the total raindrop energy, *E*, of the storm and the maximum 30-min intensity, *I*. *I*-values for a geographic location are obtained by summing the *I*-values for significant storms from a maximum 22-yr record to obtain an average annual index value. Figure 29.12 provides annual average values of *R*. Recognize that *R* is based on statistical and probabilistic analyses of rainfall records and, because of this, is not a precise factor for a given locality. Methods exist for computing more precise values of *R* from local storm data.

**Soil-Erodibility Factor (K)**

A soil's susceptibility to erosion depends on its physical and chemical characteristics. The *K* factor is an empirical value representing the erodibility per rainfall-erosion-unit. Named soils (i.e., soil series) for a region and their corresponding

*K* values can be obtained from local Soil Conservation Service offices. Table 29.5 lists approximate values for generalized soils. Generally, those soils with *K* < .23 are low-erodibility soils and soils with *K* > .41 are considered highly erodible.

**Length-Slope Factor (LS)**

The effects of slope-length and steepness are quite significant to soil erosion. The combined topographic effects of length and steepness are accounted for in the *LS* factor. The values for *LS* range from less than 1 for short, flat slopes to nearly 50 for long, steep slopes, as demonstrated by Eq. (29.2).

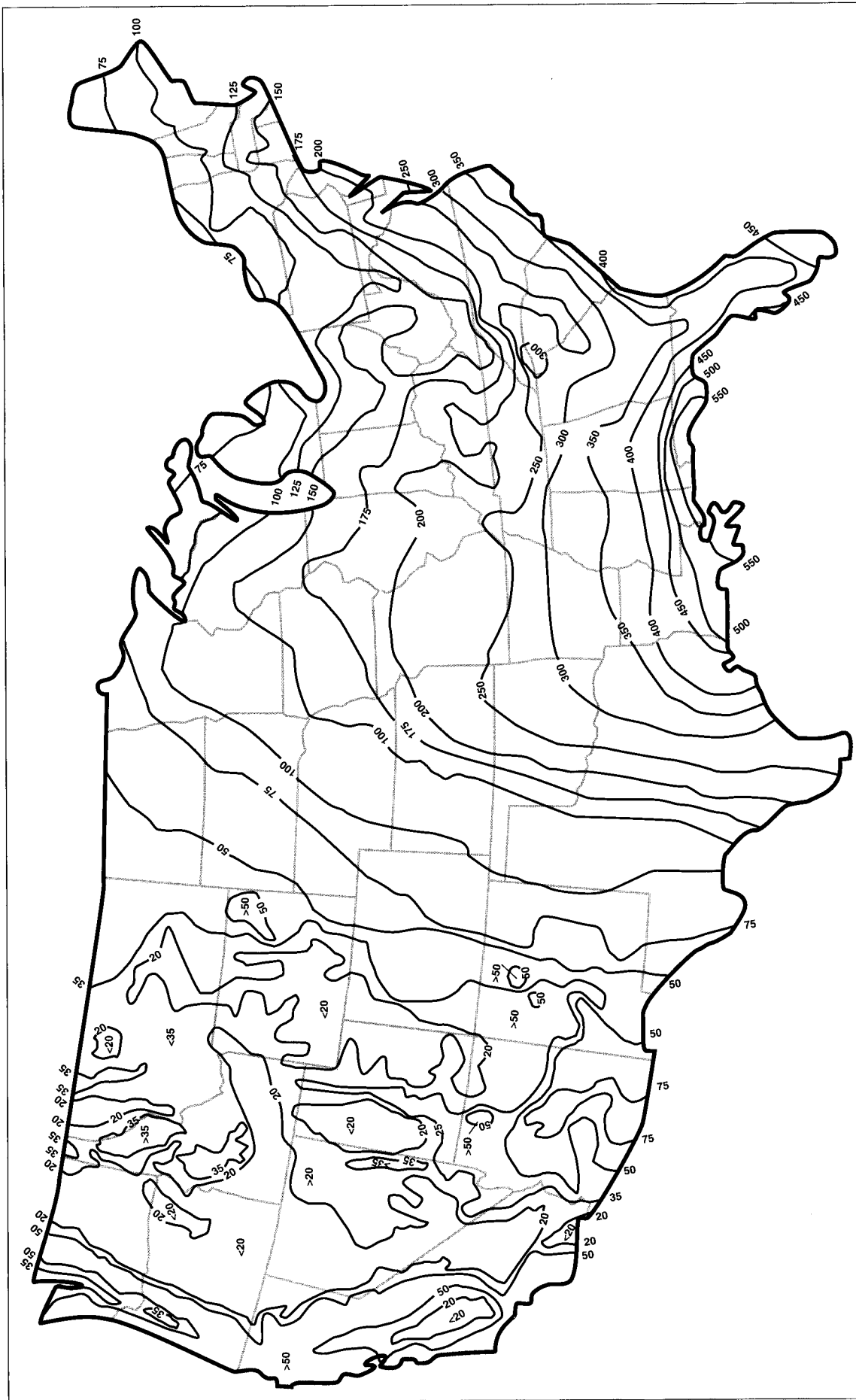
$$LS = \left( \frac{L}{72.6} \right)^m \left( \frac{430x^2 + 30x + 0.43}{6.574} \right) \quad (29.2)$$

where *L* = slope length in feet, *m* = 0.5 for slopes ≥ 5%, *m* = 0.4 for slopes between 3.5% and 4.5%, *m* = 0.3 for slopes ≤ 3%, *X* = sin θ where θ is the slope angle in degrees.

The slope-length *L* is the average distance, in feet, from the point of origin of overland flow to either the point where slope decreases to the extent that deposition begins, or the point where runoff enters well-defined channels, whichever occurs first. This empirical equation is based on data collected for slope-lengths between 10 and 400 ft and gradients between 3 and 20%. Any use of this equation beyond this data range is an extrapolation of the data and may need to be adjusted on the basis of judgment and experience.

**TABLE 29.4 Level Spreader Length for Design Flows**

DESIGN FLOW (cfs)	MINIMUM LENGTH (ft)
0-10	10
10-20	20



**FIGURE 29.12** Average annual values of the R factor. (Source: USDA, SCS 1977.)

**TABLE 29.5 K Values for Generalized Soils**

<b>K VALUE FOR TOPSOIL</b>	
<b>TEXTURE OF SURFACE LAYER</b>	<b>ESTIMATED K VALUE</b>
Clay, clay loam, loam, silty clay	.32
Fine sandy loam, loamy very fine sand, sandy loam	.24
Loamy fine sand, loamy sand	.17
Sand	.15
Silt loam, silty clay loam, very fine sand loam	.37
<b>K VALUES FOR SUBSOIL</b>	
<b>GENERALIZED SOIL CATEGORY (TEXTURE OR MATERIALS)</b>	<b>ESTIMATED K VALUE OF EXPOSED SUBSOIL MATERIAL</b>
A. Outwash soils	
Sand	.17
Loamy sand	.24
Sandy loam	.43
Gravel, fine to mod. fine subsoil	.24
Gravel, med. to mod. coarse subsoil	.49
B. Lacustrine soils	
Silt loam and very fine sandy loam	.37
Silty clay loam	.28
Clay and silty clay	.28
C. Glacial till	
Loam, fine to mod. fine subsoil	.32
Loam, med. subsoil	.37
Clay loam	.32
Clay and silty clay	.28
D. Loess	.37
E. Residual	
Sandstone	.49
Siltstone, nonchannery	.43
Siltstone, channery	.32
Acid clay shale	.28
Calcereous clay shale or limestone residuum	.24

Source: Soil Conservation Service, *Water Management and Sediment Control for Urbanizing Areas*, Columbus, Ohio, 1978, p. 79.

On slopes with non-uniform gradients, such as complex or convex slopes, the value for the *LS* factor is adjusted to account for non-uniform gradients. The adjustment is predicated on the following two assumptions:

1. A change in the gradient is not sufficient to cause up-slope deposition.

2. An irregular slope can be divided into several equal-length segments where each segment has a uniform gradient for its length.

The effective *LS* value,  $LS_e$ , is computed by dividing the irregular slope into equal-length segments, each with its own average gradient, which must be greater than 5%. Us-

ing the total length of the slope and the gradient for each segment, an  $(LS)'$  is found for each segment using Eq. (29.2). This value is multiplied by the weighing factor found in Table 29.6. The products are summed and divided by the number of segments to obtain the effective  $LS$  value. In equation form,

$$(LS)_e = \frac{\sum_{i=1}^n (LS)'_i f_i}{n} \tag{29.3}$$

where  $f_i$  is the weighing factor for the  $i$ -th segment and  $n$  is the number of equal-length segments.

**EXAMPLE 2:** Find the effective  $LS$  factor for a concave slope divided into four segments of 200 ft, with slopes (top to bottom) of 12, 10, 8 and 5%.

**SOLUTION:** Find the  $(LS)'$  value for each segment using  $L = 600$  ft and  $s_1 = 12\%$ ,  $s_2 = 10\%$ ,  $s_3 = 8\%$ , and  $s_4 = 5\%$  using Eq. (29.3). For example, a 12% slope is equivalent to  $6.8^\circ$  ( $\sin 6.8^\circ = 0.119$ ).

$$(LS)'_{12\%} = \left(\frac{600}{72.6}\right)^{0.5} \left(\frac{430(.119)^2 + 30(.119) + 0.43}{6.574}\right) = 4.4 \tag{29.4}$$

Similarly,  $(LS)_{10\%} = 3.5$ ,  $(LS)_{8\%} = 2.4$  and  $(LS)_{5\%} = 1.4$ . From Table 29.6, the weighing factors are 0.50, 0.91, 1.18, and 1.40 and the effective  $LS$  is

$$(LS)_e = \frac{4.4(.50) + 3.5(.91) + 2.4(1.18) + 1.4(1.40)}{4} = 2.5 \tag{29.5}$$

**Cover Factor (C)**

The cover factor is the vegetative cover or the cropping-management factor. It is an index of the type of ground

cover and the condition of the soil over the area. Specifically, it is a ratio of the soil loss from a specific cover condition to the soil loss from a clean-tilled fallow condition for the same soil, slope, and rainfall conditions. For denuded construction sites, a  $C$ -factor of 1 is appropriate. This condition is similar to the agricultural definition of continuous fallow, tilled up and down-slope, where  $C = 1$ . Table 29.7 shows typical  $C$ -values for undisturbed land. Table 29.8 shows  $C$ -values for various types of soil covers.

**Erosion-Control-Practice Factor (P)**

The erosion-control-practice factor accounts for ground-surface conditions that affect the velocity of the runoff. Specifically,  $P$  is defined as the ratio of the soil loss with a given surface condition to the soil loss with up-and-down-hill plowing. Such conditions would be contouring, terracing, roughening the soil, sediment basins, and control structures. Table 29.9 shows estimated values of  $P$  that apply to construction areas.

**Limitations of USLE**

The USLE is an empirical equation that initially was developed for agricultural applications. The USLE applies to relatively large, homogeneous soil areas and is based on long-term averages of rainfall and soil losses from runoff directly on the slope. It does not estimate deposition, nor does it estimate sediment yield at a downstream location.

Morphological features of agricultural land are different from urbanized developing land. Agricultural land typically is characterized by relatively long, regular, gentle slopes, whereas construction sites may have discontinuous and irregular land patterns. The land patterns of a construction site are a combination of steep slopes, sharp breaks, excavation holes and mounded piles of excavation soil. Since the USLE measures average annual soil loss, the erosion from the relatively short-term denuding-stabilization se-

**TABLE 29.6 Factors to Adjust LS Values for Successive Segments of a Slope where the Slope Length Exponent,  $m$ , Equals 0.5**

SEGMENT No. (TOP TO BOTTOM)	NUMBER EQUAL-LENGTH SEGMENTS INTO WHICH THE SLOPE IS DIVIDED FOR EVALUATION OF $L_s$			
	2	3	4	5
1	0.71	0.58	0.50	0.45
2	1.29	1.06	0.91	0.82
3		1.37	1.18	1.06
4			1.40	1.25
5				1.42

Source: Soil Conservation Service, *Water Management and Sediment Control for Urbanizing Areas*, Columbus, Ohio, 1978, p. 83.



TABLE 29.7 - a C Factors for Undisturbed Land<sup>1</sup>

VEGETATIVE CANOPY		COVER THAT CONTACTS THE SOIL SURFACE						
TYPE AND HEIGHT <sup>2</sup>	PERCENT COVER <sup>2</sup>	TYPE <sup>4</sup>	PERCENT GROUND COVER					
			0	20	40	60	80	95+
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.091	.043	.011
Tall grass, weeds, or short brush with average drop fall height of 20 in or less	25	G	.36	.17	.09	.038	.013	.003
		W	.36	.20	.13	.083	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.076	.039	.011
	75	G	.17	.10	.06	.032	.011	.003
		W	.17	.12	.09	.068	.038	.011
Appreciable brush or bushes, with average drop fall height of 6½ ft	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.087	.042	.011
	50	G	.34	.16	.08	.038	.012	.003
		W	.34	.19	.13	.082	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.078	.040	.011
Trees, but no appreciable low brush. Average drop fall height of 13 ft	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.089	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.087	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.084	.041	.011

<sup>1</sup> The listed *C* values require that the vegetation and mulch are randomly distributed over the entire area. For grazed forest land multiply these values by 0.7.

<sup>2</sup> Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to drop fall height and is negligible if fall height exceeds 33 ft.

<sup>3</sup> Portion of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

<sup>4</sup> G: cover at surface is grass, grasslike plants, decaying compacted duff, or litter. W: cover at surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

(Source: USDA, SCS 1977.)

quence, typical of a construction site, may not be indicative of the value obtained from the USLE. Runoff from an area above a disturbed slope was not a factor in establishing the USLE, yet runoff from up-slope areas does occur on construction sites. Therefore, use of the USLE, especially for construction sites, requires the site area to be broken down into homogeneous areas. The USLE should be applied to each individual area, with the sum being more representative of the soil-erosion estimate.

The use of the USLE provides an estimate of the erosion potential of a site. Using the USLE to compare different practices at a construction site is appropriate; however, using the USLE to compare one construction site to another is not recommended. Furthermore, the equation is unable to account for the deposition that occurs in the non-homogeneous irregular land forms typical of land development projects. Not all sediment eroded from a site can be classified as soil loss relative to the boundaries of the

**TABLE 29.7 - b C Factors for Mechanically-prepared Woodland Sites**

SITE PREPARATION	MULCH COVER <sup>1</sup>	SOIL CONDITION <sup>2</sup> AND WEED COVER <sup>3</sup>							
		EXCELLENT		GOOD		FAIR		POOR	
		NC	WC	NC	WC	NC	WC	NC	WC
	<i>Percent</i>								
Disked, raked, or bedded <sup>4</sup>	None	0.52	0.20	0.72	0.27	0.85	0.32	0.94	0.36
	10	.33	.15	.46	.20	.54	.24	.60	.26
	20	.24	.12	.34	.17	.40	.20	.44	.22
	40	.17	.11	.23	.14	.27	.17	.30	.19
	60	.11	.08	.15	.11	.18	.14	.20	.15
	80	.05	.04	.07	.06	.09	.08	.10	.09
Burned <sup>5</sup>	None	.25	.10	.26	.10	.31	.12	.45	.17
	10	.23	.10	.24	.10	.26	.11	.36	.16
	20	.19	.10	.19	.10	.21	.11	.27	.14
	40	.14	.09	.14	.09	.15	.09	.17	.11
	60	.08	.06	.09	.07	.10	.08	.11	.08
	80	.04	.04	.05	.04	.05	.04	.06	.05
Drum chapped <sup>5</sup>	None	.16	.07	.17	.07	.20	.08	.29	.11
	10	.15	.07	.16	.07	.17	.08	.23	.10
	20	.12	.06	.12	.06	.14	.07	.18	.09
	40	.09	.06	.09	.06	.10	.06	.11	.07
	60	.06	.05	.06	.05	.07	.05	.07	.05
	80	.03	.03	.03	.03	.03	.03	.04	.04

<sup>1</sup> Percentage of surface covered by residue in contact with the soil.

<sup>2</sup> *Excellent* soil condition—Highly stable soil aggregates in topsoil with fine tree roots and litter mixed in. *Good*—Moderately stable soil aggregates in topsoil or highly stable aggregates in subsoil (topsoil removed during rading), only traces of litter mixed in. *Fair*—Highly unstable soil aggregates in topsoil or moderately stable aggregates in subsoil, no litter mixed in. *Poor*—No topsoil, highly erodible soil aggregates in subsoil, no litter mixed in.

<sup>3</sup> NC—No live vegetation. WC—75-percent cover of grass and weeds having an average drop fall height of 20 in. For intermediate percentages of cover, interpolate between columns.

<sup>4</sup> Modify the listed C values as follows to account for effects of surface roughness and aging. *First year after treatment:* multiply listed C values by 0.40 for rough surface (depressions >6 in.); by 0.65 for moderately rough; and by 0.90 for smooth depressions <2 in.) *For 1 to 4 years after treatment:* multiply listed factors by 0.7. *For 4+ to 8 years:* use table 3-2. *More than 8 years:* use table 3-3.

<sup>5</sup> *For first 3 years:* use C values as listed. *For 3+ to 8 years after treatment:* use table 3-2. *More than 8 years after treatment:* use table 3-3. (Source: USDA, SCS 1977.)

site, since some soil is redeposited on site from natural deposition.

**The Revised Universal Soil Loss Equation (RUSLE)**

The RUSLE is a widely used computer program developed by the USDA—Agricultural Research Service to estimate rates of soil erosion caused by rainfall and associated overland flow. The RUSLE, while still using the same terms, incorporates data and additional theory for describing hydrologic and erosion processes not included in the original USLE. The new data and additional theory have allowed for additional refinement of the evaluating terms to suit conditions that are more specific.

**Water Erosion Prediction Project (WEPP)**

The WEPP soil-erosion model was developed by an inter-agency group consisting of the USDA—Natural Resources Conservation Service, USDA—Forest Service, USDI—Bureau of Land Management and others involved in soil and water conservation and environmental land planning and assessment. WEPP was completed to replace the USLE and its subsequent updates (RUSLE). The USLE is applicable for predicting sheet and rill erosion. The WEPP model's overland-flow-profile simulations are applicable to sheet and rill erosion, while its watershed simulations are applicable to field situations with multiple profiles, gullies or grassed waterways, and impoundments. The current ver-

**TABLE 29.8 C Factors for Various Types of Soil Covers**

MULCH ADEQUATELY CRIMPED INTO SOIL	C FACTOR
Bare Area	1.00
¼ ton straw mulch per acre	.52
½ ton straw mulch per acre	.35
¾ ton straw mulch per acre	.24
1 ton straw mulch per acre	.18
1½ ton straw mulch per acre	.10
2 ton straw mulch per acre	.06
3 ton straw mulch per acre	.03
4 ton straw mulch per acre	.02

(Source: Soil Conservation Services, *Universal Soil-Loss Equation*, Agronomy Note #50, Colorado SCS, 1977.)

sion of the model for use on personal computers is available over the Internet at [topsoil.nserl.purdue.edu](http://topsoil.nserl.purdue.edu).

**SEDIMENT CONTROL BEST MANAGEMENT PRACTICES**

Sediment-laden runoff is controlled on construction sites by retaining the eroded sediments on site through filtering or impoundment in trapping facilities. Sediment barriers like filter fabric and inlet protection are used to filter sediment from runoff. Trapping facilities typically are either sediment traps or sediment basins; the distinction depends on the acreage draining to the facility, the volume of sediment, and whether they are temporary or permanent. Facilities with drainage areas less than about 3 acres are sediment traps. Larger trapping facilities—sediment basins—are frequently designed as permanent facilities. The location and design of permanent sediment basins are such that they can easily be converted into retention or detention ponds after the project area is stabilized.

**Sediment Barriers**

Filtering controls (sediment barriers) intercept and filter both sheet-flow runoff as well as small volumes of concentrated flow. They can be used as stand-alone controls or with sediment traps to enhance their effectiveness. Stone, straw bales, and synthetic filter fabric are typical filtering devices.

Filter-fabric fence (Figure 29.13) is placed along a contour (i.e., perpendicular to the sheet-flow direction) to intercept the sheet-flow runoff according to the upland slope. The distance limitations for the use of filter-fabric fence are shown in Table 29.10. Filter-fabric fence should not be used in situations where it will intercept concentrated flows or act as a velocity check in ditches or swales. Filter-fabric cloth is made of synthetic fabric (polymer) that has a specified flow rate and tensile strength. The filter-fabric cloth is attached to vertical posts and embedded in a 4-in. × 4-in. trench at the bottom to keep the runoff from undermining

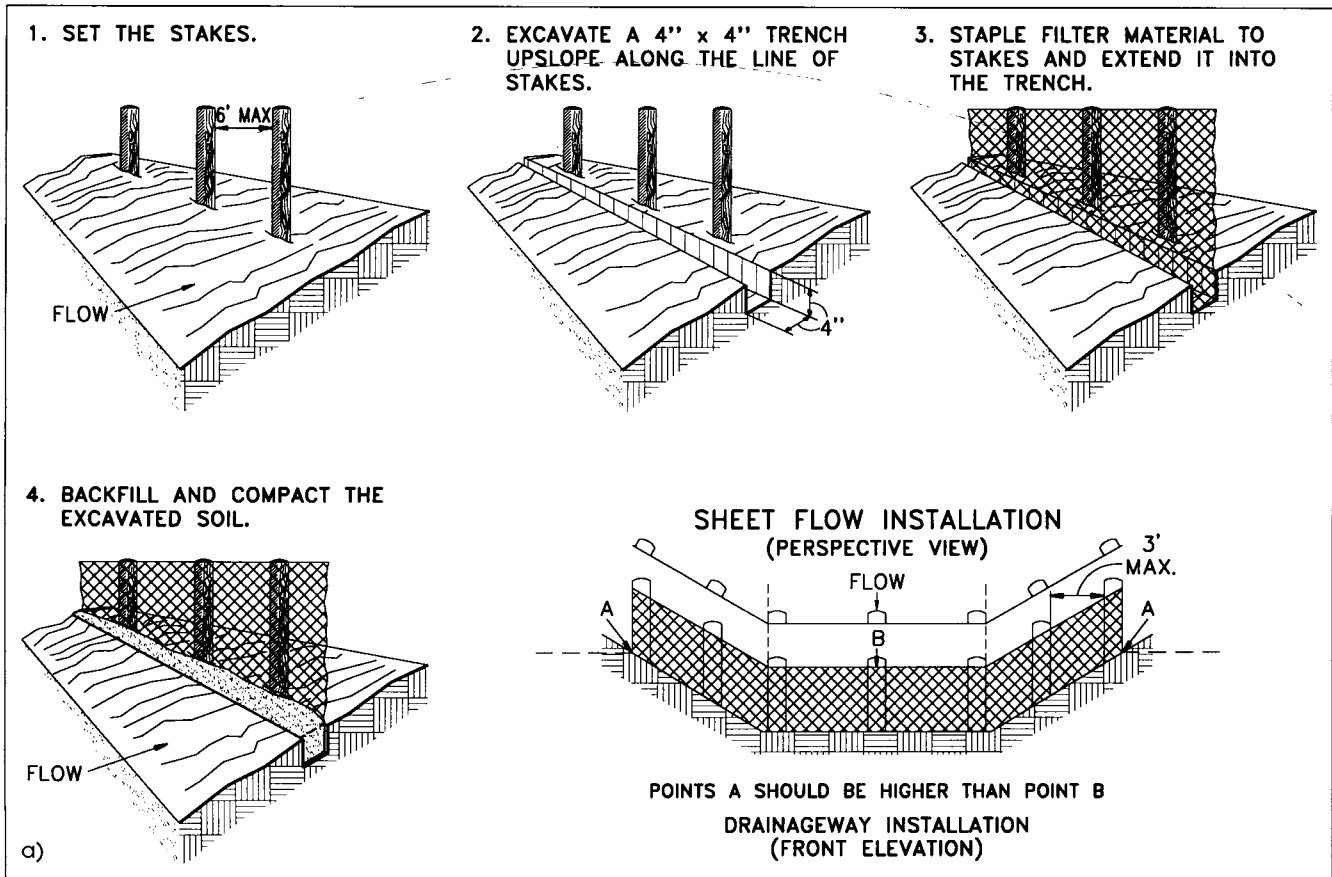
**TABLE 29.9 Erosion-control Practice Factor, P, for Construction Sites (Ports, 1973)**

Surface Condition With No Cover	Factor P
1. Compact, smooth, scraped with bulldozer or scraper up and down hill	1.30
2. Same as above, except raked with bulldozer root, raked up and down hill	1.20
3. Compact, smooth, scraped with bulldozer root, raked across the slope	1.20
4. Same as above, except raked with bulldozer root, raked across the slope	0.90
5. Loose, as in a disked plow layer	1.00
6. Rough irregular surface, equipment tracks in all directions	0.90
7. Loose with rough surface greater than 12-in depth	0.80
8. Loose with smooth surface greater than 12-in depth	0.90
<i>Structures</i>	
1. Small sediment basins:	
0.04 basin/acre	0.50
0.06 basin/acre	0.30
2. Downstream sediment basins with chemical flocculants without chemical flocculants	0.10 0.20
3. Erosion control structures normal rate usage high rate usage	0.50 0.40
4. Strip building	0.75

(Source: SWMM Users Manual which references *Use of the Universal Soil Loss Equation as a Design Standard*, ASCE Water Resources Engineering Meetings, Washington, D.C. 1973. Reprinted with permission from ASCE.)

the fabric. For added stability, the filter fence can be attached to wire support fence allowing a reduction in the tensile strength of the fabric. To maintain its effectiveness, filter-fabric fence should be inspected after each rainfall event and accumulated sediment removed when it reaches approximately one-half of the height of the barrier.

Where the length of flow contributing to conventional filter-fabric fence is too great, many permitting agencies recognize the use of filter-fabric cloth supported by chainlink fence, or super-silt fence (Figure 29.14) as an acceptable



**FIGURE 29.13** Construction details for filter-fabric fence. (Source: *Virginia Erosion and Sediment Control Handbook*, 1992.)

alternative. The distance limitation for using chainlink fence in support of the filter fabric is reflected in Table 29.11. Because of its cost of installation, the use of super-silt fence should be limited to areas where other conventional methods of trapping sediments would destroy environmentally-sensitive land.

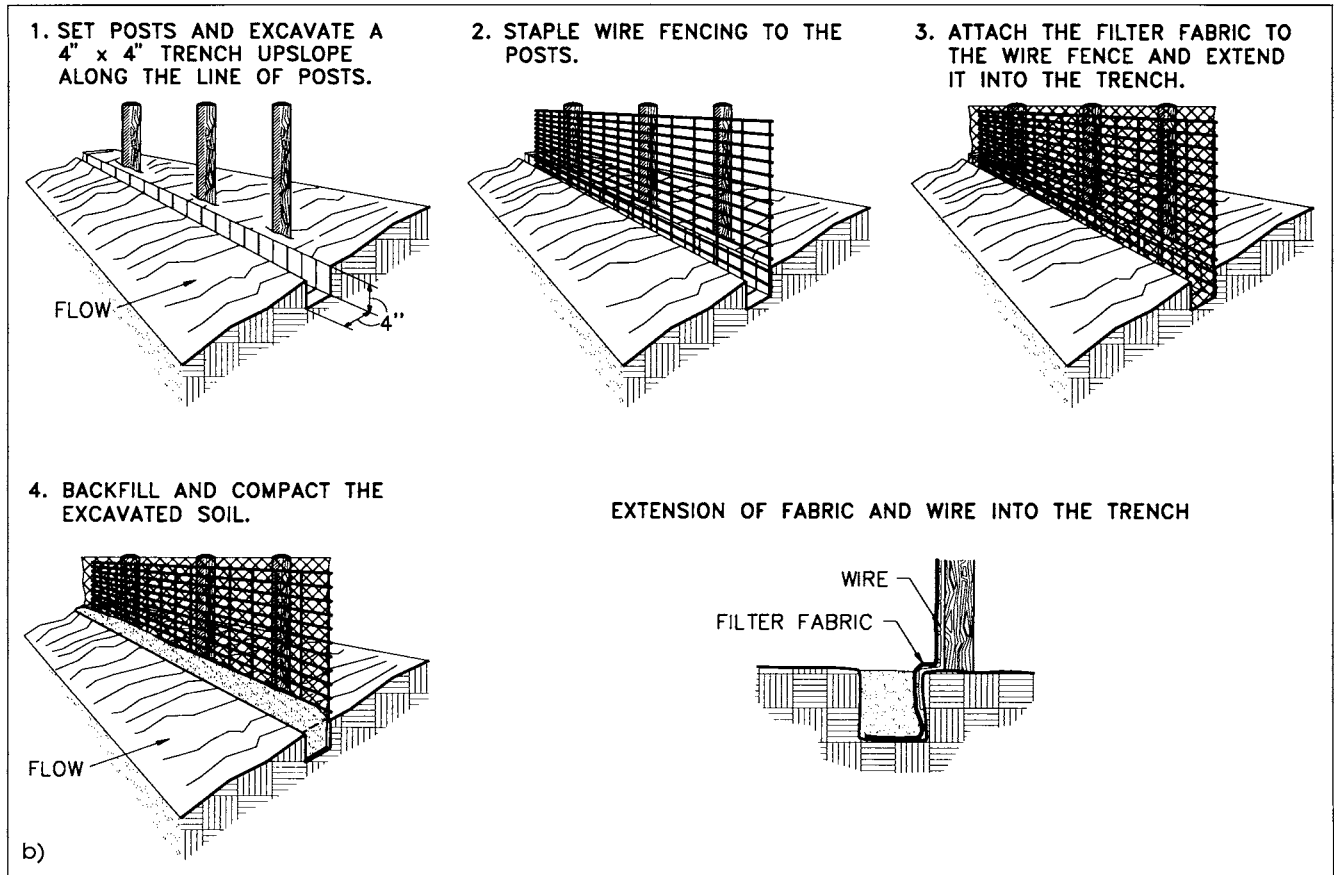
Staked, straw-bale dikes have long been used as a barrier to trap sediment. The bales are placed in a 4-in.-deep trench on the contour and staked to the ground with wooden stakes or reinforcing bars. The ends are tightly butted against one another to prevent gaps where runoff can seep through. Typically, this filtering system is placed at the toe of relatively short slopes. The use of straw-bale dikes is not recommended as a primary sediment-control device. Straw-bale dikes clog and deteriorate rapidly and require frequent maintenance. In addition, filter-fabric fence has more efficient filtering capabilities and has a significantly reduced flow-through rate. The filtering efficiency of straw is  $\approx 67\%$ , while filter fabric has an efficiency of  $\approx 97\%$ . The flow rate for straw is  $5.6 \text{ gal/ft}^2/\text{min}$  compared to filter fabric, which is about 10% of that value.

### Inlet Protection

The filtering devices employed for higher flows and velocities typically use stone to filter the sediments. Around drainage inlets a block-and-gravel device is used. The

block-and-gravel device consists of concrete blocks laid around the opening of the inlet. Wire mesh is placed over the blocks and 1-in. to 2-in.-diameter stone is placed against the wire mesh. This filtering method can be used both at curb and grate inlet openings, where an overflow capacity is necessary. Typically, this method of storm-inlet protection is used only as a secondary sediment-control device. The filtering efficiency of the gravel is limited, and this requires the primary treatment of sediment-laden runoff to occur below the discharge point of the inlet to capture the smaller silt and clay particles. Filter barriers can be used between the wire mesh and stone to increase the efficiency; however, the resultant ponding at the inlet is often undesirable.

Care should be taken with the use of a gravel and wire-mesh filter when protecting curb inlets that have been installed on a grade. Since the gravel filter reduces the ability of the inlet to intercept the stormwater runoff, more flow will by-pass the inlet and continue to the bottom of the grade. Then, the inlet located at the sump may be overwhelmed with the resultant increase in flow, causing flooding or deposition of sediment. For inlets on grade, a more effective approach may be to install a temporary berm below the inlet to direct flow to the inlet, or place small curb sediment traps above the inlet, allowing the full interception-capacity of an inlet to be utilized. Other types of inlet



**FIGURE 29.13** (Continued) Construction details for filter-fabric fence. (Source: Virginia Erosion and Sediment Control Handbook, 1992.)

protection include an excavated-drop-inlet sediment trap to capture the sediment through settling or filter-fabric fence for non-concentrated flows. Figure 29.15 shows variations of inlet protection.

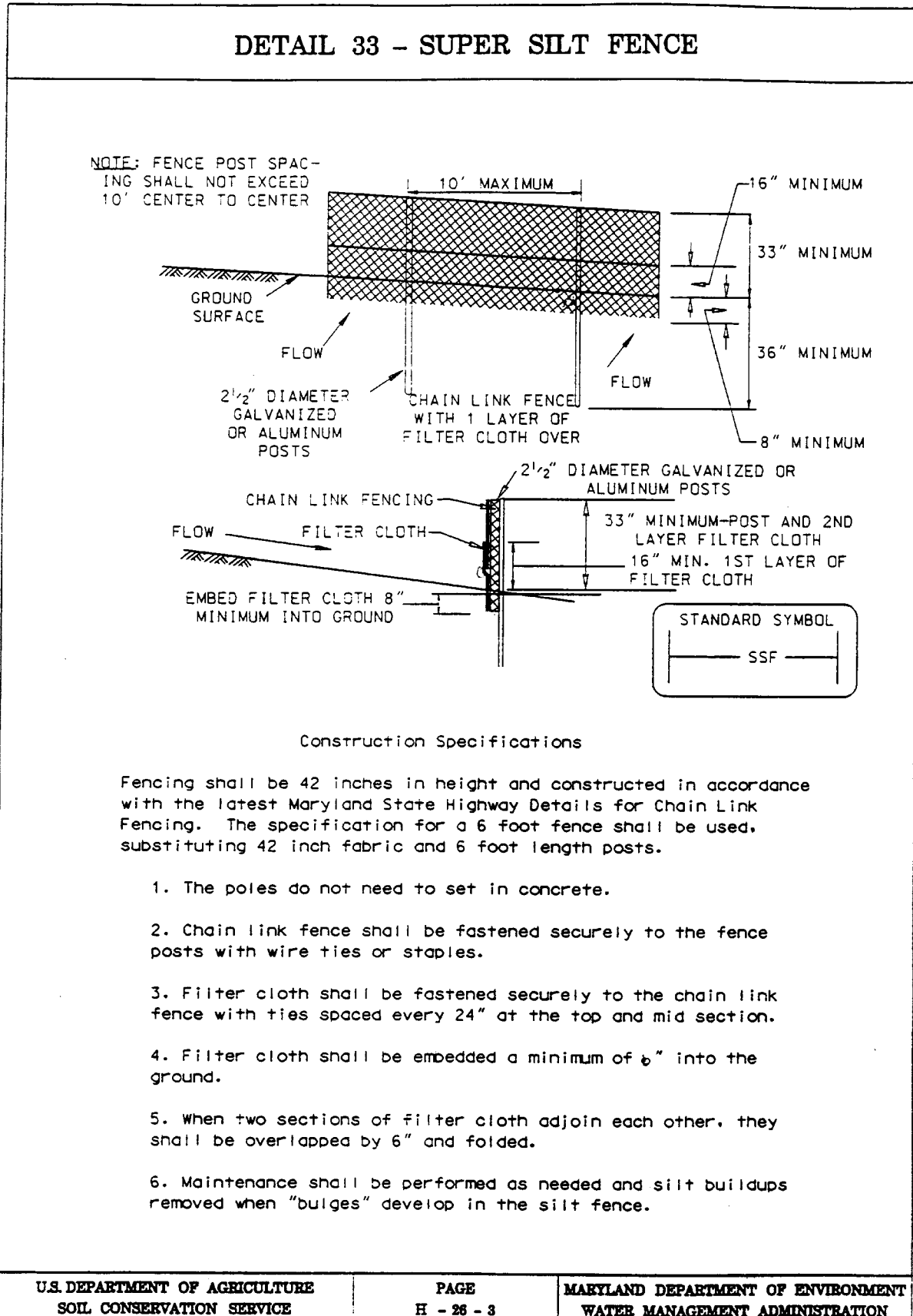
**Sediment Basins**

Sediment basins are designed to reduce the velocities and turbulence of the runoff to levels where the majority of the

suspended particles are given an opportunity to settle out by gravity. A sediment basin consists of a strategically placed embankment that takes advantage of the natural terrain to create an impounding area. The basin contains an outlet structure that releases the collected runoff at a slow rate. In some cases, the sediment basin contains an emergency spillway to allow the larger storms to pass through the pond unimpeded.

**TABLE 29.10 Filter-Fabric Fence Design Constraints**

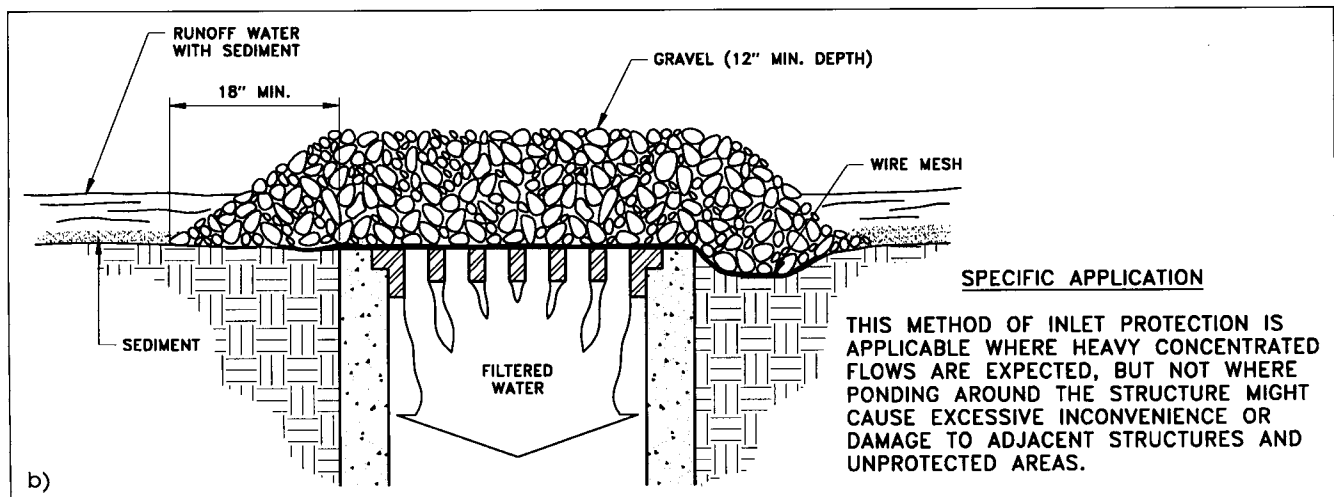
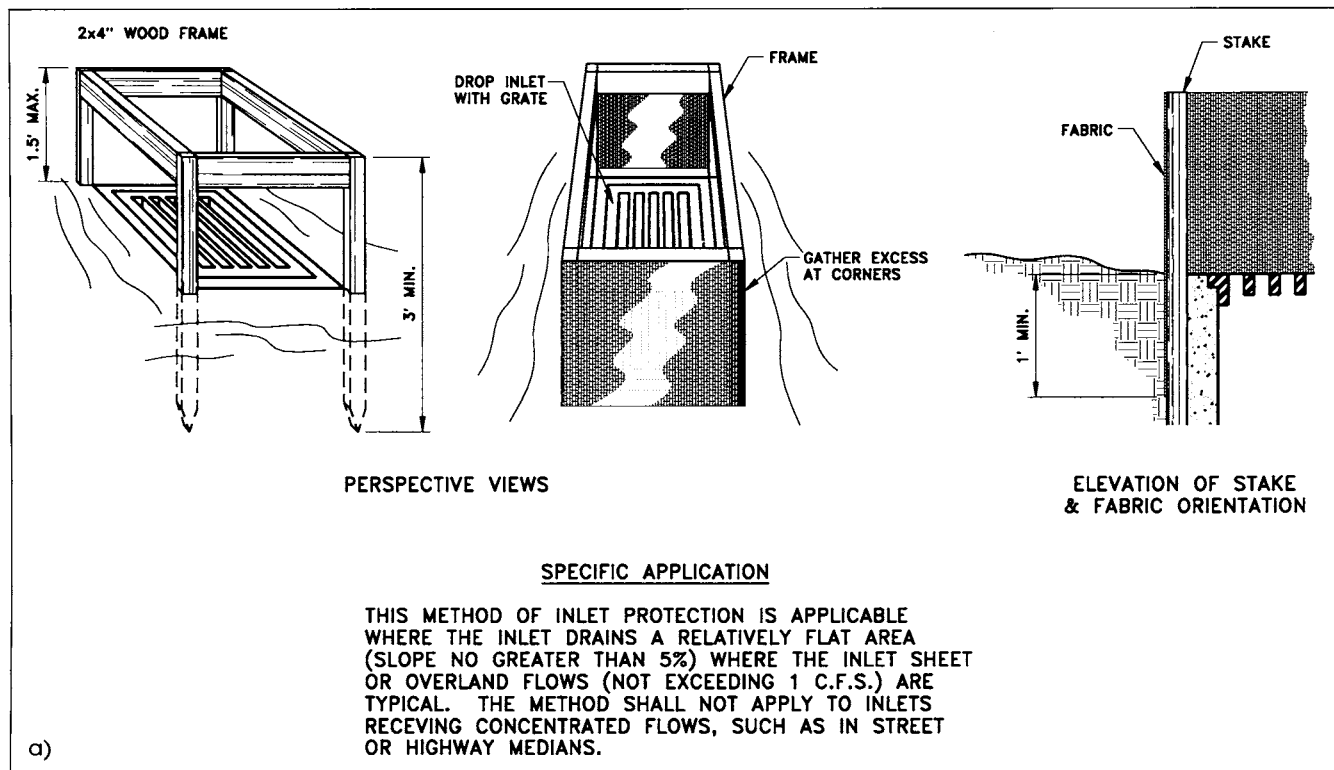
<b>SLOPE STEEPNESS</b>	<b>(MAXIMUM) SLOPE LENGTH</b>	<b>(MAXIMUM) SILT FENCE LENGTH</b>
Flatter than 50:1 (2%)	Unlimited	Unlimited
50:1 to 10:1 (2–10%)	125 ft	1000 ft
10:1 to 5:1 (10–20%)	100 ft	750 ft
5:1 to 3:1 (20–33%)	60 ft	500 ft
3:1 to 2:1 (33–50%)	40 ft	250 ft
>2:1 (>50%)	20 ft	125 ft



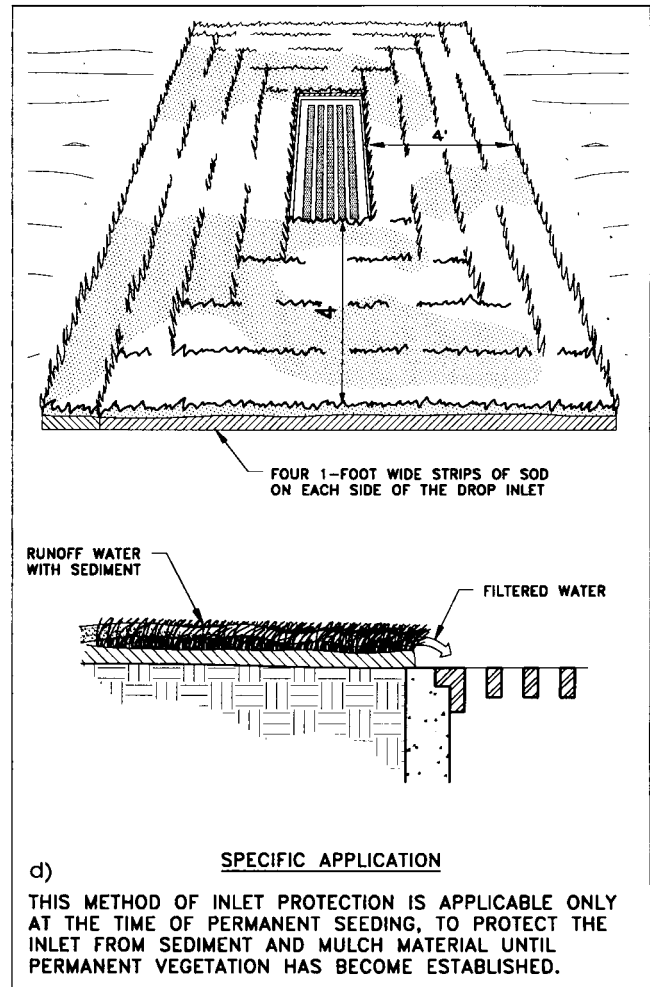
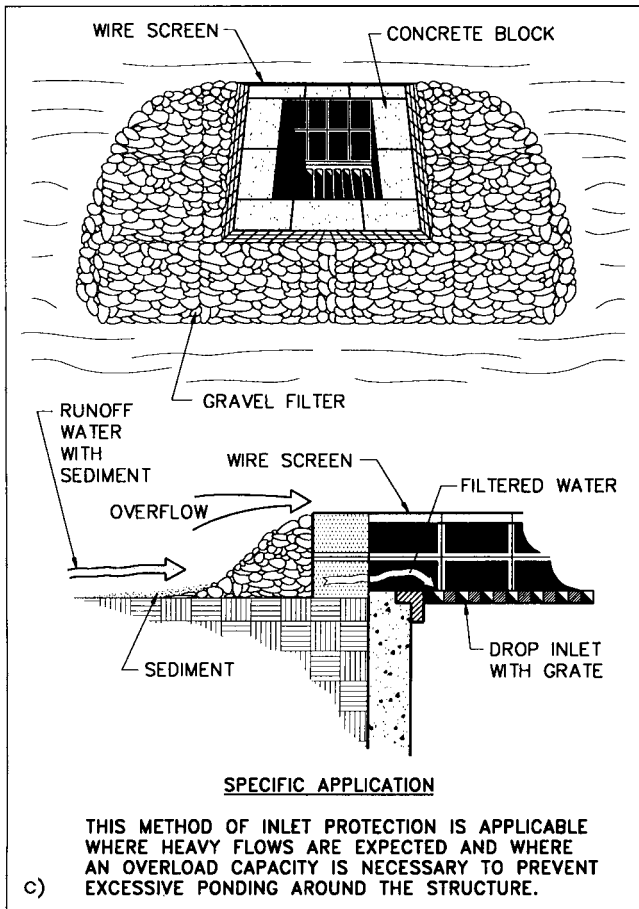
**FIGURE 29.14** Construction details for a super-silt fence. (Source: U.S. Department of Agriculture Soil Conservation Service, Maryland Department of Environmental Water Management Administration.)

**TABLE 29.11 Super-Silt-Fence Design Criteria**

SLOPE STEEPNESS	(MAXIMUM) SLOPE LENGTH	(MAXIMUM) FENCE LENGTH
0 to 10:1 (0–10%)	Unlimited	Unlimited
10:1 to 5:1 (10–20%)	200 ft	1500 ft
5:1 to 3:1 (20–33%)	100 ft	1000 ft
3:1 to 2:1 (33–50%)	100 ft	500 ft
>2:1 (>50%)	50 ft	250 ft



**FIGURE 29.15** Inlet protection.



**FIGURE 29.15** (Continued) Inlet protection.

Typically, the outlet structure is composed of a vertical pipe with several small-diameter holes near the bottom. This pipe is open at the top to allow for excess overflow of intermediate storms. The size and elevation of the holes control the release rate of the impounded water. The vertical pipe is the most common type of outlet structure because of its availability and the ease with which it can be installed and dismantled. Outlet structures other than the perforated vertical pipes are used for sediment basins intended to be converted to permanent facilities. Such facilities tend to have an outlet structure more elaborate and aesthetically pleasing, such as a multi-stage weir.

The size of a sediment basin is measured by its storage volume. The required storage volume is a function of the amount of eroded sediment and the volume of runoff entering the facility. Rainfall-runoff volumes and soil types are highly regionalized. Sizing a sediment basin depends on local municipalities' design standards, which are developed according to the regional conditions. In some cases, determining the volume of the basin may be as uncomplicated as applying a single constant to the drainage area (e.g., 100

cy of required storage-volume per drainage acre). This design parameter approximates an upper limit for the amount of sediment expected to be delivered to the facility for the design storm. The assumption here is that the design storm erodes a constant amount of sediment. This blanket value does not consider the soils or topographical features that vary from site to site nor the daily variations of the site conditions. In other cases, sizing the basin requires a detailed analysis of the on-site soils and their particle-size distribution. This information is used with the USLE or discrete-particle-settling theory to set the sediment basin size.

**Discrete-Particle-Settling Theory**

A discrete particle is one that does not change in size, shape or weight as it settles. The discrete-particle-settling theory describes the settling behavior of particles in an ideal basin in quiescent water. Particle settling, in such ideal conditions, depends on the properties of the fluid and the particles' characteristics. Any interaction between the particles within the fluid is assumed to be negligible.



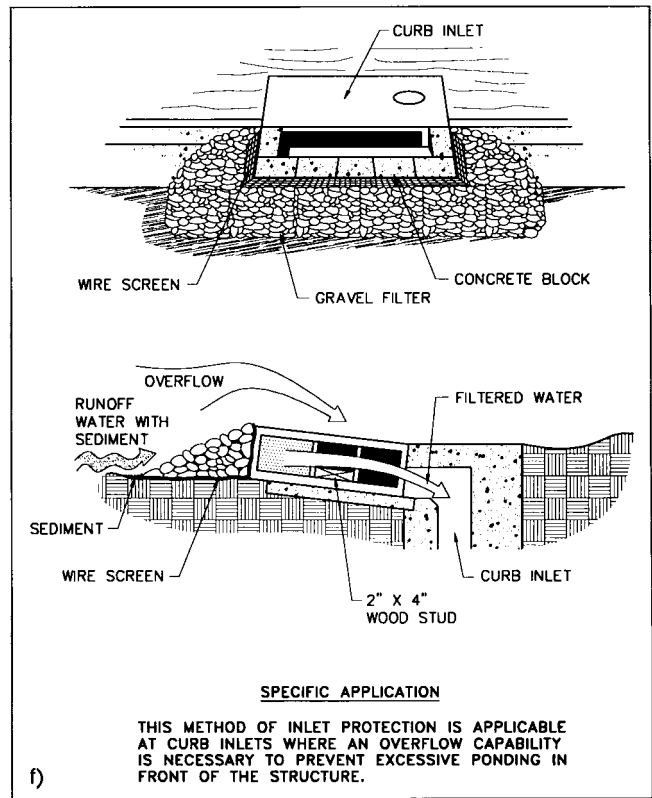
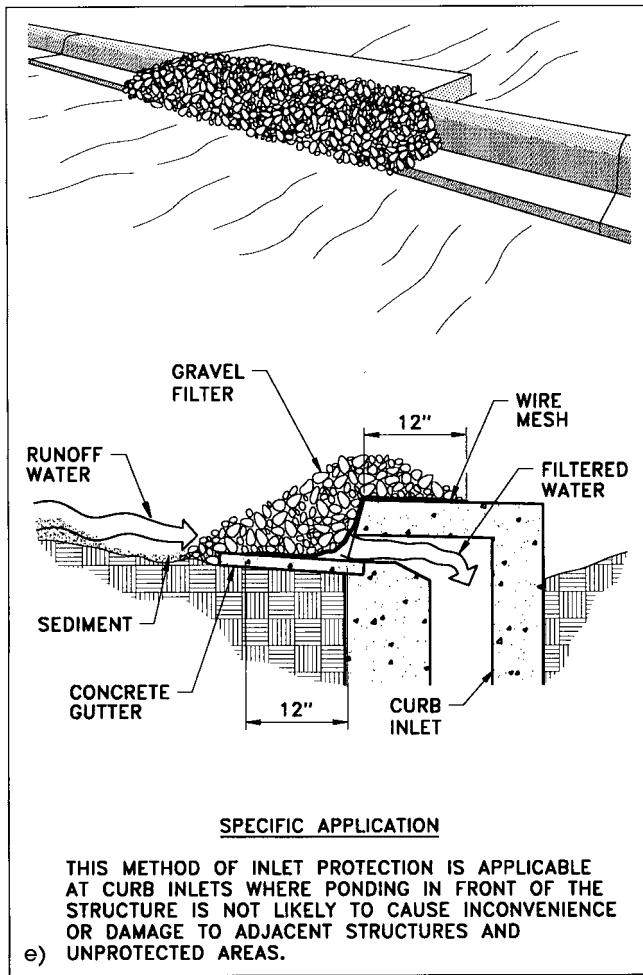


FIGURE 29.15 (Continued) Inlet protection.

A particle settling in a quiescent fluid accelerates under the influence of gravity until the driving force of gravity is balanced by the resisting drag force. At this point, the particle has achieved a terminal velocity and, as such, remains constant during the remainder of the falling distance. The terminal settling velocity,  $v_s$ , for a spherical particle is

$$v_s = \sqrt{\frac{4g(\rho_p - \rho_w)d_p}{3C_D\rho_w}} \quad (29.6)$$

where  $\rho_p$  = density of the spherical particle,  $\rho_w$  = density of water,  $g$  = acceleration due to gravity,  $C_D$  = coefficient of drag for the particle and  $d_p$  = diameter of the particle.

The drag coefficient,  $C_D$ , is approximated by

$$C_D = \frac{24}{N_R} \quad \text{for} \quad N_R < 1 \quad (29.7)$$

$$C_D = \frac{24}{N_R} + \frac{3}{N_R} + 0.34 \quad \text{for} \quad N_R \geq 1$$

where  $N_R$ , the dimensionless Reynolds number, is

$$N_R = \frac{v_s d_p \rho_w}{\mu} \quad (29.8)$$

with  $\mu$  = the absolute viscosity of water. Note that when  $N_R$  is less than one, the settling velocity for a sphere reduces to Eq. (29.9),

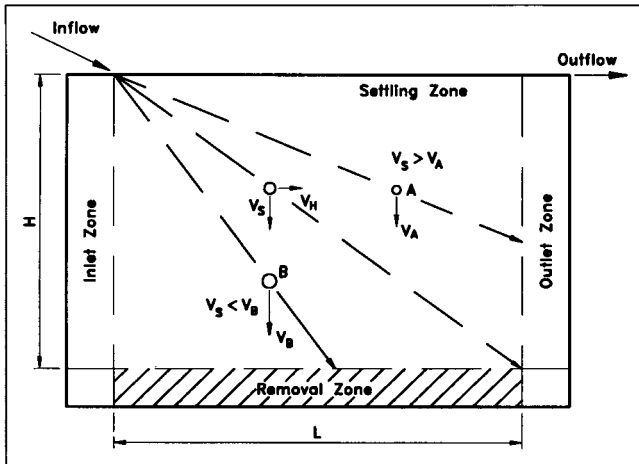
$$v_s = \frac{g(\rho_p - \rho_w)d_p^2}{18\mu} \quad (29.9)$$

which is Stoke's Law for the settling velocity of a sphere in laminar flow. This can be reduced to

$$v_s = 2.8d_p^2 \quad (29.10)$$

where  $v_s$  is in feet per second and  $d_p$  is in mm, assuming the specific gravity of the particle = 2.75 and a water temperature of 70°F.

An idealized, rectangular settling basin (Figure 29.16) consists of four zones: the inlet zone, the removal zone, the outlet zone and the settling zone. The length  $L$  is the distance between the inlet and outlet zones,  $H$  is the depth of



**FIGURE 29.16** Idealized rectangular settling basin.

the settling zone and  $W$  is the width of the basin. Under such idealized conditions, the incoming flow,  $Q_i$ , is steady and constant for the width of the basin. Particles within the incoming flow move horizontally through the basin with a horizontal velocity  $v_h = Q_i/(WH)$ . The vertical-velocity component is the settling velocity,  $v_s$ .

The design of an effective settling basin relies on the fact that an incoming particle travels the vertical height  $H$  and settles out before it travels the horizontal length  $L$  and is discharged. At or below the height  $H$  the particle is in the settling zone and is considered removed from suspension. The time,  $T_L$ , for the particle to travel the horizontal length,  $L$ , of the basin is given as

$$T_L = \frac{L}{Q_i/(W \times H)} \tag{29.11}$$

The time to travel the height,  $H$ , is given as:

$$T_H = \frac{H}{v_s} \tag{29.12}$$

Therefore, the minimum time needed for the particle to settle out of suspension occurs when the vertical time is equal to the horizontal time for the given particle. Equating Eq. (29.11) to Eq. (29.12) shows that  $v_s$  is

$$\frac{L}{Q_i/WH} = \frac{H}{v_s} \tag{29.13}$$

$$v_s = \frac{Q_i}{LW} = \frac{Q_i}{A_s}$$

Alternatively, the optimal surface area of the tank needed to capture particles with a settling velocity of  $v_s$  and greater occurs when  $A_s \geq Q_i/v_s$ , where  $A_s$  is the surface area of the settling zone. Hence, for ideal conditions, the design parameters of an effective settling basin do not include depth and volume. For all particles (with settling velocities of  $v_s$

and greater) entering the tank with the given discharge ( $Q_i$ ), the basin efficiency is determined by the surface area  $A_s$  relative to the discharge for steady-state conditions.

To summarize, the basic assumptions for discrete-particle-settling theory are:

1. Rectangular basin configuration
2. Constant inflow at one end
3. The horizontal and vertical velocities of particles are constant
4. The concentration of suspended particles, of each size, is the same at all points in the vertical cross section at the inlet
5. Particle settlement is unhindered by other particles
6. A particle is permanently removed from suspension when it reaches the bottom of the settling zone.

**Trap Efficiency**

Trap efficiency is the ratio of the amount (by weight) of sediment retained in a trapping facility to the total amount of sediment entering the facility. This efficiency is a function of the incoming sediment characteristics and the rate of flow through the facility. The rate of flow through the facility depends on the amount of inflow, available storage, and outflow rate. In turn, outflow rate depends on the configuration and the location of the outlet.

Settling behavior of particles depends on the size, shape, and density of the particle. The wide spatial variation of soil characteristics in a given region, coupled with the mixing of soil layers during land-disturbing activities, produces a varied distribution of particle sizes entering a trapping facility. Additionally, rainfall intensity and antecedent-moisture conditions affect the type and amount of sediment eroded. Hence, trap efficiency can fluctuate because of the variation of particle size, shape, and density distribution.

Trap efficiency, as determined through discrete-particle-settling theory, is based on the particle-size distribution of the sediment entering the facility. In theory, the basin is sized to remove all particles with settling velocities equal to or greater than the design particle size. A particle-size distribution curve of the incoming sediment provides an estimate of the trap efficiency. For example, if the design particle size is 200 microns and the particle-size distribution curve indicates that 60% of the sediment entering the facility is equal to or larger than 200 microns, the trap efficiency is approximately 60%. Note that this method of determining trap efficiency assumes that all particles entering the facility have the same density and behave as spherical particles during settling. Because of this assumption, this concept slightly underestimates the trap efficiency since some particles with a settling velocity greater than the design particle size will be retained.

An empirical curve for trap efficiency, proposed by Brune, is based on a study of 44 reservoirs throughout the United States. The curve (Figure 29.17) relates trap efficiency as the ratio of reservoir capacity to annual inflow. Because of the easily obtainable parameters of reservoir capacity and annual inflow, it is perhaps the model most widely used in practical design.

Sizing a trapping facility for a predetermined efficiency depends on the design philosophy that one is willing to accept. An alternative to using discrete-particle-settling concepts to size a settling basin is a method proposed by S.M. Boysen (1974). Boysen's method uses Brune's curve and the universal-soil-loss equation for development of the "urban-sediment-yield equation."

Boysen's urban-sediment-yield equation expresses off-site sediment yield caused by urban construction as

$$\begin{aligned} &\text{Sheet and rill erosion + gully erosion} && (29.14) \\ &- \text{sediment deposition} - \text{sediment trapped} \\ &= \text{off-site sediment yield (in tons)} \end{aligned}$$

The basic assumptions for the development of this method are:

1. The USLE is applicable to predict sheet and rill erosion.

2. Gully erosion is prevented by on-site control measures and, therefore, is assumed to be zero.
3. Sediment deposition before reaching the trapping facility is assumed to be zero.
4. Brune's curve for trap efficiency is applicable.

If the USLE (Eq. (29.1)) predicts the on-site eroded sediment yield and (TE) is trap efficiency, then

$$\begin{aligned} \text{Sediment trapped} &= (R \times K \\ &\times (LS) \times C \times P) * (TE) \end{aligned} \quad (29.15)$$

and the off-site (i.e., beyond the sediment basin) sediment yield can be written as,

$$\begin{aligned} &(R \times K \times (LS) \times C \times P) * (1 - TE) && (29.16) \\ &= \text{Off-site Sediment Yield} \end{aligned}$$

Assume that a prescribed maximum value limits the sediment yield allowed at a site. If the site erosion is below this value, no erosion controls will be necessary. If the site erosion, as determined by the USLE, exceeds this prescribed upper limit, then on-site erosion-control measures are necessary. The objective in the design of the on-site control measures is to set the trap efficiency at the difference be-

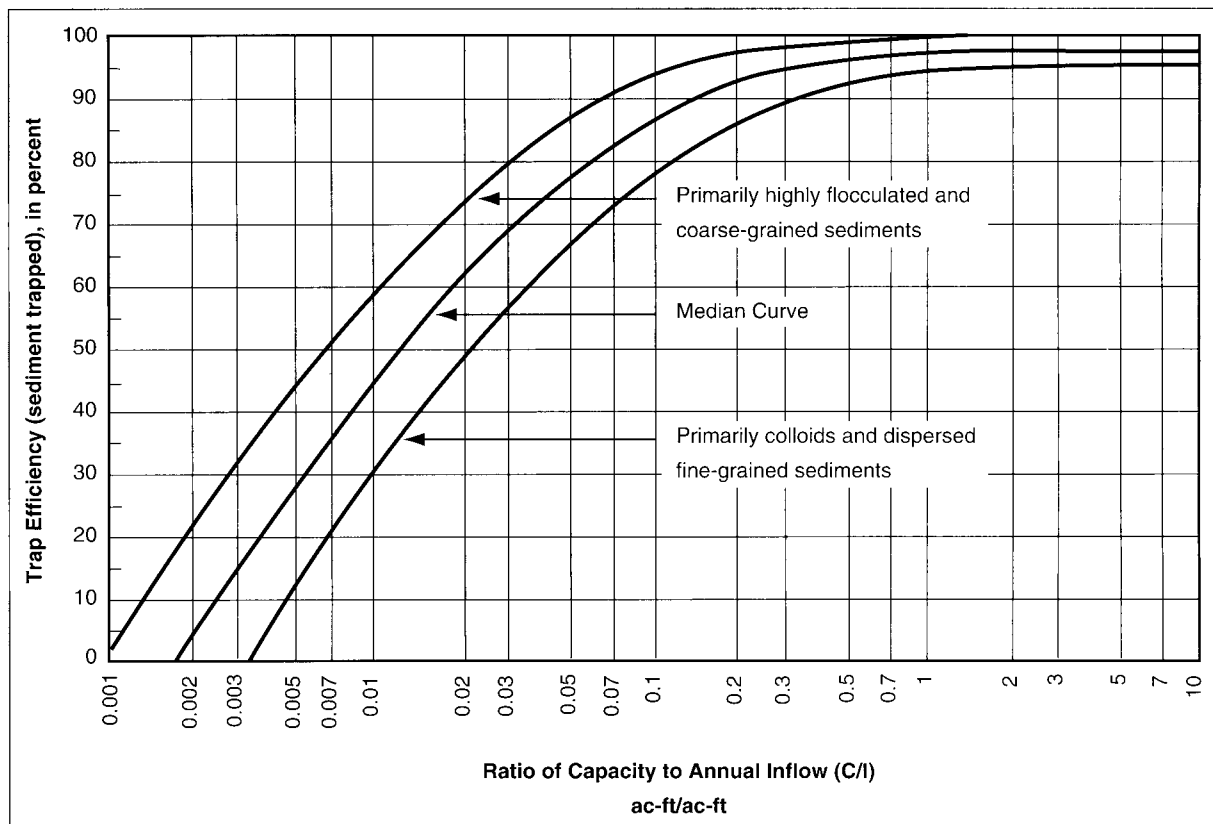


FIGURE 29.17 Trap efficiency of reservoirs. (Source: Brune, 1953.)

tween the actual eroded sediment and the allowable sediment-yield limit.

Limiting off-site sediment yield to some maximum allowable value is expressed by rewriting Eq. (29.15) as

$$\frac{(R \times K \times (LS) \times C \times P)}{\text{Allowable Sediment Yield}} = \frac{1}{(1 - TE)} \quad (29.17)$$

where the left side of the equation is shown as the ordinate on Figure 29.18.

The abscissa of the graph is the values of capacity-inflow ratio from Brune's curve converted to cubic yards of capacity per 1 in. of runoff. Brune's curves are redrawn to correspond to the units of the graph of Figure 29.18.

**EXAMPLE 3:** A 10-acre site, located in northeastern North Carolina, is being developed for commercial use. Approximately 1 acre has slopes of 5H:1V (20%) for 50-ft lengths, and the remaining 9 acres has average slopes of 4% for 250 ft. The soils range mostly from medium silt to fine sand. The average annual runoff is 15 in. The geotechnical consultant has provided the particle-size distribution of the soils

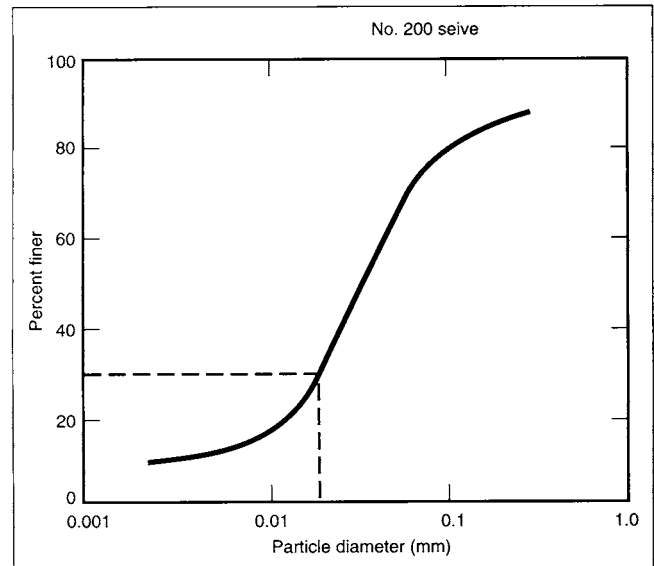


FIGURE 29.19 Particle-size distribution for example 3.

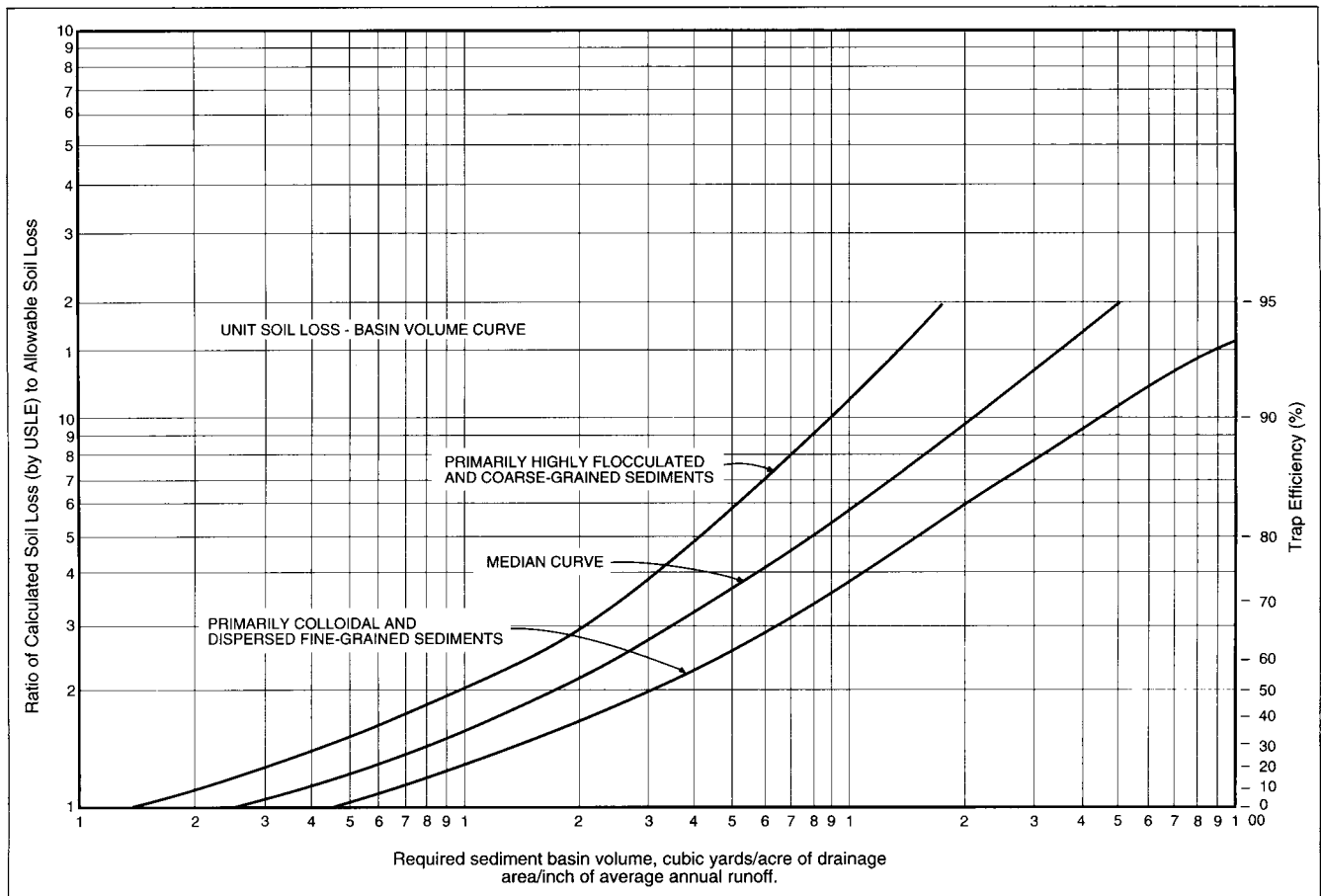


FIGURE 29.18 Sediment basin volume versus ratio of calculated soil loss to allowable soil loss. (From Boysen SM: *Predicting Sediment Yield in Urban Areas*, Proceedings National Symposium on Urban Rainfall and Runoff and Sediment Control, July 29–31, 1974, Lexington, KY. Reprinted with permission from ASAE.)

as shown in Figure 29.19. It is assumed that the total site drains to a single location.

Find the sediment-storage volume required using discrete-particle-settling concepts and Boysen's method if the required efficiency is 70%.

**SOLUTION:** Use the USLE, Eq. (29.1), to find the annual soil loss from the site, assuming the entire site will be denuded. Two separate calculations are necessary to account for the two different slopes. From Figure 29.11, the *R*-factor is estimated at 300. Use Eq. (29.2) to find two *LS* values:

$$(LS)_{20\%} = \left(\frac{50}{72.6}\right)^{0.5} \times \left(\frac{430(0.20)^2 + 30(0.20) + 0.43}{6.574}\right) = 3 \quad (29.18)$$

$$(LS)_{4\%} = \left(\frac{250}{72.6}\right)^{0.4} \times \left(\frac{430(0.4)^2 + 30(0.04) + 0.43}{6.574}\right) = 0.6$$

The cropping-management factor, *C*, during the grading operations is 1 if no vegetation or mulching is incorporated during grading. The erosion-control practice, factor *P*, is 1.3 from Table 29.9 for compacted soil. From Table 29.5, *K* is estimated at 0.3. The total estimated soil loss to the basin is then,

$$A_{20\%} = (300)(0.3)(3)(1)(1.3) = 351 \text{ tons/ac/yr}$$

$$A_{5\%} = (300)(0.3)(0.6)(1)(1.3) = 70 \text{ tons/ac/yr} \quad (29.19)$$

which corresponds to (351 tons/ac/yr)(1 ac)=351 tons/yr and (70 tons/ac/yr)(9 ac)=630 tons/yr. At a unit weight of 80 lbs/ft<sup>3</sup>, the total volume of soil is 908 yd<sup>3</sup>/yr.

From the particle-size distribution graph, 70% of the particles are larger than 0.02 mm. Assuming that the eroded sediment has a similar particle-size distribution, the amount of sediment load to the sediment basin is (0.70)(908 yd<sup>3</sup>/yr) = 636 yd<sup>3</sup>/yr.

The surface area, *A<sub>s</sub>*, depends on the inflow, *Q<sub>i</sub>*. Goldman, et al. (1986) recommend using the average runoff for the 10-yr, 6-hr storm. A 10-yr, 6-hr storm in this locality produces 4.5 in. of rainfall. Using the rational method, the average runoff in this example is

$$Q_{avg} = ACi_{avg} \quad (29.20)$$

$$= (10 \text{ ac})(0.5)(4.5 \text{ in.}/6 \text{ hr}) = 3.75 \text{ cfs}$$

Determine the settling velocity for the design particle size. From Eq. (29.10) (assume Stoke's Law applies), the settling velocity is

$$V_s = 2.8 \times 0.02^2 = 0.001 \text{ fps} \quad (29.21)$$

The Reynolds number should be checked to see if it is in the allowable range of 10<sup>-4</sup> to 0.5. Use Eq. (29.8) to find the Reynolds number for this particle size. Assume a temperature of 70°F

$$N_R = \frac{0.85 \times 0.001 \times 0.02 \times 62.4}{(1.94 \times 10^{-5}) \times 304.8} \quad (29.22)$$

$$= 0.18$$

This falls within the allowable range. If the Reynolds number had been greater than 0.5, then Eqs. (29.7) and (29.8) would have been used in an iterative process to obtain the settling velocity. (For example: if the computed *N<sub>R</sub>* is greater than 0.5, Eq. (29.7) is used to obtain *C<sub>d</sub>* using this *N<sub>R</sub>*. This *C<sub>d</sub>* is entered into Eq. (29.6) to find *v<sub>s</sub>*. Compare this *v<sub>s</sub>* to the previous *v<sub>s</sub>*. If the difference is less than an acceptable tolerance  $\epsilon$ , the recent *v<sub>s</sub>* is sufficiently accurate. A tolerance value of less than or equal to 0.005 should suffice in most cases. If the difference between the *v<sub>s</sub>* values is greater than 0.005, repeat the procedure using the most recent value of *v<sub>s</sub>*.) Determine the surface area of the basin as per Eq. (29.3).

$$A_s \geq \frac{Q_i}{v_s} \geq \frac{3.75 \text{ cfs}}{0.001 \text{ fps}} \geq 3750 \text{ ft}^2 \quad (29.23)$$

The storage depth is found by dividing the volume by the surface area (assuming the surface area is constant for the full depth of the sediment basin).

$$d_s = \frac{(636 \text{ yd}^3)(27 \text{ ft}^3/\text{yd}^3)}{3750 \text{ ft}^2} = 4.6 \text{ ft} \quad (29.24)$$

This is the depth required if the sediment basin is to capture 70% of the annual sediment load with no intermittent cleaning. Using a length-to-width ratio of 2:1, the minimum dimensions of the sediment basin are 88 ft × 44 ft × 6.6 ft. Allow an additional 2 ft in. depth to serve as the settling zone. These dimensions may be modified if periodic cleanout is specified and if the surface area is increased to reduce the depth.

**Boysen's Method**

For 70% efficiency, read the corresponding basin volume relating to the curve labeled primarily colloids and dispersed fine grain sediments as 8 yd<sup>3</sup>/ac/in. of annual runoff. The total volume for sediment storage is then

$$\text{Vol.} = (8 \text{ yd}^3/\text{ac}/\text{in.})(10 \text{ ac})(15 \text{ in. of runoff})$$

$$= 1200 \text{ yd}^3 (\approx 32,400 \text{ ft}^3) \quad (29.25)$$

Note, however, that if the median curve is used, the value

changes from 8 yd<sup>3</sup>/ac/in. to 4.5 yd<sup>3</sup>/ac/in., and the total sediment volume becomes only 18,000 ft<sup>3</sup>.

This example illustrates two methods to determine the size and storage volume for a sediment pond. Note: the design is incomplete in that hydrologic routings should be done to determine the size for the runoff volume. This entails the need to design outlet structures and emergency spillways, and to determine the overall depth based on the design storms selected for the hydrologic analysis.

### Drawdown Time

One assumption of the discrete-particle-settling theory is that the settling basin remains relatively full. Since some sediment traps are completely dewatered between storms, the settling behavior of the particles will not be as prescribed by the discrete-particle-settling theory. This will negatively affect the efficiency of the trapping facility.

The effectiveness of a sediment basin is enhanced if it remains relatively full over a period of time to establish quiescent conditions, which allow the sediment particles to settle out. The recommended time it takes for a basin to dewater is on the order of 6 to 24 hr. As a result, the volume of the sediment basin must be sized to store adequately a portion of the design storm for an extended period of time; as well, it must be sized to pass safely storms of higher design frequencies.

Drawdown time for a basin is found by using the orifice equation to determine the outflow discharge. The orifice equation is used to calculate the flow through a small opening (orifice) for a given headwater. The orifice equation is

$$Q = C_d \times A_o \times \sqrt{2gh_c} \quad (29.26)$$

where  $C_d$  = coefficient of discharge ( $\approx 0.60$  to  $0.70$ ),  $A_o$  = area of the orifice,  $g$  = acceleration due to gravity,  $h_c$  = distance from the centerline of the orifice to the water surface and  $Q$  = discharge through the orifice.

In sedimentation theory, the mean hydraulic-detention time is defined as the discharge divided by the volume,  $T = Q/V$ . Using the orifice equation and the definition of the mean hydraulic-detention time, the time,  $T$ , to dewater a sediment basin is

$$T = \frac{A_s \sqrt{2h_c}}{A_o C_d \sqrt{g}} \quad (29.27)$$

Perforated pipe-outlet structures have several different configurations of holes, as shown in Figure 29.20. Various configurations range from a single hole to several holes at different elevations. Usually the holes are covered with wire mesh screen with gravel piled against the screen to prevent clogging by debris. The gravel, which also acts as a filter, can affect the flow through the orifice(s). The above equations assume that this effect is negligible. If a cluster of holes is used instead of a single orifice,  $h_c$  is the depth to the centroid of the holes and  $A_o$  is the total area of the holes.

### Increasing Detention Time with Baffles

Detention time can be increased by increasing the flow path through the basin. This can be done by configuring the basin such that the length-to-width ratio is maximized. The value usually recommended as the lower limit for design purposes is 2.0. However, this should not be taken to the extreme. A proposed design may size a basin as 5 ft wide by 100 ft long ( $l:w = 20$ ), and this, in effect, is more representative of a channel than a sediment basin. The basin loses its effectiveness if the design causes flow-through velocities where the particles have no chance of settling out.

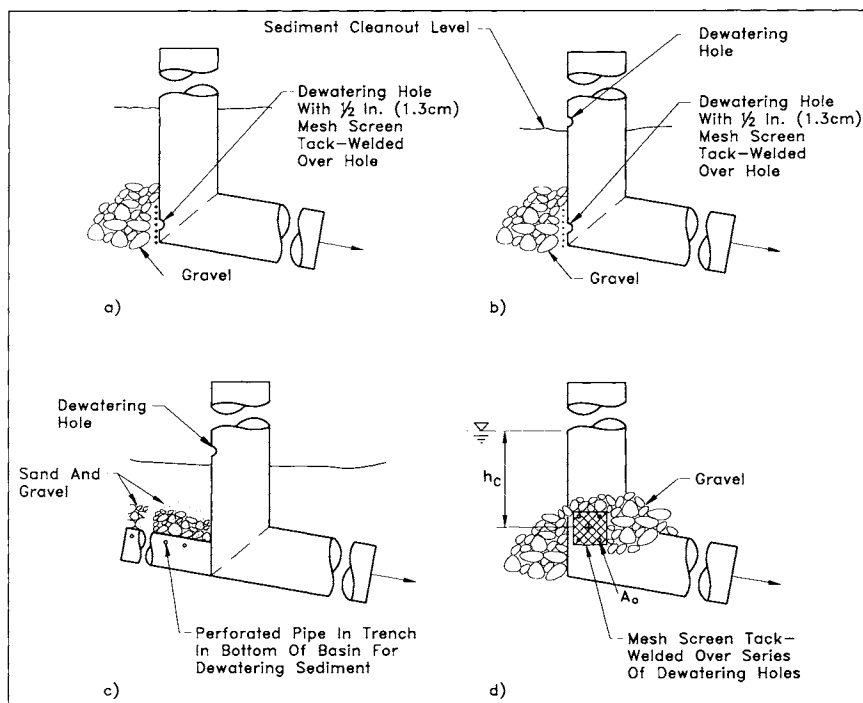
On urban construction sites, space is a premium and no contractor wants to work around obstructions if it can be avoided. Occasionally, the location and configuration of sediment basins conflicts with the contractor's operations. Typically, topographical constraints of a site dictate the location and configuration of the sediment basin.

The two main design parameters of a sediment basin are storage volume and detention time. Normally, obtaining the required storage volume is not the problem, even in constricted work areas, since you normally can excavate deeper. However, if the minimum length-to-width ratio (a controlling parameter for detention time) cannot be met because of site constraints, digging deeper will not help. One solution to the problem of detention time is to install baffles in the sediment basin.

Baffles are devices that actually serve two primary purposes:

1. Baffles enhance the basin shape to increase the length-to-width ratio, thereby increasing the flow path between the inflow and outflow point. The net result is an increase in detention time.
2. Baffles help to prevent short-circuiting, which is the movement of flow through the basin in such a manner that storage areas are by-passed, resulting in non-uniform distribution of sediment over the bottom area. Dead-storage areas, those areas where little or no sediment accumulates, result in reduced trapping efficiency. Baffles minimize dead-storage areas by distributing the sediment deposits over a wider area in the basin and increasing the efficiency (see Figure 29.21).

Baffles are placed at the mid-point between the primary inflow point and outflow point. The shortest distance from the inflow point around the baffle to the outflow point is the effective length,  $L_e$ . The effective width is defined as  $W_e = A/L_e$ , where  $A$  is the surface area of the sediment basin at the normal-pool level. The length and orientation of the baffle is modified so that the ratio  $L_e/W_e$  is at least 2.0. This minimum ratio applies to all flow paths from inflow points that convey 30% or more of the peak runoff into the basin. Figure 29.22 shows the recommended location and orientation of baffles for various configurations of sediment basins.

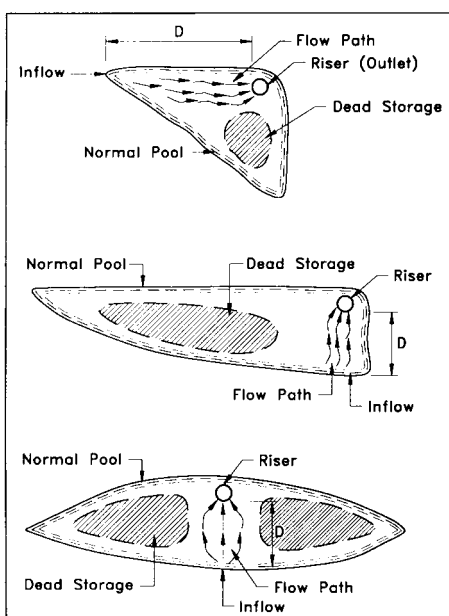


**FIGURE 29.20** Selected riser configurations to dewater a sediment basin. (From Goldman, S., et al: *Erosion and Sediment Control Handbook*, 1986. Reproduced with permission of McGraw-Hill.)

**Design Procedure for Sediment Basins**

Figures 29.23 and 29.24 schematically show a typical temporary sediment-basin layout and its components. The previous discussion focused only on sizing the sediment basin. The complete design involves structural considerations as well as hydraulic and hydrologic analysis. An outline summary of designing a sediment basin follows.

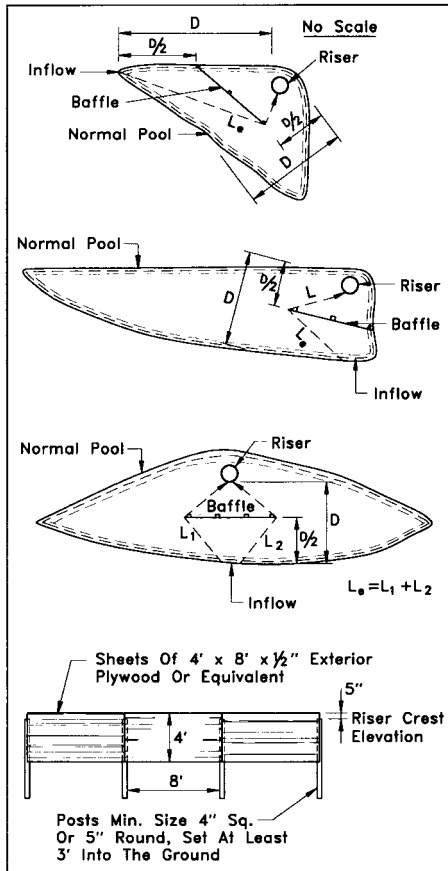
1. Determine the basin sediment volume based on local criteria or an involved analysis of the design particle size and settling criteria.
2. Determine basin location and basin shape based on site conditions. Consider baffles to establish adequate length-to-width ratios.
3. Determine the volume of inflow for the design storm through hydrologic analysis and set the height of embankment.
4. Design the principal spillway, taking into consideration the allowable release rate to ensure sufficient detention time for particle settling.
5. Design the emergency spillway (when required) for the larger design storm. The design should consider the hydraulic capacity of the spillway as well as the structural design to account for erosive velocities and buoyancy.
6. Determine whether additional devices are required, such as anti-seep collars, anti-vortex devices, trash racks, dewatering capabilities, and outlet protection.
7. Consider the safety aspect of the facility as with any permanent or temporary impoundment of water. Fence the facility to discourage access.



**FIGURE 29.21** Dead-storage areas for selected basin shapes. (From Haan CT, Barfield BJ: *Hydrology and Sedimentology of Surface Mined Lands*, © 1978, University of Kentucky Press.)

**Sediment Traps**

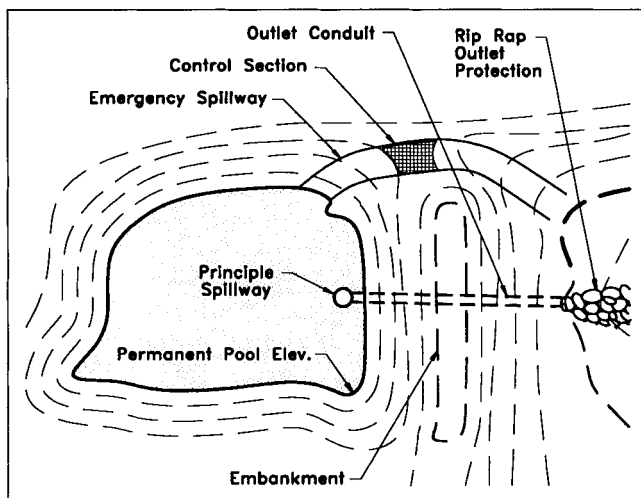
Sediment traps operate through the combined effects of settling and filtering. Here, the configuration and type of outlet affects the settling and filtering process. Sediment traps are



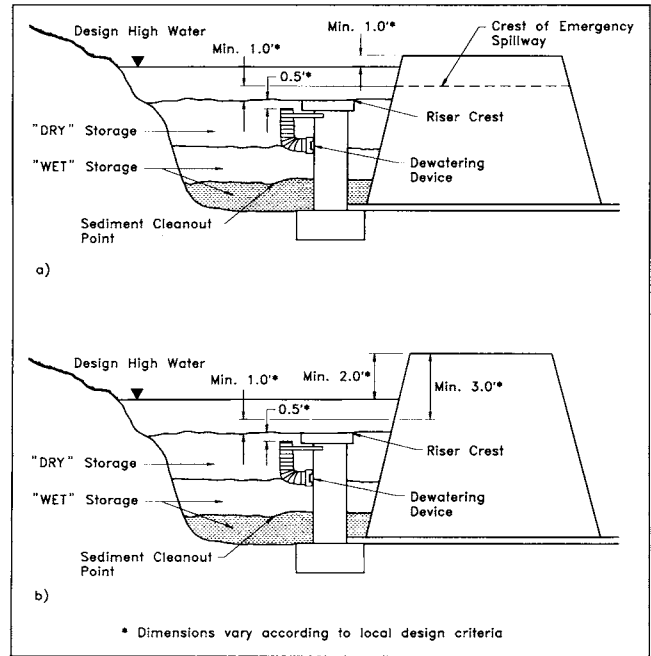
**FIGURE 29.22** Recommended placement of baffles.

similar to sediment basins in that they are an excavated hole or impoundment created by an earth embankment. However, drainage areas to sediment traps are typically limited to 3 acres or less.

Sediment traps use a variety of material and configurations for their outlet structures, the selection of which depends on local standards. Some localities do not accept



**FIGURE 29.23** Schematic plan view of sediment basin.



**FIGURE 29.24** Schematic elevation view of (a) sediment basin with emergency spillway and (b) without emergency spillway. (Source: *Virginia Erosion and Sediment Control Handbook*, 1992.)

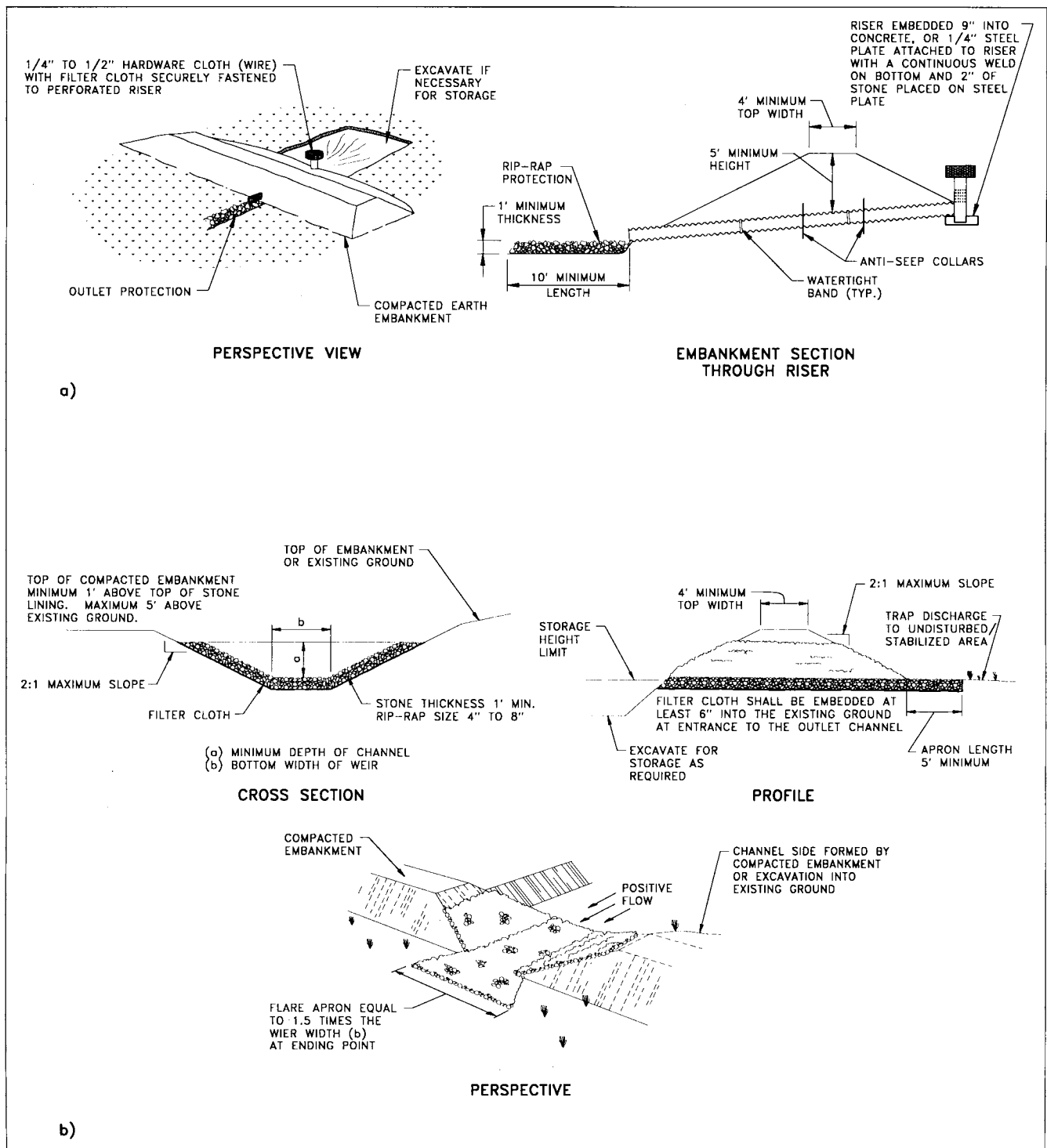
certain types of outlet structures, either because of dictating local conditions or the biases of the local reviewing agency. Outlet structures can be grass aprons, stone or riprap weirs, or even perforated riser pipes. Certain types of inlets used in grass areas are adaptable to sediment traps. Figure 29.25 shows several of these standards.

Once again, the methods of sizing sediment traps depend on local design criteria. Because the drainage area to the trap is relatively small, the engineer generally uses rule-of-thumb methods to size the trap instead of detailed analysis and computations. In many cases, the design standard for sizing the sediment trap is a single ratio applicable to all sites in a geographic location. The value is based on local hydrologic conditions and prevailing soil conditions and has the units of  $\text{ft}^2/\text{ac}$  or  $\text{ft}^3/\text{ac}$ . The first defines the surface area of the sediment trap per drainage area, whereas the latter dictates the volume to drainage area ratio as the design parameter.

It is best to minimize excavating and grading when constructing the sediment traps. Taking advantage of the natural topography does this. The outlet structure needs to be located at the natural low point of the trap. This location facilitates dewatering and minimizes grading in the trap area. For the sediment traps that use weir-type outlets, the recommended crest length, in feet, is 3 to 6 times the drainage area, in acres, serving the facility.

If properly designed, the sediment trap can provide adequate protection against downstream erosion problems. The ineffectiveness of a trap can, in many cases, be attributed to improper installation and maintenance. It is important for the trap to be cleaned out after the sediment has





**FIGURE 29.25** Sediment traps. (Source: Maryland Standards and Specifications for Soil Erosion and Sediment Control, 1991.)

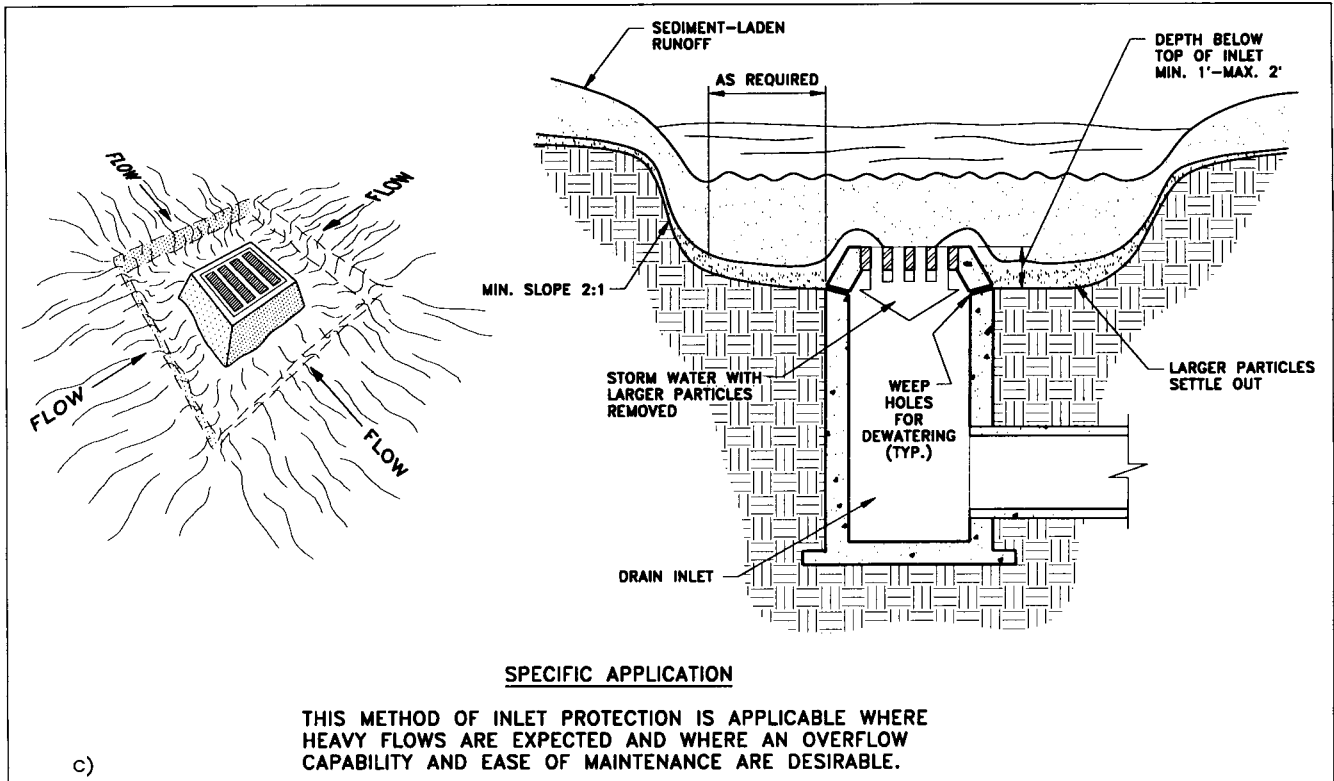
accumulated to 50 to 60% ± of the design capacity. If not maintained, sediment will eventually accumulate throughout the voids in the gravel surrounding the orifice, significantly reducing the efficiency of the sediment trap. Therefore, not only does the trap need regular maintenance to remove accumulated sediment, but the gravel needs to be removed and replaced in order to maintain the effect-

iveness of the sediment trap. Likewise, the trap should be inspected for damage, especially after large storm events.

**AESTHETICS AND SAFETY OF DISCHARGE AREAS**

**Discharging Into Stabilized Areas**

Throughout the preceding discussion on structural erosion-control devices, emphasis has been placed on directing



**FIGURE 29.25** (Continued) Sediment traps. (Source: *Maryland Standards and Specifications for Soil Erosion and Sediment Control*, 1991.)

flows to stabilized areas. Sediment-laden waters create unsightly areas as the sediment accumulates on the grass and leaves. Even though the accumulated sediment dissipates after several rain storms, such areas detract from the aesthetic value of the site and reduce its attractiveness to potential buyers. Therefore, when directing flows to stabilized vegetative areas, the area itself should be a concern. However, if the area will be used for immediate future construction, discharging sediment-laden waters onto this area presents no problem. If the area has recently been stabilized or is located at a highly visible point, the developer may not want this area used as the discharge point.

### Sediment on Pavements

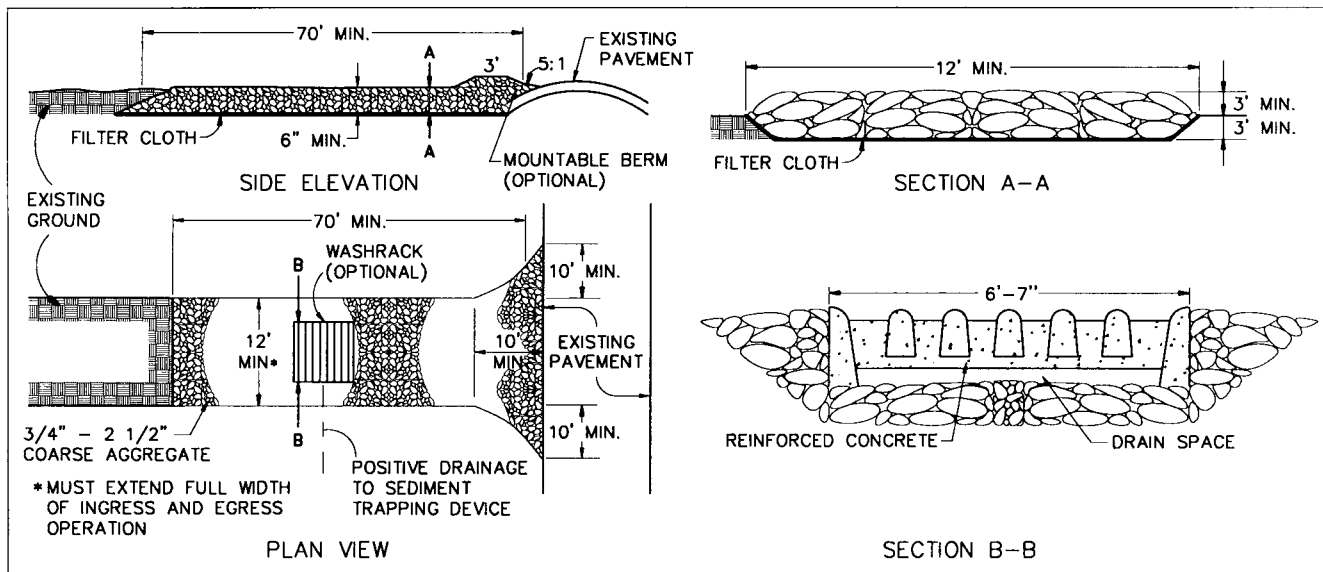
It is imperative to keep silt and sediment off of existing pavement. Wet soil is slippery and poses safety problems for moving vehicles. Basins and traps are designed to operate at 50–70% efficiency for the design storm. For those facilities that outlet onto or near pavements, the non-trapped sediments end up on the pavement. The design storm for a sediment basin or trap is typically a frequently occurring storm of short duration. As a result, the chance that a larger storm may occur, possibly blowing out the trapping facility and washing the sediments onto the pavement, should be considered. Rather than take the risk of personal injury to vehicle occupants or a major clean-up because of the washed-out facility, the engineer should locate the trapping facilities away from pavement and traffic

areas. Allowing sediment basins and traps to outlet onto streets or for sediment to collect on pavement is not recommended.

Construction traffic exiting and entering the site is another source of roadway sediment. A device that reduces the amount of sediment tracked onto the pavement is a stabilized construction entrance (Figure 29.26). This entrance consists of a bed of 2- to 3-in.-diameter stone. The stone covers an area 50 to 100 ft long and 10 to 20 ft wide. The actual size depends on the type of vehicles using it. This area is designed to collect the mud and sediment clumps accumulating on the wheels of the construction vehicles. If the site is excessively muddy, a wash rack is incorporated into the construction entrance. The wash rack is a raised section of timbers or concrete, partially buried into the stone. The wash rack, which raises the wheels of the vehicle above the stone, allows the wheels to be thoroughly rinsed before exiting the site. A small sediment trap, located on the low side of the wash rack, collects the washed mud. When the stone bed accumulates excessive mud, the stone is replaced.

### Dust Control

One problem related to erosion and sediment control, but frequently overlooked, is dust. Wind erosion is one source of dust. On construction sites, traffic, machinery, and various types of related operations (see Table 29.12) are a major source of dust. On dry days, various types of



**FIGURE 29.26** Typical construction entrance and wash rack. (Source: Virginia Erosion and Sediment Control Handbook, 1992.)

construction operations emit dust, which becomes a nuisance to surrounding property and residents. This dust also has an affect on the surrounding air quality. According to the Virginia Erosion and Sediment Control Handbook, an average value for dust emission at heavy construction sites

is 1.2 tons/acre/month. Construction operations can be terminated by the government agency (inspectors) in charge if the dust problem is not promptly addressed.

Dust control is particularly a major concern on sites with soils containing natural asbestos and natural minerals capable of generating asbestos. Geologic formations containing actinolite are sources for naturally-occurring asbestos. For example, in the mid-Atlantic states, the Orange soil group contains soils with asbestos-generating minerals.

Specific types of site-management measures can be incorporated into the construction sequencing to control dust. These types of measures include minimizing the amount of denuded land by constructing the project in phases and leaving stands of trees around and throughout the grading areas to dissipate some of the wind momentum and act as a barrier. Another measure is the suppression of the dust by mulching or spraying water or adhesive chemicals over the area. Perhaps the most popular measure is wetting the surface with water. This causes the smaller air-borne-prone particles to adhere to larger soil masses. However, the rate and frequency of application depends on climatic conditions and the traffic using the roadway. Other types of spray-on materials include synthetic and organic adhesives that, although effective on certain types of soils, are more permanent than water. Consequently, the cost is higher also. Chemical stabilization binds soil particles together to form a non-erosive surface crust (see Table 29.13).

Artificial measures that can be incorporated into a site include the various types of structures to create wind drag and act as barriers to stop the dust from leaving the site. Such measures include stabilized earth mounds, fences and walls. Recommended placement of barriers is perpendicular to prevailing air currents at intervals of approximately 15 times the barrier height (Virginia Erosion & Sediment Con-

**TABLE 29.12 Sources of Dust at Construction/Demolition Sites**

*Construction Sites*

- Pushing (land clearing and earthmoving)
- Drilling and blasting
- Batch drop operations (loader operation)
- Storage piles (soil and construction aggregates)
- Exposed areas
- Vehicle traffic on unpaved surfaces
- Mud/dirt carryout onto paved surfaces

*Demolition Sites*

- Explosive and mechanical dismemberment (blasting and wrecking ball operations)
- Pushing (dozer operation)
- Batch drop operations (loading debris into trucks)
- Storage piles (debris)
- Exposed areas
- Vehicular traffic on unpaved surfaces
- Mud/dirt/debris carryout onto paved surfaces

(Source: US EPA, Control of Fugitive Dust Sources, 1988.)

**TABLE 29.13 Adhesives Used for Dust Control**

ADHESIVE	WATER DILUTION (ADHESIVE:WATER)	TYPE OF NOZZLE	APPLICATION RATE GALLONS/ACRE
Anionic asphalt emulsion	7:1	Coarse spray	1200
Latex emulsion	12.5:1	Fine spray	235
Resin in water	4:1	Fine spray	300
Acrylic emulsion (non-traffic)	7:1	Coarse spray	450
Acrylic emulsion (traffic)	3.5:1	Coarse spray	350

(Source: *Virginia Erosion and Sediment Control Handbook*, 1992.)

control Handbook). For those structures intended to be permanent, the judicious placement and timing of construction will allow them to serve as dust-control barriers as well as site amenities or as zoning-ordinance controls. In the same manner, perimeter trees can be planted; however, only select species may be effective.

### CONSTRUCTION SEQUENCING

A familiarity with construction sequencing and techniques will aid in the development of a cost-effective erosion and sediment-control plan. It is costly for a contractor to interrupt construction operations to change the location of structural measures or to add more. Construction sequencing (i.e., the timely order in which particular items are built) depends on the contractor and his construction procedures. Interrelated with this is the site itself, ordinances, and type of project.

### Establish Perimeter Controls

Construction operations begin with the establishment of silt and erosion-control measures around the perimeter of the portion of the site to be worked. Site clearing should be limited to only that area necessary to construct the control measure. The perimeter location controls retain the sediment on site, while allowing the early grading operations to begin.

### Clearing and Grubbing

Clearing and grubbing the site involves removal of trees (that cannot be saved), stumps, brush, and other debris. Included is the removal of the topsoil layer and the stockpiling of it in an out-of-the-way location. The stockpiled topsoil is temporarily stabilized with mulch and seed. Filter-fabric fence or another suitable sediment-control measure is placed around the perimeter of the toe of the stockpiled topsoil. This initial grading operation denudes a large area, in a short time, making the area highly susceptible to erosion. The remaining (non-perimeter) first-phase erosion and

sediment-control measures are installed prior to beginning grading operations.

In larger residential developments, clearing and grubbing is performed initially in the street areas and the first few building sites, with roads occurring first. Typically, these building sites are for the model homes that will be used to market the project. In smaller single-parcel commercial areas, the site is almost totally cleared because of the high cost of land.

### Mass Grading

After clearing and grubbing, the contractor performs the mass grading of the site. Mass grading is the moving of large quantities of earth to or from the site or within the site itself. On sites where mass grading is necessary, cut and fill areas are large and may not be conveniently localized to optimize haul. Consequently, this operation is performed with larger earth-moving machinery. In many cases, extensive mass grading is necessary on a site because of poor site conditions, rather than the result of bad planning and design.

### Rough Grading

The next operation is the rough grading. Rough grading is the reshaping of the site by moving small volumes of earth, such that the site is brought to within  $\pm 0.2$  ft of finished grade. This includes preparing the site for basements and foundations, compaction, and generally grading the site to fit the proposed drainage pattern. In areas where subbase and base treatments are required (e.g., streets and foundations), the roughed grades are brought to within  $\pm 0.2$  ft of the proposed subgrade elevation. Where fill is to be placed over the proposed utilities, the area is brought to the grade slightly above ( $4 \pm$  ft to  $8 \pm$  ft) the proposed invert elevation of the utility trench. The utility is then installed and backfilled with lighter construction equipment before completing the remainder of the fill operation. This procedure protects the utilities from being crushed by the

heavier construction equipment being utilized for mass grading operations.

Rough grading should be done such that the installed erosion and sediment controls perform their functions. If the surface grades do not direct the runoff to the erosion and sediment-control facilities, new facilities should be built to accommodate the new drainage pattern.

### Utility Installation

After rough grading operations are complete, the utilities are installed. Usually the deepest utilities and/or those dependent on gravity are installed first. The main concern during this part of the sequence is the storm sewer system. As construction progresses, the site grading evolves such that the overland flow is directed to the storm sewer inlets. As the independent branches of the storm sewer system are complete and made ready to convey runoff, additional erosion and sediment controls are added around the inlets. During the transition from the surface drainage pattern to the proposed subsurface drainage pattern, some of the initial erosion and sediment-control measures are phased out as the newer ones are phased in. Some of the initial measures may not be totally abandoned until the site is completely stabilized.

To facilitate construction, the design should be laid out such that the major utilities (storm, sanitary, and water) can be constructed simultaneously. For example, in townhouse developments, the sanitary sewer branch lines are placed behind the units whenever possible. If the units are basement walkouts, the sanitary lines need only to be minimum depth. This design allows the construction of the sanitary lines in the rear, streets and storm drainage in the front, and building pads to be worked on simultaneously.

### Finished Grades

Finally, the site is brought to finished grade. Finished grades are those that are shown on the construction drawings. Final pavement layers are installed on streets and parking areas and the landscape grading is done. Areas around buildings, yards, and planting areas are brought to grade with the stockpiled topsoil and readied for seed, sod, and other landscaping amenities. Essentially, the site is permanently stabilized.

It is not necessary to have construction totally complete before final grading begins. For erosion and sediment-control purposes, the permanent forms of stabilization should be done in areas as soon as construction operations are finished.

The actual sequence of construction operations will vary depending on the site, type of development, contractor, and weather conditions. The erosion and sediment-control plan should coordinate with this construction sequencing to be sure that installation of the structural measures coordinate with the construction sequencing. The installation of erosion and sediment-control measures may precede, coincide

with, or immediately follow the operational steps just described.

### THE EROSION AND SEDIMENT-CONTROL PLAN

A well-conceived plan for a project addresses the potential erosion and sediment problems long before the final construction plans begin. The overall erosion and sediment-control plan begins at the early planning stages of the project. Such things as fitting development to the terrain, minimizing grading in sensitive areas (e.g., steep slopes, areas of highly erodible soil), and preserving existing vegetation, all factor into the erosion and sediment-control plan. The design and placement of structural-control devices is done in the latter stages of project design, after the buildings and infrastructure details have been worked out. This part of the erosion and sediment-control plan is dynamic in that the erosion and sediment-control measures change with the phasing and sequence of construction. The final design stage of the erosion and sediment-control plan is a follow-through of the overall plan devised at the beginning of the project.

The erosion and sediment-control plan sheets, included in the set of construction drawings, typically consist of plan sheets, detail sheets, and a narrative. Together, they describe and illustrate the procedures and measures that mitigate the erosion potential of the site and the off-site sediment damage. Many localities require an erosion and sediment-control plan as part of the construction drawing set. The erosion and sediment-control plan is just one part of the construction documents and, as such, is subject to review and approval by the specified public agencies. The length and complexity of the plan depends on the constraints of the site and the regulations in effect. Hence, a relatively small site, in terms of its acreage, may require extensive erosion and sediment control because of the soils, slopes, and environmentally-sensitive areas within it.

Development of an erosion and sediment-control plan requires information on:

- The erodibility of the surface and immediate subsurface soils
- Extent and type of vegetation covering the site
- Hydrologic conditions; depth of rainfall for the design storm and average intensity
- Topography; steepness of slopes, length of slopes, existing drainage pattern, contributing off-site drainage
- Capacity of outfall channels and downstream structures

Most of this information is obtained from local government agencies and other regional public agencies in the form of maps and reports. Typically, most of this information has been used earlier in the planning and design of the project and is readily available.

The overall erosion and sediment-control plan can be broken down into three categories, where each category addresses specific aspects of erosion and sediment control for the site. The three categories are (1) planning, (2) design, (3) installation and maintenance.

During the planning phase, certain issues need to be considered in the layout of the site. If these issues are adequately addressed, the measures in the later phases will be both cost-effective and successful in eliminating off-site damage.

The issues considered early in the project are as follows:

1. Fit the development to the terrain. Design and site the building structures so that they are compatible with the topography. Optimize vertical and horizontal road alignment to minimize grading activity.
2. Identify the on-site soils for their erodibility. Soil maps are available for most areas in the United States from SCS district offices. Erodibility of soils depends on the slope lengths and gradients. Long, steep slopes will have more erosion potential. Hence, locating the soil series and its erodibility on a topographic map of the site will identify critical areas. The erodibility may be given in general terms (e.g., slight or moderate), or the erodibility of the soil may be identified by the  $K$  factor of the USLE; the local SCS office can supply the  $K$  factor values. Configure the site to minimize the disturbance of the highly erodible soils. Additionally, minimize disturbance of existing steep slopes. Denuded slopes exacerbate erosion potential regardless of the erodibility of the soils on the slopes.
3. Preserve the existing vegetation. A site visit will identify the areas of good stands of trees and vegetation. Preserving existing natural vegetation not only decreases erosion potential, but also enhances the land value and conserves energy. Site the buildings and structures to take advantage of the scenic value of trees and vegetation.

Unfortunately, marketing issues, land values, and environmental preservation quite often conflict with the priorities of the developer, public officials, and citizens. It may not always be economically feasible to develop the site in a manner that best fits the interests of all parties. The priority of the previously listed issues will vary from site to site. An alternative is to complement the erosion and sediment controls through extra structural measures or regulate construction phasing. Consideration should be given to whether the cost for the extra measures is warranted. After the planning phase is complete and the planners have done everything reasonable to (1) meet the developer's program, (2) comply with local ordinances, and (3) address environmental issues, the next phase is to design the erosion and sediment-control plans. These plans show the structural

measures, related computations, and the construction phasing that reduces the erosion and sediment damage.

The placement and sizing of structural measures requires knowledge of construction techniques and construction sequencing. The engineer who is unfamiliar with construction techniques can develop a plan, but in all likelihood it will not be very effective.



















### Developing the Plan Sheets

To develop a set of erosion and sediment control drawings that are feasible and cost-effective, the designer needs to address the specific issues previously discussed. Each site is unique, and the priority or sequence of the issues is not always the same. To begin, the engineer obtains copies of the plan sheets that show the location of the buildings, streets, and utilities with the existing and proposed grades. All other pertinent features should be included on the plan sheets (e.g., streams, lakes, and other environmentally-sensitive areas, adjacent properties). These will be used as his worksheets to develop the erosion and sediment-control plan sheets. Depending on the project, the designer may decide to use a small-scale plan view of the entire project rather than the three or four sheets that comprise the large-scale plan sheets. One objective of the plan is to communicate clearly the erosion and sediment-control design to the approving public agencies and the developer. This includes delineating the appropriate erosion and sediment-control measure and its location on the site. Table 29.14 suggests the symbols used on plans and summarizes the protection afforded by the particular measure.

First the engineer considers whether any specific phasing is necessary. Features such as streams and lakes or activities such as construction on adjacent property will dictate a specific on-site phasing. Since some projects require model houses to be constructed first to promote marketing and sales within the development, phasing of the erosion and sediment plan should be considered. The engineer needs to consider such issues at the outset and weigh their impact on the project's development. The engineer or developer may require certain areas to be started before others because of specific site constraints or local conditions.

After establishing the need for project phasing, the limits of clearing and grading that apply to the phasing are addressed. Typically, the limits of clearing and grading are the point where the proposed contours tie into the existing contours. If any intermediate land-disturbing activities are planned, the engineer may decide to show the clearing and grading limits separately for each activity. For example, if the entire site can be constructed (e.g., utilities, pavement, and building) within the prescribed maximum-cleared area, only perimeter controls may be needed. If a site requires mass grading and borrow areas, intermittent controls will be necessary around the borrow areas as well as the immediate site area. Alternatively, the intermediate activities can be described or clarified in the narrative.

**TABLE 29.14 Summary of Symbols and Types of Selected Erosions and Sediment-control Measures**

TITLE	KEY	SYMBOL	PERIMETER CONTROL	SLOPE PROTECTION	SEDIMENT TRAPPING	DRAINAGEWAY AND STREAM PROTECTION	TEMPORARY STABILIZATION	PERMANENT STABILIZATION
			A	B	C	D	E	F
Temporary gravel construction entrance	(CE)		✓				✓	
Straw bale barrier	(STB)		✓		✓	✓		
Silt fence	(SF)		✓		✓	✓		
Storm drain inlet protection	(IP)				✓			
Temporary diversion dike	(DD)		✓	✓				
Temporary sediment trap	(ST)		✓		✓			
Temporary sediment basin	(SB)		✓		✓			
Temporary slope drain	(TSD)			✓			✓	
Paved flume	(PF)			✓				✓
Outlet protection	(OP)					✓		✓
Check dams	(CD)					✓	✓	
Level spreader	(LS)		✓			✓	✓	
Temporary seeding	(TS)						✓	
Permanent seeding	(PS)							✓
Sodding	(SO)					✓		✓
Mulching	(MU)						✓	
Tree preservation and protection	(TP)							✓
Dust control	(DC)						✓	

(Source: Virginia Erosion and Sediment Control Handbook, 1992.)

Clearing limits for trenching of utilities often extends beyond the width of the easement. The clearing limits for these activities will depend on how deep the trenches are and whether the contractor will use shoring to stabilize the sidewalls. Sloping or benching the sidewalls of the trenches, which is an alternative to shoring, requires extending the clearing and grading limits beyond where the proposed and

existing contours meet. If the trench walls will be sloped, typically 1H:1V, the clearing limits will be a distance equal to the depth of the trench on either side of the center line of the excavated trench. Note: extra room is necessary on one side of the trench for placing the excavated soil.

The selection and placement of erosion and sediment controls is related to the existing drainage pattern of the

site as well as the intermediate drainage patterns during construction and land phasing.

The engineer delineates the drainage divides for the site's existing and proposed conditions. The drainage divides will reveal off-site sources contributing surface runoff, drainage areas to existing and proposed storm drain inlets, and drainage areas to all outlet points discharging concentrated runoff outside the denuded area. After establishing the existing and proposed drainage divides, the engineer quantifies the runoff for each drainage area. For small drainage areas (<50 acres), the rational method provides adequate peak-runoff estimates. The engineer also needs to quantify the water discharged onto the site from swales, ditches, and pipes conveying water from off-site sources. On most projects, it is likely that the drainage areas and calculations of the runoff have been done during an earlier stage in the design process. The design and installation of structural erosion and sediment-control measures may be considered a two-phase process, described next.

**Phase I:** The initial clearing and grubbing operation denudes a large area. This is the most critical stage for control of erosion and sedimentation. Before denuding a large area, some erosion and sediment controls are installed. These controls are located typically along the perimeter and at the outfall areas of the site. Since some sites are not located along streams, outfall means anywhere where concentrated water will be discharged from the site. Placing these initial controls along the perimeter allows the builder to begin clearing and grading operations with minimal obstructions. The construction operations should be able to continue with little interruption from the relocation or construction of additional controls. Along the perimeter on the up-slope sides, berms are built to divert the runoff away from denuded areas. Along the low areas, berms and trapping facilities are constructed to divert and detain the sediment-laden runoff. If the drainage pattern of the denuded area supplies low amounts of sheet flow at 1 cfs or less, silt fence is used to filter the runoff sediments. If the denuded area is large, major sediment basins may be needed to protect the outfall points. These sediment basins are kept in place until the storm sewer system is functioning and the upstream area is stabilized.

**Phase II:** As the construction operations progress, the drainage pattern evolves from the natural one (existing prior to construction) to the ultimate pattern, as shown on the construction plans. This evolution creates smaller drainage areas from the existing larger drainage areas. During the course of the evolution, modification to erosion and sediment controls is necessary to reflect the changes in the drainage pattern. This includes eliminating the unnecessary controls and establishing new controls where appropriate. As the branches of the permanent drainage channels and subsurface storm drain system become operable, small sediment traps are installed around the storm sewer inlets. These small sediment traps reduce the sediment load to the sediment basin by collecting the runoff from the smaller

drainage areas. Eventually, the temporary sediment basins are eliminated and the area around the sediment basin is worked to match the grading shown on the construction plans.

As construction of the site progresses to the point where the storm sewer system becomes operable, another phase of controls is installed. Diversion berms need to be relocated to account for the changing topography. Sediment traps and berms that are no longer functional are removed. The storm drain inlets are modified to function as sediment traps. Gravel-inlet protection is installed and temporary sediment basins are removed. As areas are stabilized with vegetation and mulch, the erosion and sediment-control devices are removed and the area is reworked and graded to match the construction plans. Removal of control devices is usually contingent on the approval of a site inspector representing the engineer or local public agency.

### The Narrative

The narrative to the erosion and sediment-control plan is a list of notes that describe pertinent site features relating to erosion and sedimentation. It describes the expected erosion and sedimentation problems, identifies the problem areas, explains any phasing of land-disturbing activities, clarifies any erosion and sediment-control sequencing, documents the types of measures used and the maintenance schedule, as well as the inspection and provisions for repair of the measures.

The purpose of the narrative is to summarize the plan for the reviewer in the public agency, the site inspectors, and the contractor. In preparing the narrative, the engineer assumes that the reviewing agency personnel are not familiar with the site and provides sufficient detail for their benefit. The narrative needs to be short and concise, yet still supply the information needed.

Although the narrative may not be required in all jurisdictions that require erosion and sediment-control plans, it is recommended that the narrative be included as part of the erosion and sediment-control plan. Since some sites require complex arrangements of erosion and sedimentation controls, notes that explain the problem areas of the site, that specifically explain the phasing, and that clarify any confusing situations shown on the plan view, are helpful.

The narrative should include information regarding the following topics:

1. Project description: type of project, area that will be disturbed, number of units (residential) or square feet (single-parcel commercial/industrial sites)
2. Description of existing site conditions: topography, vegetation, streams, lakes, drainage features
3. Description of bounding areas that might be affected by land-disturbing activities: streams, lakes, roads, residential and commercial areas



**General land conservation notes**

1. All land conservation activities shall be performed in accordance with the latest Fairfax County checklist for erosion and sediment control, the public facilities manual, and the Virginia Erosion and Sediment Control Handbook.
2. No disturbed area will remain denuded for more than 14 calendar days unless otherwise authorized by the director or his agent.
3. All erosion and sediment control measures are to be placed prior to or as the first step in grading. First areas to be cleared are to be those required for the perimeter controls.
4. All storm and sanitary sewer lines not in streets are to be mulched and seeded within 5 days after backfill. No more than 500 feet are to be open at one time.
5. Electric power, telephone and gas supply trenches are to be compacted, seeded and mulched within 5 days after backfill.
6. All temporary earth berms, diversions and sediment control dams are to be mulched and seeded for temporary vegetative cover immediately after grading. Straw or hay mulch is required. The same applies to all soil stockpiles.
7. During construction, all storm sewer inlets will be protected by inlet protection devices, maintained and modified as required by construction progress.
8. Any disturbed area not covered by note #1 above and not paved, sodded or built upon by November 1, or disturbed after that date, is to be mulched with hay or straw mulch at the rate of two tons per acre and over-seeded immediately.
9. At the completion of construction projects and prior to the release of the bond, all temporary siltation and erosion controls shall be removed and all disturbed areas shall be stabilized.
10. No area shall be left denuded for a period longer than 14 days except for that portion of the site in which work will be continuous beyond 14 days. In the event such maximum period is exceeded and any such areas remain exposed without cover, the county will (in the event the developer or builder does not) install the necessary temporary or permanent vegetative stabilization measures to achieve adequate erosion and sediment control.
11. The cost of any such temporary measures taken by the county shall be borne by the developer and shall be a charge against the conservation deposit.

**Project description**

The purpose of this project is to provide adequate ingress and egress from the property, allowing it to be fully developed into a residential subdivision under the current zoning. The clearing and grading area required for this development is approximately 9.42 acres.

**Existing site conditions**

The site is primarily wooded with some open areas found near the existing Vernon drive cul-de-sac. The site is divided into two distinct drainage areas split by a ridgeline that runs northwest to southeast through the site. The southern side of the site drains by sheet flow into an existing adequate swale. This swale connects downstream to an unnamed tributary of Difficult Run, which drains through the Wolf Trap Green subdivision. The remainder of the site drains by sheet flow and shallow concentrated flow to an existing unnamed tributary of Difficult Run which crosses the northeast corner of the site. 100% of the onsite runoff enters these two swales. The majority of the onsite runoff (approximately 70%) flows to the northeastern most swale. These swales ultimately convey the on-site storm runoff to Difficult Run.

**Soils**

Soils were mapped by Fairfax County soil survey office February 10, 1982 with the following results: mixed alluvial land that is basically contained within the EQC line at the northeastern side of the property. Connection to an existing sanitary sewer and outfall of the storm sewer is proposed within the area of this soil. Meadowville silt loam at 3% slope, Manor silt loam at 6% in the rolling phase, and Glenelg silt loam at 4% in the rolling phase. The meadowville soil type is characterized as having wetness problems and therefore creating possible foundation problems. Adequate foundation drainage as well as inspection of foundation subgrades should be done on construction of these areas. Glenelg silt loam and Minor silt loam, however, are characterized as having both good subsurface drainage and foundation support but also a severe potential for erosion.

**FIGURE 29.27** Erosion and sediment-control narrative.

**PHASE 1: (Initial clearing and grading)**

## Erosion and Sediment Control Measures

- A. A construction entrance with a wash rack shall be installed) if water is not readily available, a water tank shall be the source of water). A small temporary silt trap will catch wash rack water.
- B. Silt fences and tree protection shall be installed as shown in this plan.
- C. Two sediment basins shall be installed as shown on the plan. See sediment basin calculations on sheet 16 and details on sheet 11. The sediment basins shall be inspected on a regular basis and restored to their original capacity when sediment has accumulated to the clean out elevation. (See sediment basin computations.)
- D. A temporary diversion dike will be installed near the limit of grading as shown on the plan and directed towards the sediment basins.

**Maintenance program**

- 1. The site superintendent, or his/her representative, shall make visual inspection of all mechanical controls and newly stabilized areas (i.e. Seeded and mulched and/or sodded areas) on a daily basis; especially after a heavy rainfall event to insure that all controls are maintained and properly functioning. Any damaged controls shall be repaired prior to the end of the work day including re-seeding and mulching or re-sodding if necessary.
- 2. All sediment trapping devices shall be cleaned out at 50% trap capacity and the sediment shall be disposed of by spreading on the site or hauling away if not suitable for fill.

**Drainage Area A:**

The majority of the runoff contributing to this area is from the proposed Vernon Drive extension. A very small portion of the runoff comes from offsite as shown in the plan. Flows are picked up by structure 5. Flows are then conveyed to bed and banks on parcel "A" through the underground storm pipes.

**Drainage Area B:**

This area is comprised of portions of lots 3 through 8. Runoff is picked up at the low point in the cul-de-sac by structure 4. Flows are then conveyed to bed and banks on parcel "A" through the underground storm pipes.

**Drainage Area C:**

This area is comprised of portions of lots 8 and 9. Runoff is picked up by structure 6. Flows are then conveyed to bed and banks on parcel "A" through the underground storm pipes.

**Drainage Area D:**

This area is comprised of lots 9, 10 and 11. Runoff is picked up by structure 7. Flows are then conveyed to bed and banks on parcel "A" through the underground storm pipes.

**Drainage Area E:**

This area is comprised of portions of lots 15 and 16, and portions of offsite Kenmore Farms Lot "7." Runoff is picked up by structure 12. Flows are then conveyed to bed and banks on parcel "G" of adjacent Wolf Trap Green, Section Two by means of underground storm pipes and a sodded ditch.

**Drainage Area F:**

This area is comprised of lots 11 through 14. Runoff is picked up at the low point in the cul-de-sac by structure 11. Flows are then conveyed to bed and banks on parcel "G" of adjacent Wolf Trap Green, Section Two by means of underground storm pipes and a sodded ditch.

**Drainage Area G:**

This area is comprised of portions of lots 14 and 15, and offsite lots 4 and 7 of Kenmore Farms. Runoff is picked up at a low point by structure 13. Flows are then conveyed to bed and banks on parcel "G" of adjacent Wolf Trap Green, Section Two by means of the underground storm sewer pipes and a sodded ditch.

**Drainage Area H:**

This area is comprised of portions of lots 12 through 14, parcel "B," and offsite lot 4, Kenmore Farms and Route 7. This area also receives flows from drainage areas E, F and G through the underground storm pipes. Flows are conveyed by bed and banks on Parcel G of adjacent Wolf Trap Green, Section Two by means of a sodded ditch.

**FIGURE 29.27** (Continued)

**Drainage Area I:**

This area is comprised of a portion of lot 9 and the existing Myra drive turn-around. Runoff is picked up at the low point of the Myra Drive turn-around by structure 8. Flows are then conveyed to bed and banks on parcel A through the underground storm pipes.

**PHASE 2: (Final clearing and grading)****Erosion and Sediment Control Measures**

- A. After all sediment and erosion control measures have been installed in accordance with the phase I plan, and upon approval of the county inspector, the contractor shall clear only the areas needed for street and utility construction to minimize erosion. As soon as grading allows streets will be stabilized with base stone and the remainder of the site shall be cleared when necessary. Phase I diversion dikes and basins shall be left in place for as long as possible and gradually removed after street areas are stabilized.
- B. Inlet protection shall be placed on storm inlets as soon as they are installed. Outlet protection shall be placed as soon as outlet structures are in place.
- C. For additional information please see land conservation notes.

**Maintenance Program**

- 1 The site superintendent, or his/her representative, shall make visual inspection of all mechanical controls and newly stabilized area (i.e. Seeded and mulched and/or sodded areas) on a daily basis; especially after a heavy rainfall event to insure that all controls are maintained and properly functioning. Any damaged controls shall be repaired prior to re-sodding if necessary.
- 2 All sediment trapping devices shall be cleaned out at 50% trap capacity and the sediment shall be disposed of by spreading on the site or hauling away if not suitable for fill.

**FIGURE 29.27** (Continued)

- 4. Description of soils: soil series and their erodibility
- 5. Critical areas such as steep slopes and environmentally-sensitive areas
- 6. Erosion and sediment-control measures to be used (e.g., total length of berms, number and size of basins, and general location); when construction phasing is necessary, a description of phasing sequence and amount of area to be denuded is essential
- 7. Permanent stabilization: methods used to stabilize the site permanently (e.g., seed, sod)
- 8. Maintenance: identify a schedule for inspections, clean out, and repair of the structures  
Occasionally, the phrase "as needed" is used as an all-encompassing blanket phrase to describe when repairs and clean out should be performed, but the phrase should be avoided. Technically, the term is vague: "As needed" by whom? The public agency, contractor, engineer, etc., will have different definitions for "as needed." Therefore, the description on inspections and repair should be explicitly specified (e.g., "... every 2 weeks", or "... after any rainstorm greater than 0.25 in.").
- 9. Calculations and assumptions: Design of sediment basins, their efficiency, etc., requires detailed calculations. Along with the calculations, the engineer will

have to make assumptions when data is not available. The calculations should include a brief heading describing the calculation step followed by any assumptions used to facilitate that particular step. Most calculations and assumptions are shown on the erosion and sediment-control plan sheets where they describe the particular facility.

An example of a narrative is shown in Figure 29.27, and the corresponding erosion and sediment-control plan (Phase I) is shown in Figure 29.28. Once the storm drain conveyance structures are operable, the erosion and sediment-control plan is amended to correspond to the new drainage pattern. The control devices remain until the contributing area is stabilized. A Phase-II type of erosion and sediment-control plan is shown in Figure 29.29

**SUMMARY**

Grading operations change surface features at a site constantly. The altering of the land is quite rapid during the first half of the project. The methods and sequencing for altering the surface features would be done differently by different contractors for the same site. Therefore, it is difficult for an engineer to propose an erosion and sediment-control plan that would be both practical and cost-effective for the total duration of the project for all changes in surface features. The best that can be done is to address the major



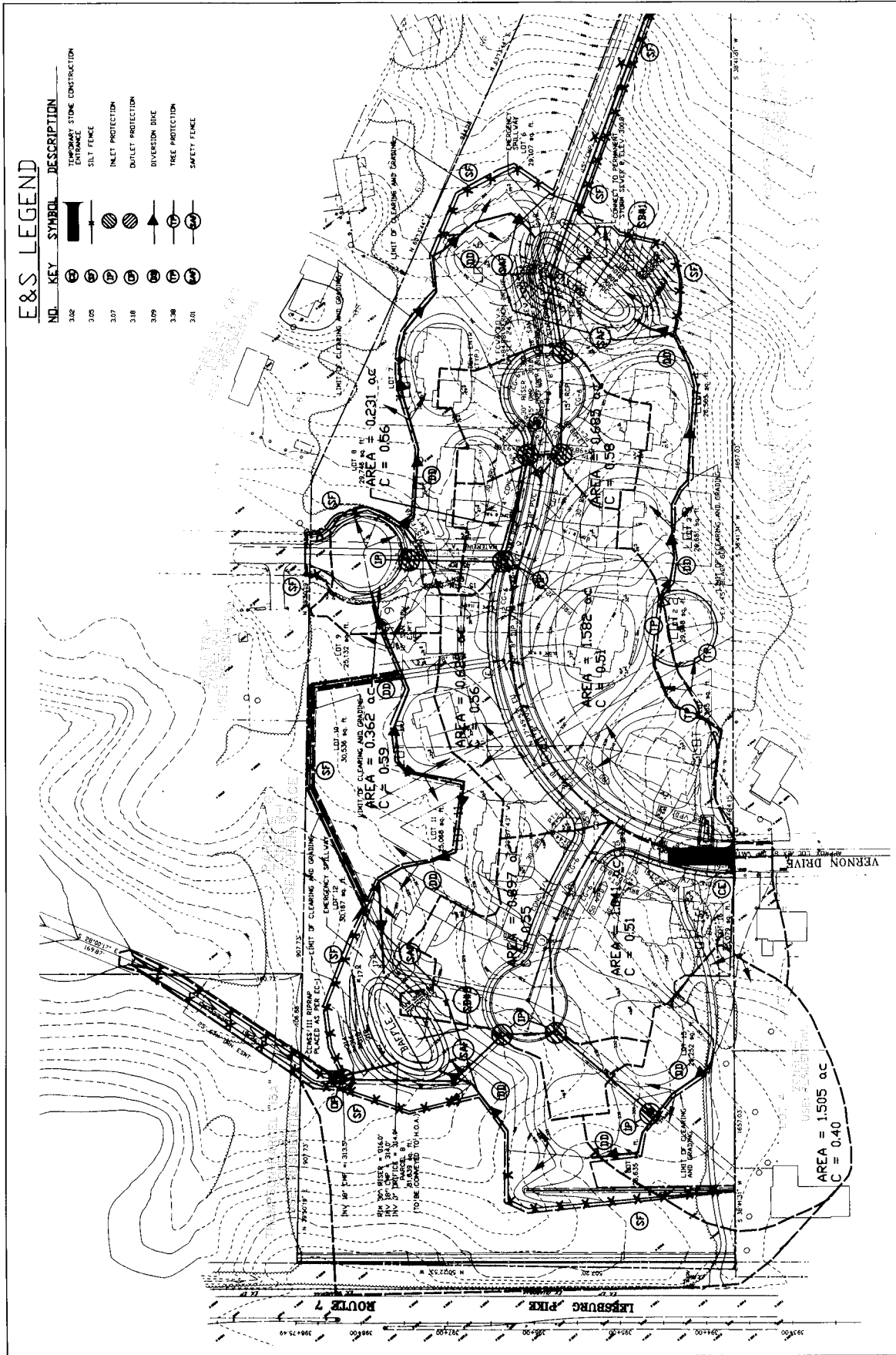
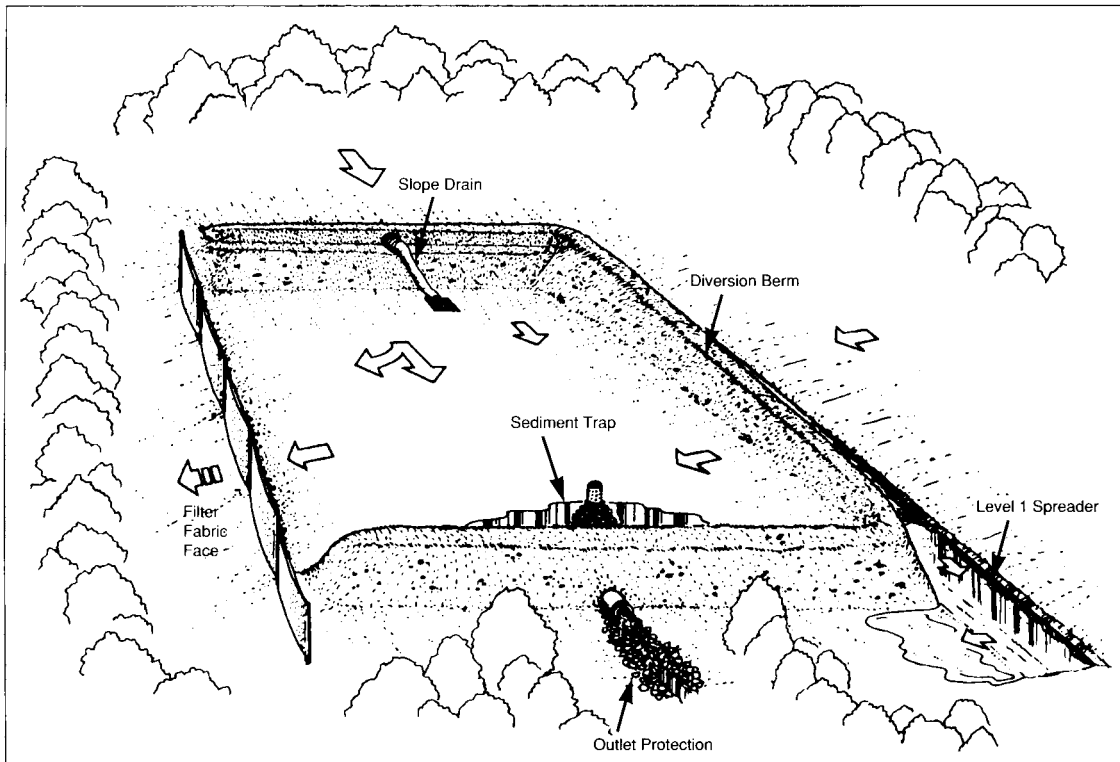


FIGURE 29.29 Phase II erosion and sediment-control plan.



**FIGURE 29.30** Schematic for erosion and sediment controls.

changes. The daily intermittent changes will have to be addressed by the field engineer or site superintendent. Provisions for liability of the intermittent changes can be explained in the narrative as part of the maintenance and inspection paragraph.

The fundamental concepts common to erosion and sediment control plans are:

1. Clear only areas that will be worked on immediately. Mass site-grading should be avoided.
2. Stabilize all disturbed areas as soon as construction is complete or when the areas will not be worked on for extended periods of time (e.g., >2 weeks).
3. Direct non-sediment-laden water away from disturbed areas and divert sediment-laden water to trapping/filtering controls.

The erosion and sediment-control plan is an integral part of the overall scheme of the development program and should not be created as an afterthought just because regulations require it (see Figure 29.30). Often, designers add erosion and sediment controls to the plans after the plan is nearly complete without any thought to the practicality of the controls. Any erosion and sediment-control measure called for in the plan should be able to be implemented in the field at the time of its need. It is easy to put symbols for silt basins, silt fence, berms, etc. on paper, but the engineer has to ask if these things can be built and will they

function as intended during that particular phase of construction. The engineer should think through the construction process with respect to the controls shown on the plan and clarify ambiguous or controversial situations in the narrative.

Additionally, as a result of the National Pollutant Discharge Elimination System (NPDES) permit program, construction sites will be required to prepare a Storm Water Pollution Prevention Plan (SWPP) prior to commencement of any land-disturbing activities. Phase I of the program was promulgated in November 1990 and covered construction sites greater than 5 acres in size. Phase II of the program, initiated in November 1999, requires operators of construction sites from 1 to 5 acres to obtain a permit. The goal of the SWPP is to maximize the potential benefits of the pollution prevention and erosion and sediment-control practices through the use of best management practices during the construction process. Since the primary focus of this program is on controlling pollutants in any stormwater discharge, incorporation of well-thought-out and carefully implemented erosion and sediment-control measures is more important than ever.

The descriptions of the specific structural measures herein have been limited to those that are most frequently used in an erosion and sediment-control plan. Other measures exist, and the reader is referred to local-standards manuals for measures needed to accommodate the nuances for a specific locality or site condition.

## REFERENCES

- American Society of Civil Engineers, *Residential Erosion and Sediment Control*, American Society of Civil Engineers, New York, 1978.
- Becker, B.C., and T.R. Mills, *Guidelines for Erosion and Sediment Control Planning and Implementation*, U.S. EPA, Washington, D.C., 1972.
- Boysen, Stephen M., *Predicting Sediment Yield in Urban Areas*, Proc. National Symposium on Urban Rainfall and Runoff and Sediment Control, Lexington, Ky., 1974.
- Boysen, Stephen M., *Erosion and Sediment Control in Urbanizing Areas*, Proc. National Symposium on Soil Erosion and Sedimentation by Water, Chicago, Ill., 1977.
- Cowherd, C. and J.S. Kinsey, *Control of Open Fugitive Dust Sources*, Research Triangle Park, N.C. Office of Air Planning and Standards, U.S. Environmental Protection Agency, Research Triangle Park, N.C., 1988.
- Goldman, Steven J., Katherine Jackson and Taras A. Burtsztny, *Erosion and Sediment Control Handbook*, McGraw-Hill, New York, 1986.
- Guy, H.P., *Sediment Problems in Urban Areas*, USGS Circular 601-E, U.S. Government Printing Office, Washington, D.C., 1970.
- Haan, C.T., and B.J. Barfield, *Hydrology and Sedimentology of Surface-Mined Lands*, University of Kentucky, Lexington, Ky., 1978.
- Holy, Milos, *Erosion and Environment*, Pergamon Press, Oxford, U.K., 1980.
- Kanerva, R.A. and M.A. Ports, *Urban Sediment Controls: The Maryland Experience*, Proc. National Conference on Soil Erosion, West Lafayette, Ind., 1976.
- Kirkby, M.J. and R.P.C. Morgan, eds., *Soil Erosion*, Chichester: John Wiley, Chichester, U.K., 1980.
- Lafren, John, L.J. Lane and G.R. Foster, "WEPP—A New Generation of Erosion Prediction Technology," *Journal of Soil and Water Conservation*, Vol 4, Jan–Feb, Ankeny, Iowa, 1991.
- Law, Dennis L., *Mined Land Rehabilitation*, Van Nostrand Reinhold, New York, 1984.
- Lord, Byron N., Effectiveness of Erosion Control. In Ben Urbanas and Larry A. Roesner, editors: *Urban Runoff Quality Proceedings of an Engineering Foundation Conference*, American Society of Civil Engineers, New York, 1986.
- Maryland Water Resources Administration, Soil Conservation Service and State Soil Conservation Committee, *Standards and Specifications for Soil Erosion and Sediment Control*, Annapolis, Maryland, 1983.
- Renard, K.G., G.R. Foster, G.A. Weesies, and J.P. Porter, "RUSLE—Revised Universal Soil Loss Equation," *Journal of Soil and Water Conservation*, Vol 46, Jan–Feb, Ankeny, Iowa, 1991.
- Renard, K.G., G.R. Foster, D.C. Yoder, and D.K. McCool, "RUSLE—Revisited: Status, Questions, Answers, and the Future," *Journal of Soil and Water Conservation*, Vol 46, May–June, Ankeny, Iowa, 1994.
- Schwab, G.O., R.K. Frevert, et al., *Soil and Water Conservation Engineering*, 3rd ed., John Wiley, New York, 1981.
- Tchobanoglous, G. and E.D. Schroeder, *Water Quality*, Addison-Wesley, Reading, Mass., 1985.
- U.S. Department of Agriculture, SCS, *Sediment Storage Requirements for Reservoirs*, Technical Report 12, U.S. Government Printing Office, Washington, D.C., 1968.
- U.S. Department of Agriculture, SCS, *National Handbook of Conservation Practices*, U.S. Government Printing Office, Washington, D.C., 1977.
- U.S. Transportation Research Board, *Design of Sedimentation Basins*, National Research Council, Washington, D.C., 1980.
- Virginia Department of Conservation and Historic Resources, *Virginia Erosion and Sediment Control Handbook*, 3rd ed., Richmond, Va, 1992.
- Walesh, Stuart G., *Urban Surface Water Management*, John Wiley, New York, 1989.
- Wischmeier, W.H., "Use and Misuse of the Universal Soil Loss Equation," *Journal of Soil and Water Conservation* 31:5–9, 1976.
- Vanoni, V.A., ed., *Sedimentation Engineering*, New York: American Society of Civil Engineers, New York, 1975.
- Vice, R.B., H.P. Guy and G.E. Ferguson, *Sediment Movement in an Area Of Suburban Highway Construction*, Scott Run Basin, Fairfax County, Va., 1961–64, USGS Water Supply Paper 1591-E, U.S. Government Printing Office, Washington, D.C., 19??.

## CHAPTER 30

# CONTRACT DOCUMENTS AND SPECIFICATIONS

*John Matusik, P.E.*

### INTRODUCTION

Contract documents, drawings and specifications are the major tools of communication among the numerous participants involved in the land development design process. These tools are used throughout the design process to convey ideas and formulate the project's design. The type of map or drawing, level of detail and technical information included depends on the targeted audience. Technical drawings and plans are used by professionals for analysis and construction. Non-technical drawings, such as colored or shaded pictorial drawings, are used to convey project concepts and show the finished product to the general public. It is important for the design team to compose the map or drawing to the level of detail appropriate for the audience. A careful selection of the type of map or drawing and appropriately selected level of detail for the audience are helpful in winning project approval.

Contract drawings differ significantly from design drawings in that they specify the owner's exact wishes as to how construction is to take place. Contract documents and specifications serve to ensure that both the contractor and owner are in agreement about these construction standards. Such drawings and documentation work on a multitude of levels. Documentation may include bidding documents, contract documents, and, in some cases, writing specifications. This documentation can delineate the requirements, standards and procedures for project construction. They can act as a legal agreement between contractor and owner, formalizing that both parties are in concordance with project construction requirements. These documents are informative as well. Such specifications indicate construction methods and the equipment and materials that will be

needed to complete the project. If any unforeseen problems should arise, this documentation can be used to determine which party is responsible for rectifying the situation. These documents and drawings are used to ensure that the project requirements are fulfilled correctly, in a timely fashion and in a manner that is equitable to both owner and contractor.

### CONTRACT DOCUMENTS

The entire arrangement between the owner and the contractor detailing bidding, bonds, liabilities, construction services, legal and financial rights and responsibilities, quality of workmanship, and numerous other items is packaged into the set of contract documents, which includes the construction drawings *and* specifications. The contract documents constitute a legally binding agreement that the contractor and owner each agree to perform according to the specified terms. The design professional is responsible for the information in these two items.

### CONTRACT DRAWINGS

Those drawings prepared by a design professional (e.g., architect, engineer) for bidding purposes are referred to as design drawings. When they are sealed by the design professional and approved by public agencies, they may be referred to as construction drawings. Finally, when the contract is let, the drawings are then referred to as contract drawings. However, in practice it is not uncommon to use the terms design drawings and construction drawings interchangeably.

The purpose of the design drawings is, in part, to show the contractor graphically what and where to build. The drawings also show that the design complies with codes



and ordinances. Even though the design drawings convey more information pictorially than what could be detailed through written descriptions, some aspects of the design and construction are better served by written descriptions.

## SPECIFICATIONS

The entire arrangement between the owner and the contractor detailing bidding, bonds, liabilities, construction services, legal and financial rights and responsibilities, quality of workmanship and numerous other items is packaged into the set of contract documents. Included in the contract documents are the construction drawings and specifications. When the construction contract is signed, these drawings become part of the contract documents. A drawing does not necessarily have to be drawn to scale or be a neat, organized drawing to be included as part of the contract documents. A freehand sketch on scratch paper can be just as legally binding as any drawing prepared by a professional and skilled technician. The contract documents constitute a legally binding agreement, which the contractor and owner each agree to perform according to the specified terms. The design professional is responsible for the information in these two items.

The specifications of the project have a twofold purpose. One part of the specifications detail in writing the performance of the work and the interrelationships of the contractor, owner and engineer. In addition, the specifications are the instructions affecting materials, equipment, standards and workmanship. They essentially spell out those items relating to the construction that cannot adequately be shown pictorially. The specifications are considered an extension of the information on the drawings and, accordingly, do not reiterate information already included in the drawings. However, they *must* agree exactly in reference to specific items. For example, the water main labeled on the drawings cannot be called the water-supply line in the specifications. Every occurrence of the same item of the work and specific reference to it should be identified by the same words both in the drawings and specifications. Any ambiguity and inconsistency between the drawings and specifications leads to construction delays, increased costs, and possibly even lawsuits. When rendering legal decisions the courts have, in most cases, used the specifications for interpreting intent, unless the specifications explicitly state that the drawings take precedence over the specifications.

Specifications directly affect the cost and quality of construction. Therefore, they need only define the minimum requirements of quality of material and workmanship, prescribe tests to measure and identify these minimum requirements, and establish methods of measurement and payment. In order to attain this level of specificity, the specifications must also be coordinated with the designated materials and tests as well as the writing and symbols used on the drawings.

In general, the specifications are divided into three major categories that appear in the following sequence: bidding documents, general conditions, and the technical section.

## Bidding Documents

The bidding documents detail the procedure and requirements to the contractors interested in submitting a proposal to the project. The owner can evaluate the proposals equally if they are all prepared under the same guidelines and rules. Not only must prospective contractors have the technical capabilities to perform the work, they must also have the financial capabilities. Therefore, the contractor may be required to post surety bonds as part of the process for procuring the contract. On public works contracts, the job is awarded to the lowest responsible bidder, and that bidder is the one who is free from defaults, can post the surety bond, and has not been proven dishonest on past contracts. On private projects, the contract can be awarded to anyone the owner chooses. The project can be open to bidding or the owner can negotiate the contract with his favorite contractor. There is no restriction on private construction projects.

Many public agencies have design standards and corresponding specifications that the design professional is expected to incorporate into the design as part of the contract documents. For example, state highway departments have their own set of design standards and specifications. The dimensions and locations of such items as width of pavement, depth of pavement, curb and gutter and many other items are detailed in the design standards. When these criteria are used, the corresponding specifications detail the type of material used or the minimum expected performance of the material. This allows for a uniform consistency in construction over a wide area and spares the engineer from writing specifications for every project. Such standard specifications expedite design and construction techniques after repeated use because of familiarity. In addition, the language and references between the two are, for the most part, consistent, clear and unambiguous. Any inconsistencies between the two can usually be clarified with a phone call to the appropriate agency.

## General Conditions

The general conditions, also sometimes called the general provisions, address the procedures for dealing with situations and problems that may occur with the contractor. This section addresses the legal aspects of the arrangement between the contractor and owner. The general conditions detail the rights and responsibilities of those involved in the contract, provide guidelines for administering the contract, define the legal relationship, and define the relationship among the parties affected by the contract (e.g., subcontractors, the engineer, construction manager, etc.). Sections within the general provisions include: definitions and abbreviations, scope of work, contract and subcontract pro-

cedure, control of the work, control of materials, legal and public relations, prosecution and progress. These are then further subdivided into more specific topics. The specific topics depend on the nature of the project and are not always the same from one set of specifications to another.

### Technical Section

The technical section relates directly to the information on the construction drawings. Typically, the technical section is arranged in the order of the way the work will be executed. As an example, a small commercial site could be broken down into the following items: earthwork, sanitary sewer, storm sewer, water line, pavement, foundation, building structure and landscaping. Within each item there are five topics:

1. *Description*—a short description of the nature and extent of work and its pertinent features covered in the particular item.
2. *Materials*—description of the materials that can be used for the particular item. This includes the physical and chemical characteristics of the material as well as any requirements for tolerances, mix proportions, tests, certification, fabrication, shop painting, shipping and storage of the material. Industry standards for testing or performance are also referenced here (e.g., American Society for Testing and Materials (ASTM), American National Standards Institute (ANSI) among others).
3. *Construction requirements*—prescribes the desired end results related to workmanship and finish. This allows the contractor flexibility in how to accomplish these end results. Also covered in this topic are such items as disposition of removed materials, allowed tolerances in the end results, inspections, and tests to assure the end results.
4. *Method of measurement*—describes how each item is measured for payment purposes (e.g., linear foot, cubic yard, each (complete units), ton etc.).
5. *Basis of payment*—specifically describes those costs that are paid for under the contract price as defined by the method of measurement. Frequently, items 4 and 5 are combined into one topic. It is imperative to define each item, its method of measurement and all incidental items affecting payment.

### WRITING SPECIFICATIONS

When the situation exists where no standard specification applies, the design professional must write an applicable specification. Most land development projects use standards and specifications already established by public agencies or professional organizations. It is rare for the design engineer to compose the complete specifications for the entire pro-

ject. For this rare situation, it might be prudent to seek qualified experts rather than accept the liability. However, the land development engineer may need to specify certain materials or procedures pertaining to site-specific situations. Following are general guidelines and thoughts to consider when composing any specification.

Writing technical specifications requires a fairly complete knowledge of construction techniques, the behavior of material or items to be specified and the testing techniques if applicable. The objective of the specifications is to “give detailed information regarding each and every thing and operation within the scope of the contract, to serve as a single and definite basis for competitive bidding, and to constitute a book of instructions for all concerned with the work. Some basic requirements for proper specifications are: technical accuracy and adequacy, definite and clear stipulations, fair and equitable requirements, and careful preparation to the end that they will be legally enforceable.”<sup>1</sup>

Important considerations in composing specifications include:

- Be specific—not indefinite or vague and by all means avoid ambiguity
- Be brief—state only the necessary facts; avoid unnecessary words, phrases and avoid repetition
- Use correct grammar and short sentences
- Specify standard sizes and patterns when possible
- Do not specify both methods and results
- Do not specify requirements that conflict with each other
- Do not justify a requirement
- When inserting a national standard (e.g., ASTM), read the standard to be sure that it does not conflict with job requirements

Another reason for written specifications is to provide a permanent record, primarily to avoid or resolve disputes. If care is taken to write clear unambiguous specifications the resolution of disputes is facilitated.

Special organizations and agencies exist for the purpose of standardizing testing procedures and establishing minimum performance criteria. Examples of such organizations are the American Society for Testing and Materials (ASTM) and American National Standards Institute (ANSI). Other organizations establish minimum requirements for materials specific to that industry. An example is the American Water

<sup>1</sup> *Contracts, Specifications, and Law for Engineers*; C.W. Dunham et al. 1979, 3rd ed. McGraw-Hill

Works Association (AWWA). The standards developed by such organizations are accepted by most manufacturers and installers. It can be expected then that a reference in the specifications to such standards will be recognized by the contractor. When referencing the testing procedures, the writer should be absolutely certain of the understanding of any industry standard and ensure that there is no conflict with other contents of the specifications. The Construction Specification Institute (CSI) has a variety of information on specifications and specification writing.

## REFERENCES

- Clark, John R. 1986. *Using and Understanding Engineering Service and Construction Contracts*. New York: Van Nostrand Reinhold.
- Durham, C.W., R.L. Young, and J.T. Bockrath. 1979. *Contracts, Specifications, and Law for Engineers*. 3rd Ed. New York: McGraw-Hill.
- Goldbloom, Joseph. 1989. *Engineering Construction Specifications*. New York: Van Nostrand Reinhold.
- Simmons, H. Leslie. 1985. *The Specifications Writers Handbook*. New York: John Wiley and Sons.

## CHAPTER 31

# CONSTRUCTION COST ESTIMATING

*J. Thomas Tanner, P.E.*

### INTRODUCTION

Land development engineers are frequently asked to prepare cost estimates as part of the land development design process. The estimate's intended use largely determines the level of detail and the information required. In turn, the level of detail determines the time and, hence, the cost of preparing the estimate.

This chapter will define the possible purposes of estimates and identify estimate types and their limitations. It is always important to state the assumptions that were made in the process of preparing the estimate. When preparing estimates with more detail, the importance of including all elements cannot be overstated. Therefore, this chapter provides a recommended checklist of elements that should be included in the detailed estimate. Methods of estimating are identified, as well as sources of current information on construction costs.

Frequently the land development engineer is asked to compare the economic costs and benefits of several alternatives. For this reason, basic engineering economic theory and typical example problems are included.

### PURPOSE OF COST ESTIMATES

Although cost estimates are prepared for both private and public projects, the uses for cost estimates for land development projects differ from those for public projects in a very basic sense. Construction cost estimates prepared for public projects, also called capital projects, are used to determine the economic benefit of the project based on the public need. Cost estimates prepared for land development projects are first used by the entrepreneur to determine whether a project is feasible economically. Later, if the de-

cision is made to proceed with development, a further-refined cost estimate can be used to establish the selling price of the product.

### Public Project Cost Estimates

On a public project, three types of estimates are performed:

1. Preliminary cost estimates
2. Detailed cost estimates
3. Engineer's estimate or bid estimates

It is important for the land development engineer to understand the process by which a cost estimate for a public facility is prepared since a publicly funded project is frequently an integral part of the overall land development venture.

The preliminary cost estimate for public projects is prepared prior to the development of detailed engineering plans and is used to solicit funding, generally from an annual capital improvement program or a voter referendum or bond issue. It is prepared with only a basic understanding of the general scope and extent of the project. Additional considerations are included, if known, such as special structures that are required, land acquisition costs and access considerations. Often, costs from recently constructed projects of similar design are used as a source of the per-foot or per-structure cost. Generally, a significant contingency is included, as much as 30%, and frequently the cost of inflation is estimated since it may be several years before the project actually is constructed.

The detailed cost estimate is performed after the project is funded and designed. It is based on actual quantities

estimated from the design plan. It is very similar to the detailed cost estimate for private projects, which is discussed in more detail later in this chapter. The detailed cost estimate should have an accuracy range of 10 to 15%.

The engineer's estimate or bid estimate is prepared as the project is awarded for construction bidding. This estimate is very detailed and it should be as accurate as possible, within  $\pm 5\%$ . It should be based on a thorough review of the plans and specifications, the solicitation of prices from suppliers, and the most recent cost of construction of the specific elements of the plan.

### Private Project Cost Estimates

For private land development projects the types of estimates are:

1. Feasibility cost estimates
2. Preliminary cost estimates
3. Construction cost estimates

The feasibility estimate is used by the developer in making a "go" or "no go" decision. The preliminary cost estimate is performed after a plan has evolved but before the preparation of actual construction drawings. It is used by the developer to secure funding and to assist in developing alternatives before a final decision on product details. The construction cost estimate is prepared to finalize loan commitment and to compare bids from contractors. Each of these types of estimates is defined further in the next section of this chapter.

In summary, while there are some similarities between public and private project cost estimates, the uses to which these are applied varies. The purpose of the cost estimate for a public facility is to justify and prioritize the project, as well as to determine the economic benefit or need as weighed against the cost to the public. The land development cost estimate provides the developer with that all-important part of the development process: the cost. This estimates also provides, consequently, the amount that must be charged to recover the costs and still achieve the desired profit.

### TYPES OF ESTIMATES

As stated above, there are three basic types of estimates used in the land development process, although there may be refinements or variations. Generally, as a project progresses from the conceptual stage through design to construction, these estimates come into play in order to make basic decisions for the project. The primary difference among the three estimates is the level of detail on which the estimate is based.

### Feasibility Estimates

Land development engineers are frequently asked by clients to assist in determining the initial feasibility of a project. In

this process, the general procedure is to review such elements as availability of services (utility services, public services, schools), the potential yield in terms of number of units or square footage of commercial or office space, potential environmental impacts and a host of other elements, all of which can affect the developer's "bottom line" (profit).

Many of these elements involve substantial expenditures, both in terms of fixed and capital costs. If the project is constructed at a time of high interest rates or when loaned capital is scarce, a well-prepared, all-inclusive cost estimate for the planned project is essential. It can insure that the project is worthwhile or can result in a decision to abandon the project before the incurring of any additional cost, including the development of detailed plans. Investment decisions by both the developer and the lender rely heavily on these estimates.

As technology changes, the consideration of alternate ways of performing some of the basic construction processes can be important. Cost estimates of each alternative can influence the final mix of technology. While the engineering portion of a feasibility study can determine whether the project can be performed technically, the economic or cost-feasibility portion of the study will tell if it makes sense for the project to proceed financially.

It is common knowledge that many projects that have lost money have resulted from decisions that were based on incomplete studies or that made optimistic rather than conservative assumptions. A good rule-of-thumb when estimating costs is to err on the side of conservatism.

Cost feasibility studies are frequently used to identify a more economical use of the parcel of land. For example, in evaluating the different lot sizes that can be placed on a parcel of land, one may attempt to maximize development lot yield by minimizing lot size. However, a cost feasibility estimate can provide information to establish the true cost of the individual lots. The result can determine that the most profitable mix might not be the one that results in the most units.

As one can see, the feasibility estimate is used for exactly that purpose; that is, to determine the economic feasibility of any given item. That item can be the entire project or, on a smaller scale, a variation of a particular roadway alignment, or even for a bike path or water main extension. Feasibility estimates can be separated into two types: (1) rough or "ballpark" estimates; (2) detailed feasibility estimates.

**Rough or "Ballpark" Estimates.** Many times during the course of designing a land development project, an engineer is asked to provide an informal estimate in order to determine if a particular item under discussion should be pursued or given further study and design. These informal estimates are sometimes called "off-the-cuff" or "seat-of-the-pants" estimates. Unfortunately, the whimsical nature of these terms can imply that these estimates need not be accurate and, hence, can be done by anyone. In truth, only engineers with experience should give ballpark estimates,

as only years of experience can allow them to ascertain what should be included in such an estimate. In addition, only with experience can one look at the final answer and know if it “makes sense”. It may be better if the land development engineer provides the quantities and allows the owner/developer to provide the unit cost numbers, unless the engineer has some current costing data on which to render his opinion.

In any case, it is important that the engineer document, in a memorandum, his or her assumptions and methods of determining such an estimate. That memorandum can be given to the owner/developer if appropriate or requested, but in any case, it should be placed in the project files for later documentation if questions arise.

These types of estimates are also called “order-of-magnitude” estimates. Generally they have a low level of precision and can vary as much as 25 to 50%. One can not expect precision closer than  $\pm 20\%$ . While precision is sacrificed in these types of estimates, they can be useful since one can evaluate a large number of alternatives in a short period of time. One must always document and be aware of the inherent limitations of this process.

This rough-estimating procedure presumes an “upper end” average cost for the significant items of the project. One might assume a certain dollar cost per linear foot for installation of storm sewer pipe, even though the cost would actually vary according to pipe size and depth. If, however, there is only a small amount of storm sewer on the project and this cost, relative to other items in the project, is insignificant, then it might not be included in the “ballpark” estimate. Items such as this are lumped together in the  $\pm 50\%$  allowable error. This is where the experience level of the engineer is important—in knowing what or what not to include in the analysis and knowing what “upper end” average cost can be used.

The following two example problems are intended to show how the rough-estimating process can be used for a general large-scale project and for a more limited portion of a specific site problem.

**EXAMPLE 1:** The owner/developer is considering proposing to provide 20% of the construction cost for an elementary school as part of the residential development project. Before he makes this proposal, he asks the engineer for an order-of-magnitude estimate of the cost.

In this example, the engineer decides to provide a “ballpark” or order-of-magnitude estimate by comparing the anticipated school with a recently completed school of similar nature. In doing some research, he finds a school was constructed two years ago in a neighboring jurisdiction for \$3.5 million. He is aware that soil and terrain characteristics are similar for the two sites but that the anticipated student population for the proposed school is one-half of that of the comparison school. He decides to compare the

two schools based on a construction cost per student basis. In other words, he assumes that a school for one-half the number of students would cost 50% of the value of the school being compared.

The engineer performs the following calculations:

1. He assumes a 6% inflation rate/year in construction costs; therefore, the comparison school would cost \$3.93 million today (two years of inflation at 6%).

2. He assumes for this estimate that a school serving  $\frac{1}{2}$  the number of students would cost 50% of the \$3.93 million or \$1.96 million. He decides to round this to \$2 million.

3. He assumes that the school will be constructed at the end of the development phasing of the project (5 years from today). With assumed inflation of 6%/year, the final cost in 5 yr of the school is \$2.68 million.

4. To allow for errors in the assumption that school costs are directly related to school population, and to recognize that the differences in site characteristics can influence costs substantially, he assumes a 25% contingency. The costs for the new school are then estimated to be \$2.68 million  $\pm 25\%$  or a range of \$2.0 to \$3.4 million.

5. The engineer provides the cost estimate of \$2.0 to \$3.4 million to the developer. Since the developer is considering offering 20% of the cost, he can consider a future obligation of between \$400,000 and \$680,000.

6. The engineer documents his assumptions and the source of his data in a memorandum to the owner/developer and a copy is kept in the project files.

**EXAMPLE 2:** A recreation center is proposed on a project. It is known that 10,000 cubic yards of borrow material is needed. It has been determined that this material can be taken from a 2-acre site near the recreation center site. No more than 4 ft of depth can be removed or the site will fall below street grade. Is this site of borrow fill sufficient?

The engineer performs a “ballpark” estimate to determine the acreage of the borrow site and to calculate the earthwork yield going through the following steps:

1. He determines the cubic yards generated in an acre-foot of earth (1 acre, 1 ft deep) by knowing that there are 43,560 ft<sup>2</sup> in an acre:

$$\frac{43,560 \text{ ft}^2 \times 1 \text{ ft deep}}{27 \text{ ft}^3/\text{yd}^3} = 1613 \text{ yd}^3$$

Engineer rounds to 1600 yd<sup>3</sup> per acre

2. Knowing that the site is 2 acres in size with 4 ft of soil available, the engineer calculates the following:

$$1600 \text{ yd}^3/\text{acre} \times 2 \text{ acres} \times 4 \text{ ft depth} = 12,800 \text{ yd}^3$$

available. Please note that this example does not address topsoil removal for structural fill.

3. The engineer, assuming that this fill will be placed in an engineered or controlled fill with a “shrinkage” or compaction factor of 15%, calculates the following:

$$12,800 \text{ yd}^3 \text{ available less } 15\% = 10,880 \text{ yd}^3 \text{ fill quantity}$$

4. The engineer recommends that the site should provide sufficient fill if not more than 15% compaction is performed. The notation should be made that while the quantity may be sufficient, the quality of the soil still needs to be determined.

5. The engineer documents his assumption and calculations in a memorandum with a copy to owner/developer if requested. A copy is kept in project files.

To summarize, “ballpark” estimates should be understood to represent nothing more than a rough approximation; they are intended to be quick estimates. They can be refined when more accurate data is available. They can be an important part of any project development in that they enable quick evaluation of some topic without detailed design or cost estimating. They should only be prepared, evaluated or checked by experienced personnel.

Too many times these “ballpark” estimates are taken to be exact, especially when they are underestimated. The contractor, developer, or other land design professional then wonders what went wrong. This is another reason to err on the conservative side.

**Detailed Feasibility Estimates.** A feasibility estimate should be performed for all land development projects. Often, the feasibility estimate is part of a larger project-feasibility study that evaluates availability of services, potential yields, and potential environmental impacts along with other important factors. If a “ballpark” estimate indicates a favorable economic condition, it is always desirable to pursue the development of a more detailed feasibility estimate. This type of estimate makes a far more refined assessment of quantities and costs associated with project construction and should incorporate all expenditures, including those for land and construction costs, as well as infrastructure and off-site construction. It is important that this estimate contain the costs for engineering design and surveying, legal fees, governmental fees and, if appropriate, the cost of borrowed capital.

This estimate is also called a conceptual or budget estimate and should be accurate to within 15 to 20% of the actual cost. To perform this level of estimate, a preliminary

layout of the project with streets and lots is needed. This layout is important in order not only to place the streets on the horizontal plane, but to insure careful consideration of a preliminary design of utility infrastructure. Preliminary lots are needed to realize yield and to indicate if the market price will offset the cost of the development and still yield a reasonable profit.

It is important to note that attaining additional yield does not always guarantee additional profit. Often, the cost to develop the additional yield is offset by additional infrastructure or other costs. Example 3, which follows later in this chapter, reinforces this concept.

**Feasibility Checklist.** In developing a cost estimate at this stage of the project, some design data is needed so that rough take-offs of quantities can be performed. Some engineers include the cost estimate right on the development plan, so that as the plan changes and becomes more refined, the cost estimate can be updated as well.

Most engineering firms develop a feasibility checklist so that all items are considered even if some might not be needed for all estimates. The checklist also makes it easy to compare estimates with other projects and insures consistency in checking by a second party (i.e., someone other than the design engineer).

Generally, the following cost elements are to be included:

- Land
- Roadways
- Storm sewers
- Clearing and grading
- Demolition
- Sanitary sewers
- Water supplies
- Other utilities
- Earthwork (lots only)
- Erosion and sedimentation control
- Stormwater management system
- Recreational facilities
- Off-site costs (special costs)
- Permits and bonds
- Geotechnical testing professional fees

See the following section describing each of these elements in detail.

### Preliminary Estimates

Preliminary estimates, sometimes referred to as semi-detailed estimates, although considerably refined from the feasibility estimates, are developed from plans that have not

yet reached sufficient detail to derive construction costs. It is felt that these estimates should be accurate within 10 to 15%.

Preliminary estimates are usually made after a plan has evolved to a fine-tuned stage but prior to actual preparation of hard-line construction drawings. They can serve as advanced-conceptual or advanced-budget estimates and are usually accurate enough for making decisions regarding the feasibility of a project or for a decision involving a choice among alternatives within a project.

For preliminary estimates, the land development plan, which should be drawn to scale, should include the preliminary road alignments and in some cases preliminary site grades. These preliminary grades can be used to perform initial designs of storm drainage, water and sewer, as well as other infrastructure-related items. These plans can be used to estimate several important design quantities, which when multiplied by the applicable unit costs can provide a good estimate.

A checklist should be prepared listing all cost items, and the checklist for the preliminary estimates should be at least as inclusive as that employed for the detailed feasibility estimate. Since the quantities are still in a preliminary stage, a contingency factor of 15 to 20% should be added to the bottom-line subtotal, depending on the completeness and accuracy of the plans.

It is critical that the engineer and the client agree on the units of measure to be used for each of the elements included in the estimate. The unit of measure may be obtained from a source such as the State Department of Transportation or Agency Department of Public Works. Private projects may not always coincide with public specifications. The developer may require the units of measure in a certain format for loan application purposes or for analysis by his contractors. Additionally, it is important to establish with the client the exact unit of measure for each element.

The elements that should be included in the preliminary estimate are similar to those in the feasibility estimate but should be based on the more complete plans. The elements to be included are discussed below.

**Land Costs.** Land costs are usually expressed in dollars per acre and are provided by the owner. This is the line item within the estimate where the initial cost or price paid to the former owner for the parcel is accounted for in the estimate. Since the price is usually determined on an area or acre basis, it is critical that a complete boundary survey be done to determine the exact parcel size. Boundary surveys are described in detail in a separate chapter of this text. It is important here to note that it is a land cost and should be included in the cost estimate.

To be complete, the cost of the deed, title insurance and any transfer taxes should be included in the estimate. Real estate commissions are usually paid by the seller, but this should be verified since they can be as high as 6 to 8% of the selling price.

**Roadway Costs.** Costs are usually expressed in dollars per linear foot for roads and streets and a lump sum cost for parking lots. This can be misleading since there could be many separate items included in a linear unit cost, including earthwork, pavement, aggregate for the base and sub-base course, curb and gutter, underdrains, and sidewalks, along with general street landscaping and sodding. In addition, if appropriate, the costs associated with street lighting, street trees, signs and signals should be included. It should be emphasized that one of the objectives of road design is to obtain an earthwork balance of both the road and the entire site. Without an earthwork balance, the entire project costs can increase significantly.

Pavement costs are usually expressed in dollars per ton of asphalt pavement or per square yard per inch of depth for concrete pavement, and wither cost is usually a considerable portion of the roadway cost. For this reason, the development engineer is expected to minimize these costs by innovative design. The traffic count or anticipated traffic load will dictate the street width and pavement design. It is important to verify the sidewalk requirements of the jurisdiction early in the design process. Sidewalks are both an economic impact as well a limitation on lot design. Please note that sometimes sidewalk requirements vary by lot size and street classification. Sidewalk costs can be reduced through judicious use of lot sizes and street types. In some cases, less expensive bike trails can substitute for sidewalks, but the engineer must be assured that they will be approved by the jurisdiction.

**Storm Sewer Costs.** Storm sewer costs are usually expressed in dollars per linear foot of constructed pipe. Included in this cost is the pipe itself, culverts, inlets, manholes, endwalls, riprap or other erosion protection, special drainage structures, underdrains, box culverts, rock excavation and special pipe bedding if needed. Additional costs include backfilling and any site restoration costs, including seeding or sodding if the pipe is placed outside the roadway section. Costs of storm sewers can be reduced by utilizing the minimum adequate pipe size, using the least number of drainage structures, and in placing pipe depths as shallow as possible.

**Sanitary Sewer Costs.** Sanitary sewer costs are usually expressed in dollars per linear foot. Some land development projects may require construction of sewage treatment facilities or enhancement of existing facilities.

Basic sanitary sewer costs include the piping, laterals to the property line (house connections), manholes, drop connections, ejection pumps, and special structures. Restoration costs such as seeding and/or sodding should be included if the pipe is placed outside the roadway section. As with storm sewers, costs for sanitary sewers can be reduced by keeping pipe lengths and sizes to the minimum, locating the trenches to more shallow depths, and minimizing the number of structures. Particular interest should be shown to rock excavation, roadway crossings, and boring/tunneling, as these items can be very costly.



**Water-Supply Costs.** As with sanitary sewers, costs for water supply are expressed in dollars per linear foot. If a water-supply facility, such as a reservoir or a special pumping facility, is needed, it is recommended that the land development engineer utilize the services of an engineer experienced in this specialty to provide the cost estimate. Other items included in the water-supply costs are valves, hydrants, water service line, firelines, blow-off and air releasers, tees and pumps. Cost-minimization methods are similar to storm sewer and sanitary sewer design, and the elements included in the estimate are pipes, laterals services, fittings, and special structures such as valve vaults.

**Other Utility Costs.** Other utility costs such as gas mains, fuel transmission pipelines, electric supply, telephone cables and cable television should be included. The utility companies themselves, which can be either public or private companies, can best supply the cost estimates for gas, electric, and cable TV facilities. Some estimate should be included in the project estimate total for these facilities, if feasible.

**Earthwork Costs (Lots Only).** Earthwork costs for the various infrastructure elements are usually included in the per foot cost of these elements; however, earthwork costs for the lots should be estimated separately. Earthwork costs are usually expressed in dollars per cubic yard and include cut, fill, topsoil, borrow and excess (earth hauled off the site). The engineer is directed to the chapter on grading for discussion on the importance of attempting to balance the earthwork on the site. Earthwork costs can be significant, and earthwork engineering often is not readily understood by the developer. Care must be taken during design so that, if possible, phasing of construction can be done. It is cost-effective to construct first in areas of cut so that earth can be moved to areas of fill without a secondary storage step or without having to be moved off site only to be brought back later for fill. Also, considerations should be made if rock is shallow on the site. The engineer may consider raising the grade of the proposed site to eliminate the need to rip or blast through rock. Earthwork costs at this stage of plan development are based on spot elevations rather than on detailed contour grading.

**Erosion and Sedimentation-Control Costs.** Most jurisdictions have now adopted strict erosion and sedimentation-control requirements. The land development engineer must become familiar with the latest requirements, both for design purposes and for cost estimating. Cost elements include sediment basins, straw and maintenance bales, silt fence, berms, gravel for truck-wash facilities and wash pipes, traps at stream inlets, riprap, fencing for sediment/soil control, and erosion matting. Similar to other elements of design, proper phasing of construction activities can minimize the cost of these elements. Also included within this section of costs is the initial clearing and grubbing of the site. This cost can vary significantly based on the amount and extent of the vegetation growth on the site.

**Stormwater Management Systems Costs.** Similar to erosion control, jurisdictions are adopting more stringent stormwater management systems requirements. In some cases these systems can equal or exceed the costs of the storm drainage system. Included in the system are the costs for embankment and excavation, oil and grit separator structures, spillway and outlet structures, concrete, riprap, sod and landscaping materials and piping.

During the design of the project, systems costs can be minimized by carefully placing facilities where they can serve both as natural aesthetic elements (ponds) and stormwater facilities.

**Recreational Facilities Costs.** Often, the development of a project will include the cost of recreational facilities. These may be proposed by the developer to enhance the project's *viability* or in an attempt to seek support for approval of the development. In addition, the approving jurisdiction may impose a requirement for additional recreational facilities during the approval process. These facilities may serve only the proposed development or may serve additional users outside the project. For example, the jurisdiction may require the dedication of land for a regional park and require the developer to construct a pond as his commitment. The jurisdiction could construct the park features such as trails and buildings, but provide for future maintenance as the public commitment to the park's development. In this example, the pond could serve as the stormwater detention facility for the project and recreational facility for the jurisdiction. As part of the cost-estimating process, only the developer's costs should be included in the cost estimate. A well-planned development could take advantage of these features through good initial planning and design.

Elements that should be included in the recreational facility costs are park development, sod and/or seeding, earthwork such as mounding, tennis courts, swimming pools, basketball facilities, fencing, and tot lots. If there is an excess of earthwork in the project, good landscape design through the creation of sound berms and mounds in the park can reduce the costs of hauling away excess, unsuitable material. On the other hand, if there were a shortage of fill material, sound landscape design would consider lowering the future park site.

**Off-site Costs (Special Costs).** In order to serve the development with adequate utilities or to direct sewage or stormwater away from the site adequately, many off-site costs can be incurred by the developer. Good engineering design should identify the downstream effects of stormwater leaving the site and determine the increase in size of the downstream element of both the stormwater and sanitary sewer piping systems. Often, these enhancements are the responsibility of the developer and should be included in the estimated cost of the project. Care must be taken not to state prematurely that these are 100% the responsibility of the project development. In some instances, the case can be made that these enhancements benefits adjacent future

development or the public in general, and, therefore, the costs should be shared. At this stage in the development, the engineer preparing the cost estimate can make an assumption and use the assumption in his calculation, but footnotes should be made to identify the assumption and to provide a brief justification of the assumption.

Special or off-site costs that should be considered are: streets and/or street widening and reconstruction, traffic controls, signalization, bridges, sidewalks, hike/bike trails, stormwater management facilities (in particular, regional facilities), landscaping, lighting, and signs.

**Permits and Bonds Costs.** On most projects, permits from the local, and sometimes the state, governments must be obtained before construction can commence. Typically, permits are also required when making connections to utility systems such as electrical service, water service and sanitary systems. Frequently, these are called “hook-up” fees and can be used by the jurisdiction to make system-wide improvements or upgrades. Many kinds of construction activities require inspections, both during and after construction, that are intended to provide consistency in construction practices and to provide for safety during both construction and subsequent overall system use.

The permits associated with developing a tract of land can be numerous. The cost associated with procuring the vast number of permits is one that must be accounted for. Permits are usually required for grading, paving, drainage, sewer connections, water connections, electrical and gas connections, wetlands delineation and protection, and forest or tree stand delineation and protection. Additionally, the local and state governments could have additional permit requirements. Generally, the fees charged by the reviewing agency are structured to recover their costs for review of plans and inspection, but some governments use these fees as a way of securing additional general revenue. In any case, it is the responsibility of the engineer to identify carefully all fees for the required permits, since these fees and permits can be in the range of 20 to 25% of the physical construction costs for the project (excluding land and house construction costs).

On most projects of substantial size, the developer and/or contractor must post a surety bond to insure adequate construction quality. While this bond is returned at the end of the project, it can involve a loan from a bonding company or, at a minimum, lost interest for the construction period. Many jurisdictions now permit the potential partial release of bonds as project phases are completed. While this does not affect cost estimates, it can substantially reduce the developer's cost, and if the engineer can identify this possibility and move promptly to get the jurisdictional partial bond release, the client can save in terms of lost interest cost.

**Professional Fees.** As part of any project, professional consultant fees should be included in any cost estimate since they can represent as much as 15% of the project

cost. These fees include those of the engineer, surveyor, and architect, as well as geotechnical, construction observation, and attorney expenses. Generally these fees are based on the cost of the facilities, factored by the complexity of the project. Attorney fees vary widely from a lump sum for the entire project to an hourly fee. Whatever method is chosen to estimate the cost, the assumptions should be clearly stated in the report that accompanies the estimate.

The engineering and architectural fees can be divided into two categories: (1) the planning and engineering cost of development of the plans and specifications; and (2) the cost of engineering, surveying and construction management during construction. Typically, professional fees vary between 7 and 15% percent of the total project cost.

**Financial Fees.** The inclusion of financial fees in the cost estimate is best left for the client to estimate, based on his particular situation. Financial fees are usually based on a percentage of borrowed money. Interest charges during the life of the project can be substantial if the project has a long history from purchase of land to completion of construction. The cost of financing may range from 5 to 20% of the total project cost.

**Contingency Allowance.** Allowing for contingencies (unforeseen conditions or variances in design elements) is a key part of the estimate of project cost. There is no generally accepted percentage. The amount varies by the complexity of the project, the anticipated time for completion and the individual idiosyncrasies of the engineer (e.g., conservative versus liberal). It is common to see contingencies used from 5 to 30%. If the engineer is unsure of the contingency to allow, it is recommended that a minimum of 15% be utilized. If the client disagrees, the amount can be altered by the client. However, it is recommended that the engineer's original contingency be documented in the project files. If the project is to be phased over numerous years, the cost of escalation should affect the contingency, or alternatively a separate line item can be included for inflation.

**Other Miscellaneous Costs.** Other miscellaneous costs should be included if pertinent. Consideration should be given to such items as marketing analysis, maintenance cost of project elements constructed but not accepted by the operating agency until later, and other costs not accounted for in any previous section. These might include such items as demolition of existing structures and costs associated with environmental mitigation and special landscaping.

Among other uses, preliminary estimates are valuable for obtaining developer loan applications, developing cash-flow schedules, and fine-tuning preliminary market prices for product.

### Construction Estimates

Construction cost estimates, also referred to as detailed estimates, are generally developed only after the construction plans have been approved by all necessary agencies. Quantity take-offs, which provide the quantities of work to be

performed, are often made before all final approvals are obtained. Therefore, it is critical that all take-offs and estimates be dated and referenced to a dated plan sheet. The importance of dating the estimate can not be overstated. The plan set is often revised during the final approval process, and you need to insure that the final construction cost estimate is based on the plans as used for construction.

Construction cost estimates can be performed by different professionals but, as always, should be reviewed and approved only by persons with many years of experience. This is to assure thoroughness and credibility of the estimate. It is important that somewhere in the process an individual with knowledge of the details of construction work either develop the estimate or check the efforts of others. This experience is best obtained through actual experience in construction work. It is also important for the estimator to be able to visualize the various construction phases of a project and to be able to incorporate equipment needs, materials needed and the handling of these during construction.

It is important during the development of the construction cost estimates that the latest available information be used. An experienced engineer should keep an updated file with the latest bid information on materials, labor and the availability of construction items. Later in this chapter, a listing of possible sources of this information is provided.

In general, the construction estimate should include the quantities and unit costs of all items needed to construct the project. The construction cost estimate can be used for some of the same items as identified for the preliminary estimate, such as loan applications, cash flows, and setting of market prices. In addition, this estimate can provide the owner/developer with a reasonably accurate guide of the amount of funding needed to complete the project.

Often the developer will ask contractors for bids on selected portions of the project, including infrastructure, grading and roadway/street construction. The construction cost estimate should be formatted so that the data needed to compare actual bid prices with the estimate is easily obtainable. Wide disparities in the bids, if several bids are being compared, usually indicate a lack of clarity in the plans, specifications, or instructions to the bidder. One or two marked deviations from all of the bids, however, usually indicates some oversight or lack of understanding by the bidder.

It is felt that the construction cost estimate should be accurate within 5 to 10%. The accuracy depends on the complexities of the plans and specifications and on the currency of the bid data used to develop the unit costs. Errors in the design plans can greatly affect the accuracy of the cost estimate. Back charges and extras by contractors because of incomplete and/or inaccurate plans will increase costs significantly.

$$C \pm \varepsilon_c = (A \times B) \sqrt{(A \pm \varepsilon_B)^2 + (B \pm \varepsilon_A)^2} \quad (31.1)$$

Since construction cost estimates utilize quantity take-offs from drawings to be used in constructing the project, the cost estimate obviously will be only as accurate as the quantities to which costs are assigned and the accuracy of those assigned costs. In addition, the engineer must recognize that multiplication of numbers, as performed when applying unit cost to quantities, can result in magnification of errors. Propagation of errors in the product of two independent variables, where A and B are the independent variables and e is the uncertainty or error in the variables, is shown in Equation 31.1. For example, suppose the estimate for excavation is 10,000 CY  $\pm$  20% and the cost is \$4.70  $\pm$  10%. Using Equation 31.1, the accuracy of the cost is

$$\begin{aligned} (10,000 \pm 20\%) \times (\$4.70 \pm 10\%) & \quad (31.2) \\ = (10,000 \pm 2000) \times (\$4.70 \pm \$0.47) \\ = (10,000 \times 4.70) \pm \sqrt{(10,000(0.47))^2 + (4.70(2000))^2} \\ = 47,000 \pm 10,510 \\ = \$47,000 \pm 22.4\% \end{aligned}$$

Note that although the estimated range (i.e., error) in quantity is 20% and the estimated range (i.e., error) in the cost is 10%, the overall accrued error is 22.4%

When two independent variables are added together, the accrued error is represented by Equation 31.3.

$$C \pm \varepsilon_c = (A + B) \pm \sqrt{\varepsilon_A^2 + \varepsilon_B^2} \quad (31.3)$$

Table 31.1, generated from Equation 31.1, provides the expected error given the errors of the two independent variables. For example, the error of 22.4% calculated using Equation 31.2 can be obtained by entering 20% for variable A and reading across to the 10% column of variable B. Table 31.2 shows how errors are accrued when numbers are multiplied and added together. Care must be taken to minimize the errors; otherwise, they can be compounded throughout the estimate.

It is imperative that every item indicated in the construction plans be included in the estimate, since the contractor installing these items will most surely include it. It is especially important for the construction cost estimate to try and give at least a "guesstimate" of contingent items such as rock excavation, pipe bedding, excavation and removal of unsuitable material, undercutting and other similar items. These items need to be recognized initially, so that if they become necessary in the actual construction, they will have been planned for, even though they were not specifically indicated on the construction drawings. When such "guesstimates" are included, it is recommended that they be footnoted to differentiate them from the normally more accurate cost items.

It is recommended that engineers utilize a standard checklist and a statement cost estimate sheet or develop ones if they do not exist. Existing computer programs for

**TABLE 31.1 Percent Error Expected in the Product of Two Independent Variables**

% ERROR IN VARIABLE A	PERCENT ERROR IN VARIABLE B						
	0	5	10	15	20	25	30
0	5	5.0	10.0	15.0	20.0	25.0	30.0
5	5	7.1	11.2	15.8	20.6	25.5	30.4
10	10	11.2	14.1	18.0	22.4	27.0	31.6
15	15	15.8	18.0	21.2	25.0	29.2	33.6
20	20	20.6	22.4	25.0	28.3	32.1	36.1
25	25	25.5	27.0	29.2	32.1	35.4	39.1
30	30	30.4	31.6	33.6	36.1	39.1	42.5

data manipulation work well to standardize the form and can perform all arithmetic functions automatically. When computerized, these forms can be the basis of a method of filing historic costs. A typical checklist is shown in Figure 31.1. A standard cost-estimate sheet is shown in Figure 31.2.

For construction cost estimates, a contingency of 10% is usually employed, depending on the source and timeliness of the unit costing.

Table 31.3 shows a procedure list as used by a major engineering firm specializing in land development engineering. It is intended to provide a framework for proceeding through the preparation of a cost estimate. Not all items will be needed for all estimates, but it is felt that all should be considered.

### VARIABLES AFFECTING COST

Many variables affect the cost of the project. As the engineer is preparing the cost estimate, he should be aware if any of the following items are significant enough to be investigated in more detail in the estimate.

#### Size and Complexity of the Project

The size and complexity of the project can have a significant effect on the cost of physical improvements, particularly

when considered on the cost per unit. Smaller sites can require a non-proportional cost for infrastructure improvement, particularly with respect to streets and underground utilities. Quantity discounts, which are given for construction materials, may not apply for smaller sites. In addition, it is important to note that attaining additional yield, (e.g., more housing units) does not always guarantee additional profit. This is especially true if the cost to develop the additional yield is offset by additional infrastructure costs. The following example shows how this condition could occur.

**EXAMPLE 3:** After a 60-unit, single-family development project is into the design stage, the client (developer) is considering an offer to purchase an adjacent site to add to the total project. The site would add 12 units and, because of its location, would enhance the attractiveness of the project. However, the parcel of land being considered has steep terrain and must be elevated by fill. To accommodate the additional units, the sanitary sewer and storm drain system for a portion of the initial site (Phase I) has to be constructed deeper for 25% of the units at an additional cost of \$2000 per unit. How should you as the developer's engineer analyze the situation and what advice should you give to your client?

**TABLE 31.2 Compounding of Errors Showing Quantities Times Unit Costs**

ITEM	QUANTITY	UNIT COST	TOTAL COST	ACCURACY (\$)	ACCURACY (%)
Excavation	10,000 CY ± 20%	\$ 4.70 ± 20%	\$47,000	\$10,510	± 22%
Concrete Sidewalk	250 LF ± 15%	\$17.00 ± 20%	\$ 4250	\$ 1063	± 25%
36-in. Diameter Concrete Pipe	320 LF ± 10%	\$60.00 ± 15%	\$19,200	\$ 3461	± 18%
			\$70,450	\$11,116	± 16%

SUBDIVISION NAME \_\_\_\_\_ (Section, Lot No.) \_\_\_\_\_

Contract/Work Order No. \_\_\_\_\_ Date \_\_\_\_\_

\_\_\_\_\_  
Signature

\*\*\*All cost estimates shall be prepared by a senior engineer who will include all the following items in the cost estimate. (If not, briefly describe why!)

<p>1. _____ <b>Clearing and grubbing</b></p> <p>2. _____ <b>Excavation:</b>                  a. _____ Class A, Rock excavation                  b. _____ Class B, Regular                  c. _____ Class C, Wet                  d. _____ Class D, Haul                  e. _____ Class E, Borrow                  f. _____ Class F, Shrinkage</p> <p>3. _____ <b>Sheeting and shoring</b></p> <p>4. _____ <b>Utility lines</b>                  a. _____ Sanitary sewer lines                  b. _____ Storm sewer lines                  c. _____ Underdrains                  d. _____ Waterlines                  e. _____ Electric                  f. _____ Gas                  g. _____ Lighting                  h. _____ Remove and replace pavement</p> <p>5. _____ <b>Utility lines should be estimated considering the following:</b>                  a. _____ Depth of excavation                  b. _____ Gravel underdrains                  c. _____ Size and classification of main                  d. _____ Linear foot quantities                  e. _____ French underdrains                  f. _____ Sheeting and shoring                  g. _____ Hand excavation                  h. _____ Rock excavation                  i. _____ Wet excavation</p> <p>6. _____ <b>Concrete work</b>                  a. _____ Curb and gutter work                  b. _____ Sidewalks                  c. _____ Wingwalls                  d. _____ Cut-off walls                  e. _____ Riprap                  f. _____ Reinforced concrete                  g. _____ Miscellaneous concrete work                  h. _____ Retaining walls</p> <p>7. _____ <b>Pavement</b>                  a. _____ Subbase                  b. _____ Base                  c. _____ Paving - Asphalt/concrete                  d. _____ Paving - Portland cement</p>	<p>8. _____ <b>Seeding</b></p> <p>9. _____ <b>Sodding</b></p> <p>10. _____ <b>Driveways and driveway aprons</b></p> <p>11. _____ <b>Guardrails, reflectors</b></p> <p>12. _____ <b>Painting</b></p> <p>13. _____ <b>Lighting</b></p> <p>14. _____ <b>Engineering</b>                  a. _____ Engineering design                  b. _____ Surveying stakeout</p> <p>15. _____ <b>*Point out in cover letter items not included, such as:</b>                  a. _____ Land costs                  b. _____ Interest costs                  c. _____ Inspection costs                  d. _____ Builder's overhead                  e. _____ Legal fees                  f. _____ Onsite work                  g. _____ Planning and preliminary engineering work                  h. _____ Advertising and public relations                  i. _____ Permits and bonds.</p> <p>16. _____ <b>Riprap</b></p> <p>17. _____ <b>Gravel - various kinds</b></p> <p>18. _____ <b>Structural steel work</b></p> <p>19. _____ <b>Timber construction</b></p> <p>20. _____ <b>Temporary construction</b>                  a. _____ Access roads                  b. _____ Diversion ditches                  c. _____ By-passes</p> <p>21. _____ <b>Silt control</b>                  a. _____ Seeding                  b. _____ Silt basin</p> <p>22. _____ <b>Contingency item (10%)</b></p>
--	--

FIGURE 31.1 Cost estimate checklist.

Project Name	_____			
Prepared By	_____	Date	_____	
Checked By	_____	Date	_____	

<u>Item</u>	<u>Quantity</u>	<u>Unit</u>	<u>Price</u>	<u>Cost</u>
<b>A. <u>Clearing and Grading</u></b>				
1. Clearing and Grubbing	_____	Acre	_____	_____
2. Disposal	_____	L.S.	_____	_____
3. Excavation (cut)	_____	C.Y.	_____	_____
4. Embankment (fill)	_____	C.Y.	_____	_____
5. Borrow	_____	C.Y.	_____	_____
6. Spoil/Disposal	_____	C.Y.	_____	_____
7. Topsoil Removal/Storage	_____	C.Y.	_____	_____
8. _____	_____	_____	_____	_____
<b>Sub-Total for Clearing and Grading</b>				_____
<b>B. <u>Erosion/Sediment Control</u></b>				
1. Earth Berm	_____	L.F.	_____	_____
2. Perimeter Dike	_____	L.F.	_____	_____
3. Interceptor Dike	_____	L.F.	_____	_____
4. Filter Fabric Fence	_____	L.F.	_____	_____
5. Gravel Filters: (Excl. filters with basins)	_____	C.Y.	_____	_____
6. Inlet Sediment Traps	_____	Each	_____	_____
7. Sod	_____	S.Y.	_____	_____
8. Seeding (disturbed area minus pavements and buildings)	_____	S.Y.	_____	_____
9. Sediment Basins (including filters)	_____	Each	_____	_____
10. Wash Rack (incl. water service)	_____	Each	_____	_____
11. _____	_____	_____	_____	_____
<b>Sub-Total For Erosion/Sediment Control</b>				_____
<b>C. <u>Streets</u></b>				
1. Bituminous Concrete Surface:				
1"	_____	S.Y.	_____	_____
2"	_____	S.Y.	_____	_____
2. Base Course: 6" Aggregate	_____	S.Y.	_____	_____
3" Bit. Conc.	_____	S.Y.	_____	_____
3. Aggregate Sub-base	_____	C.Y.	_____	_____
4. Cement Treated Aggregate	_____	C.Y.	_____	_____
5. Cement Treated Sub-grade	_____	C.Y.	_____	_____
6. Surface Treatment	_____	S.Y.	_____	_____
7. Gravel Shoulder	_____	S.Y.	_____	_____
8. Curb & Gutter	_____	L.F.	_____	_____
9. Header Curb	_____	L.F.	_____	_____
10. Median Curb	_____	L.F.	_____	_____
11. Curb Cut Ramps	_____	Each	_____	_____
12. Commercial Entrance	_____	L.F.	_____	_____
13. Driveway Entrance	_____	Each	_____	_____
14. Guard Rail	_____	L.F.	_____	_____
15. Traffic Barricade	_____	Each	_____	_____

FIGURE 31.2 Cost estimate sheet.

16. Edge Delineators	_____	Each	_____	_____
17. Sidewalk (4' wide, concrete)	_____	L.F.	_____	_____
18. Rock Excavation	_____	C.Y.	_____	_____
20. Soil Testing (Road)	_____	L.F.	_____	_____
21. _____	_____	_____	_____	_____
		<b>Sub-Total for Streets</b>	_____	_____
<b>D. Storm Sewer</b>				
1. Easement Clearing & Restoration	_____	L.F.	_____	_____
2. Pipe RCCP, Classes II-IV:				
15"	_____	L.F.	_____	_____
18"	_____	L.F.	_____	_____
21"	_____	L.F.	_____	_____
24"	_____	L.F.	_____	_____
27"	_____	L.F.	_____	_____
30"	_____	L.F.	_____	_____
33"	_____	L.F.	_____	_____
36"	_____	L.F.	_____	_____
42"	_____	L.F.	_____	_____
48"	_____	L.F.	_____	_____
3. Pipe, CMP:				
15"	_____	L.F.	_____	_____
18"	_____	L.F.	_____	_____
21"	_____	L.F.	_____	_____
24"	_____	L.F.	_____	_____
27"	_____	L.F.	_____	_____
30"	_____	L.F.	_____	_____
33"	_____	L.F.	_____	_____
36"	_____	L.F.	_____	_____
4. Endwall				
15"	_____	Each	_____	_____
18"	_____	Each	_____	_____
21"	_____	Each	_____	_____
24"	_____	Each	_____	_____
27"	_____	Each	_____	_____
30"	_____	Each	_____	_____
33"	_____	Each	_____	_____
36"	_____	Each	_____	_____
5. End Section				
15"	_____	Each	_____	_____
18"	_____	Each	_____	_____
21"	_____	Each	_____	_____
24"	_____	Each	_____	_____
27"	_____	Each	_____	_____
30"	_____	Each	_____	_____
33"	_____	Each	_____	_____
36"	_____	Each	_____	_____
42"	_____	Each	_____	_____
48"	_____	Each	_____	_____
6. Endwall	_____	Each	_____	_____
	_____	Each	_____	_____
	_____	Each	_____	_____

FIGURE 31.2 (Continued) Cost estimate sheet.

7. Curb Inlet	_____	Each	_____	_____
8. Other Inlets	_____	Each	_____	_____
	_____	Each	_____	_____
9. Junction Box JB-1 Pipe	_____	Each	_____	_____
size:	_____	Each	_____	_____
	_____	Each	_____	_____
10. Manhole	_____	Each	_____	_____
11. Modified Structure	_____	Each	_____	_____
12. Connect to Existing Structure	_____	Each	_____	_____
13. Remove Existing Structure	_____	Each	_____	_____
14. Pavement Restoration	_____	S.Y.	_____	_____
15. Curtain Wall	_____	C.Y.	_____	_____
16. Concrete Cradle/Encasement	_____	L.F.	_____	_____
17. Concrete Anchors	_____	C.Y.	_____	_____
18. Paved Ditch	_____	C.Y.	_____	_____
19. Sod Ditch	_____	S.Y.	_____	_____
20. Rip Rap: Class I UngROUTed	_____	C.Y.	_____	_____
Class I GROUTed	_____	C.Y.	_____	_____
Class II	_____	C.Y.	_____	_____
21. Piling	_____	L.S.	_____	_____
22. Box Culvert (compute Cost on Box Culvert Computation Form)	_____	L.S.	_____	_____
23. Underdrains	_____	L.F.	_____	_____
24. _____	_____	_____	_____	_____
		<b>Sub-Total for Storm Sewer</b>	_____	_____
<b>E. Sanitary Sewer</b>				
1. Easement Clearing & Restoration	_____	L.F.	_____	_____
2. Pipe,(Including 0-8' Trenching and Select Fill): 8"	_____	L.F.	_____	_____
10"	_____	L.F.	_____	_____
3. Pipe, RCCP (Including 0-8' Trenching and Select Fill): 8"	_____	L.F.	_____	_____
10"	_____	L.F.	_____	_____
4. Pipe, Ductile Iron (Including 0-8' Trenching and Select Fill): 8"	_____	L.F.	_____	_____
10"	_____	L.F.	_____	_____
5. Extra Trenching (Not including Pipe):				
8'-12'	_____	L.F.	_____	_____
12-16'	_____	L.F.	_____	_____
16'+	_____	L.F.	_____	_____
6. Manhole (All Manholes) 0'-8'	_____	Each	_____	_____
(Depth over 8') 8'+	_____	V.F.	_____	_____
7. Drop Connection	_____	V.F.	_____	_____
8. Concrete Encasement	_____	L.F.	_____	_____
9. Concrete Cradle	_____	L.F.	_____	_____
10. Concrete Anchors	_____	C.Y.	_____	_____
11. Rock Excavation	_____	C.Y.	_____	_____
12. Sheeting and Shoring	_____	L.S.	_____	_____
13. Pavement Replacement	_____	L.F.	_____	_____
14. Laterals: 4"	_____	L.F.	_____	_____
5"	_____	L.F.	_____	_____
15. Risers	_____	V.F.	_____	_____

FIGURE 31.2 (Continued) Cost estimate sheet.



16. "T" or "Y" Connection:			Each		
	8" x 4"	_____	Each	_____	_____
	8" x 5"	_____	Each	_____	_____
	10" x 4"	_____	Each	_____	_____
17. Lateral Connection to Manhole		_____	Each	_____	_____
18. Connect to Existing Structure		_____	Each	_____	_____
19. Aerial Pipe Supports		_____	L.S.	_____	_____
20. Boring or Jacking		_____	L.F.	_____	_____
21. Select Fill Material		_____	C.Y.	_____	_____
22. _____		_____		_____	_____
			<b>Sub-Total for Sanitary Sewer</b>	_____	_____
<b>F. Water Line</b>					
1. Easement Clearing & Restoration		_____	L.F.	_____	_____
2. Pipe, DIP:	3"	_____	L.F.	_____	_____
	4"	_____	L.F.	_____	_____
	6"	_____	L.F.	_____	_____
	8"	_____	L.F.	_____	_____
	10"	_____	L.F.	_____	_____
		_____	L.F.	_____	_____
3. Valve	3"	_____	Each	_____	_____
	4"	_____	Each	_____	_____
	6"	_____	Each	_____	_____
	8"	_____	Each	_____	_____
	10"	_____	Each	_____	_____
		_____	Each	_____	_____
4. Fittings (Tees, Bends Crosses, Reducers, and Offsets)		_____	LB.	_____	_____
5. Copper Water Service (Includes Curb Stop) Note: Tap fee may cover this item		_____	L.F.	_____	_____
6. Wet Tap		_____	Each	_____	_____
7. Fire Hydrant		_____	Each	_____	_____
8. Blow-off		_____	Each	_____	_____
9. Air Release		_____	Each	_____	_____
10. Rock Excavation		_____	C.Y.	_____	_____
11. Boring and/or Jacking		_____	L.F.	_____	_____
12. Pavement Restoration		_____	L.F.	_____	_____
13. Shoulder Restoration		_____	L.F.	_____	_____
14. Concrete Anchor/Encasement		_____	C.Y.	_____	_____
15. Concrete Cradle		_____	L.F.	_____	_____
16. Relocate Existing Hydrant		_____	Each	_____	_____
17. Sheeting and Shoring		_____	L.F.	_____	_____
18. Select Fill Material		_____	C.Y.	_____	_____
19. _____		_____		_____	_____
			<b>Sub-Total for Water</b>	_____	_____
<b>G. Miscellany</b>					
1. Septic Field		_____	L.S.	_____	_____
2. Pump		_____	Each	_____	_____
3. Monuments and Pipes		_____	L.S.	_____	_____
4. Monuments and Pipes		_____	Lot	_____	_____
5. Walkways and Trails: 4' Asphalt		_____	L.F.	_____	_____
	8' Asphalt	_____	L.F.	_____	_____
	4' Gravel	_____	L.F.	_____	_____
	4' Woodchip	_____	L.F.	_____	_____

FIGURE 31.2 (Continued) Cost estimate sheet.

6. Landscaping	_____	L.S.	_____	_____
7. Recreational Amenities	_____	L.S.	_____	_____
8. Screening	_____	L.S.	_____	_____
9. Fencing	_____	L.S.	_____	_____
10. Retaining Wall	_____	L.S.	_____	_____
11. Well	_____	Each	_____	_____
12. Street Sign	_____	Each	_____	_____
13. Wheel Stops	_____	Each	_____	_____
14. _____	_____	_____	_____	_____
<b>Sub-Total for Miscellany</b>			_____	_____
<b>H. Total Cost</b>			<b>Cost</b>	
1. Sub-Totals:		Clearing and Grading	_____	
		Siltation/Erosion Control	_____	
		Streets	_____	
		Storm Sewer	_____	
		Sanitary Sewer	_____	
		Water	_____	
		Miscellany	_____	
		<b>GRAND SUB-TOTAL</b>	_____	
2. Engineering & Contingencies ( %)			_____	
		<b>TOTAL COST</b>	_____	
<b>Note: Cost Data Taken from available records.</b>				

FIGURE 31.2 (Continued) Cost estimate sheet.

**ANALYSIS AND SOLUTION:** You, as the engineer, are aware that the developer is seeking a 12% profit or \$15,000 per unit. You decide to evaluate the additional costs related to the incremental development (Phase II) against the required profit.

First, you determine the required incremental profit: \$15,000/unit × 12 units = \$180,000 (required profit). Based on a preliminary evaluation of elevations of the Phase II site, you estimate the cost of the fill required to be \$75,000. You also perform an analysis for incremental infrastructure cost for Phase II and estimate the cost to be \$60,000. You check with the local jurisdiction and find that the additional units in Phase II will increase traffic volumes on the primary access route, which also serves Phase I. The increase in traffic volume will require the road to have a wider cross section and you estimate the incremental cost of the widening to be \$20,000. The total incremental infrastructure cost for Phase II is \$80,000.

Next, you estimate the additional cost for the deeper sewers in Phase I. You have been told by the developer's contractor that he would estimate the cost to be \$2000/unit for 25% of the Phase I units or 15 units. This results in an additional cost for Phase I of \$2000/unit × 15 units = \$30,000.

You now compare the total incremental costs to the required profit:

Additional cost for fill (Phase II) = \$75,000  
 Additional cost for infrastructure (Phase II) = \$80,000  
 Additional cost for infrastructure (Phase I) = \$30,000  
 Total cost = \$185,000  
 (31.4)

You now prepare a report to your client indicating that, based on your assumptions, he will not achieve the required profit since the total incremental cost exceeds the required profit by \$5,000. You also advise that any unforeseen costs not included in your evaluation will make the differential even less appealing. You document your findings and assumptions clearly in the report to the client and include a copy of the report in the project files.

**Specifics of Site**

Development costs can vary greatly due to the specifics of the site. The provision of services, particularly utility services, can cause a wide disparity in costs between sites. If potable water, trunk sanitary, and/or storm sewer systems are not readily available, the provision of connections to these utilities can greatly increase construction costs for a project. The reconstruction or widening of a major highway can involve substantial construction costs, which are attributable to the project and which can have a serious impact on the per unit costs.

**TABLE 31.3 Procedure for the Preparation of Cost Estimates**

All cost estimates shall be prepared by a *senior engineer* who shall, prior to beginning any quantity and cost estimate, undertake the following.

1. Secure the plan that is to be used as the basis for the estimate, and *clearly reference* a set of the plans as the source material upon which the estimate is predicated.
2. Research files and senior company engineers to determine what jobs, if any, have been done in the vicinity of the subject tract.
3. Obtain soil maps or reports of the tract under consideration, if available.
4. Conduct site inspection by physically walking site, not just a “ride-by.”
5. Use a standard cost estimate checklist and itemize each quantity or clearly state why quantity is not defined.
6. Secure from reliable source (reputable contractor, current bids, contractor's data report, recent cost estimates) unit costs for all improvements required.
7. Verify that costs secured are installed/finished construction prices.
8. Check all prices with another senior engineer to obtain concurrence on prices used.
9. Prepare chart-form cost estimate detailing type of improvement, quantity, units, unit cost, and total cost.
10. Have addition and multiplication cross-checked by other qualified person.
11. Prepare a cover letter to accompany all cost estimates, making the cost estimate an integral part of the letter (i.e., number of all pages—1 of 4, etc.).  
In this way the cost estimate should not get separated from the cover letter that qualifies the estimate.
12. The cover letter should enumerate that only those items listed in the cost estimate were considered. Additionally, it should detail those items that were not considered and why.
13. All cost estimates, regardless of how they are prepared, should qualify soil conditions and their possible effect on development costs.
14. All cost estimates must clearly define source base plan used for estimate.
15. All cost estimates must show preparer of same and the date estimate was prepared, irrespective of closing signature.
16. All cost estimates should be checked by senior staff prior to being sent to client.
17. All cost estimates should clearly state that costs are based on present-day dollars.
18. All cost estimates should have rounded totals as final figure, which should include a 10% contingency factor added in at the end of the estimate.
19. All cost estimates should state the source of unit costs applied to the quantities.
20. Prepare estimate in the order of construction events.
21. Include base plan with cost estimate and letter.

All cost estimates shall be prepared by a *senior engineer* who shall, before beginning any quantity and cost estimate, undertake the following:

If earthwork balance of cuts and fills cannot be achieved, fill must be borrowed from an off-site source or excavation and embankment material must be exported. These earthwork transportation costs can seriously affect the economic viability of the site. Errors in estimating earthwork balance can lead to misleading profit percentages, so care must be taken to estimate these costs as accurately as possible. The quantity-takeoff estimate has become much more accurate with development of a series of proprietary software packages, which allow for electronic compilation of quantities from the topographic files.

The political climate, particularly with respect to neighboring parcels, should not be underestimated. Costs to protect adjacent neighborhoods using extensive landscaping, sound berms, or walls, can add to the project cost significantly. If possible, these costs should be estimated and included within this early feasibility-cost estimate.

### **Soils (Rocks)**

The type of soils found on the project site can affect project construction costs. The types of soil can cause significant costs because of the possible need to change the existing soil characteristics. If a significant portion of the soils cannot be made compatible with the proposed development program, the costs for replacing the existing soils can be

extreme. Excavation and the provision of sufficient suitable backfill can also increase costs.

The presence of rock on the proposed site will also have a tremendous effect on the construction cost. If excavation of rock is required, blasting may be required.

As one can see, the development of a feasibility-cost estimate can be very time-consuming. Care must be taken, however, not to over-detail the estimate at this early stage of design. Actual costs, of course, will not become known until the project development is completed. The estimation of project costs as completely and accurately as possible is essential to the client and to his decision to continue planning and designing the project. The identification of potential unexpected costs is also critical in order to allow for alternative planning.

### ACCURACY VERSUS COST FOR ESTIMATES

It is generally felt that the accuracy of the estimate should bear some relationship to the cost of preparing that estimate. While little hard research has been performed on this topic; however, the following graph and related material might prove useful in determining some guidelines.

Parks and Jackson, in the book entitled *Cost Engineering Analysis*, show the relationship presented in Figure 31.3. This figure documents the relationship between the anticipated accuracy of the estimate as a function of the cost of the estimate, expressed as a percentage of the total project cost. Generally, the estimate for a higher-cost project would

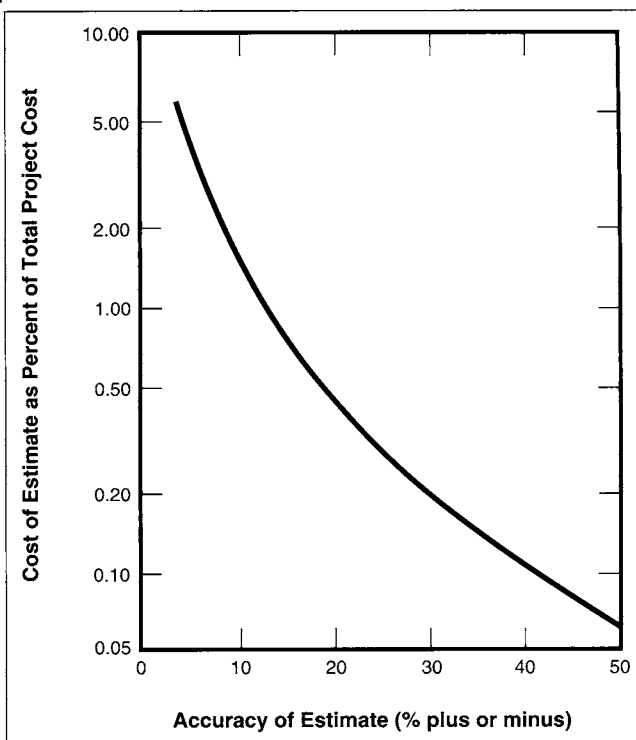
require a lower expenditure per project dollar for the cost estimate to achieve the same level of accuracy, with all other things being equal. If the engineer is preparing a bid estimate where the accuracy should be  $\pm 5\%$ , Figure 31.3 would show that the cost estimate would be about 4% of the total project cost (or \$40,000 for the \$1 million project). A preliminary estimate, as previously indicated, should be accurate within 10 to 15%. According to Figure 31.3, the cost associated with preparing an estimate that is accurate to 10% of the cost is 1.5% of total project cost (or \$15,000), while an estimate accurate to within 15% would require 0.8% of the total project cost (or \$8000). For a feasibility-cost estimate, which may be accurate to 25%, Figure 31.3 indicates that 0.4% of the total project cost is required (or \$4000). For \$600, one can expect a ballpark estimate accurate to about 50%.

### SOURCES OF UNIT-COST DATA

As important as quantities and contingencies are to cost estimating, the accuracy of the cost estimate also depends heavily on the accuracy of the unit-cost data. When providing unit-cost data for an estimate, it is recommended that the engineer check first with the client-owner (or developer) to determine if a relationship has been established with a reliable local contractor. If one exists, the contractor can provide information on recent bid data on similar projects, which is the best method of estimating unit costs. In some areas, local construction companies form trade associations to evaluate common problems and to promote their work. Frequently, these groups publish listings of recent successful bids. There are also nationally published bid tabulations such as the Bid Reporter and the Contractors Data Report. Many state departments of transportation publish recent bid tabulations, from which general information on unit costs can be obtained.

Local public works agencies also can provide information on recent costs. Care must be exercised when using this information, since the construction costs for a public project may be different from those for privately constructed projects. An example of bid tabulation from a local agency is shown in Figure 31.4. In addition, many local governments require performance bonds for private projects. The purpose of a performance bond is to set aside sufficient funds in an escrow account to protect the public if a private developer/contractor fails to complete the project. The funds are then utilized by the public agency to finish the work. Many public agencies publish a unit-price list to be used for the performance bond estimate. An example is shown in Figure 31.5. Care must be exercised by the engineer in using such information, as performance-bond unit prices generally are higher than private construction costs, since additional amounts are included to reflect administration and start-up expenses when the agency must step in to complete a project.

In addition, there is published information generally identified as "Guides to Construction Costs." These include:



**FIGURE 31.3** Accuracy versus cost for estimates. Park, W. and D.E. Jackson, *Cost Engineering Analysis*, 2nd ed., New York: John Wiley & Sons. Used by permission.

DESCRIPTION	QTY	MEAS.	Company A		Company B		Company C		Company D		Company E	
			UNIT PRICE	TOTAL	UNIT PRICE	TOTAL	UNIT PRICE	TOTAL	UNIT PRICE	TOTAL	UNIT PRICE	TOTAL
Clearing & Grubbing	1	LS	2,000.00	2,000.00	2,500.00	2,500.00	3,280.00	3,280.00	2,000.00	2,000.00	1,000.00	1,000.00
Construction Stakeout	1	LS	2,500.00	2,500.00	1,500.00	1,500.00	1,000.00	1,000.00	2,000.00	2,000.00	1,500.00	1,500.00
Maintenance of Traffic	1	LS	2,500.00	2,500.00	5,000.00	5,000.00	3,000.00	3,000.00	4,000.00	4,000.00	5,000.00	5,000.00
Temp. Concrete Barrier for M.O.T.	140	LF	10.00	1,400.00	15.00	2,100.00	12.00	1,680.00	20.00	2,800.00	40.00	5,600.00
Traffic Drums	50	EA	25.00	1,200.00	85.00	4,250.00	55.00	2,750.00	30.00	1,500.00	50.00	2,500.00
Temp. Pavement Tape Marking	1,020	LF	1.40	1,428.00	1.60	1,632.00	0.60	612.00	1.50	1,530.00	2.00	2,040.00
Class 1 & 2 Excavation	638	CY	15.00	9,570.00	18.00	11,484.00	20.00	12,760.00	22.00	14,036.00	39.00	24,882.00
Select Borrow	100	CY	15.00	1,500.00	5.00	500.00	5.00	500.00	22.00	2,200.00	10.00	1,000.00
Silt Fence for Sediment Control	600	LF	1.50	900.00	1.50	900.00	0.01	6.00	1.00	600.00	2.00	1,200.00
Test Pit Excavation Cont.	40	CY	50.00	2,000.00	10.00	400.00	35.00	1,400.00	50.00	2,000.00	5.00	200.00
Stabilized Construction Entrance	50	SY	25.00	1,250.00	11.00	550.00	11.00	550.00	15.00	750.00	10.00	500.00
Strawbales / Sandbags (Sed. Control)	50	EA	5.00	250.00	5.00	250.00	5.00	250.00	1.00	50.00	3.50	175.00
Temp. Curb Inlet Protection	3	EA	200.00	600.00	200.00	600.00	125.00	375.00	150.00	450.00	75.00	225.00
Select Borrow for Trench Backfill	25	CY	40.00	1,000.00	5.00	125.00	22.00	550.00	24.00	600.00	10.00	250.00
15" RCP Class V	87	LF	35.00	3,045.00	83.00	7,221.00	60.00	5,220.00	35.00	3,045.00	60.00	5,220.00
18" RCP Class V	14	LF	50.00	700.00	87.00	1,218.00	55.00	770.00	60.00	840.00	60.00	840.00
Bituminous Concrete Base	162	TN	45.00	7,290.00	45.00	7,290.00	50.00	8,100.00	70.00	11,340.00	50.00	8,100.00
Bituminous Concrete Surface	126	TN	47.00	5,922.00	50.00	6,300.00	55.00	6,930.00	70.00	8,820.00	60.00	7,560.00
Aggregate Subbase for M.O.T.	50	TN	50.00	2,500.00	15.00	750.00	15.00	750.00	10.00	500.00	10.00	500.00
Calcium Chloride for M.O.T.	2	TN	250.00	500.00	300.00	600.00	350.00	700.00	50.00	100.00	200.00	400.00
Bitumin. Conc. (Cold Mix) M.O.T.	25	TN	50.00	1,250.00	45.00	1,125.00	50.00	1,250.00	50.00	1,250.00	30.00	750.00
Pavement Profiling (Milling)	552	SY/I	3.00	1,656.00	4.00	2,208.00	5.00	2,760.00	5.00	2,760.00	4.50	2,484.00
Poly. Paving Fabric for Bit. Conc.	120	SY	10.00	1,200.00	1.75	210.00	2.00	240.00	3.00	360.00	2.00	240.00
5" Conc. Sidewalk & S/W Ramps	185	SY	27.00	4,995.00	24.00	4,440.00	27.00	4,995.00	20.00	3,700.00	22.50	4,162.50
TOTALS				57156		63153		60428		67231		76328.5

FIGURE 31.4 Bid tabulation example.

ARLINGTON COUNTY  
DEPARTMENT OF PUBLIC WORKS  
PUBLIC IMPROVEMENT PERFORMANCE BOND ESTIMATE

SUBDIVISION/SITE PLAN NAME \_\_\_\_\_  
ADDRESS/LOCATION \_\_\_\_\_  
SUBMITTED BY \_\_\_\_\_ DATE \_\_\_\_\_

Item	Quan.	Unit	Unit Price	Cost	Item	Quan.	Unit	Unit Price	Cost
Excavation including Clearing & Grubbing		Cu. Yds.	\$23.75		Trees on Public R/W		Each	\$330.00	
Conc. Curb & Gutter		Lin. Ft.	\$15.00		Tree Grates		Each	\$700.00	
Conc. Drive Entrance		Sq. Yd.	\$35.00		Street Lights Type - Thoroughfare		Each	\$6,000.00	
4" Conc. Sidewalk		Sq. Yd.	\$25.50		Street Lights Type - Colonial		Each	\$4,500.00	
Brick Paver Sidewalkon Aggregate Base		Sq. Yd.	\$46.25		Restoration: Sod, etc.		Lump Sum		
Brick Paver Sidewalk on 4" Conc. Base		Sq. Yd.	\$60.50		Erosion/Silt Controls		Lin. Ft.	\$6.00	
Brick Paver Sidewalkon 6" Conc. Base		Sq. Yd.	\$70.00		CB-2 Catch Basin		Each	\$2,275.00	
Brick Paver Crosswalkon 6" Conc. Base		Sq. Yd.	\$140.00		Catch Basin (misc.)		Each		
Wheel Chair Ramps		Each	\$315.00		CB-2 Top Remove & Replace		Each	\$1,225.00	
Bike Trail 4" Asphalt with 6" Comp. Subbase		Sq. Yd.	\$17.00		MH-1 Storm Sewer Manhole		Each	\$2,525.00	
Subbase - C.B.R. 30		Cu. Yd.	\$29.00		MH-2 Storm Sewer Manhole		Each	\$3,300.00	
B-3 Bituminous Concrete Base		Ton	\$57.00		Storm Sewer Manhole (misc)		Each		
S-5 Bituminous Concrete Top		Ton	\$60.00		MH-1 Top Remove and Replace		Each	\$1,000.00	
Remove Conc. Curb & Gutter		Lin. Ft.	\$7.00		MH-2 Top Remove & Replace		Each	\$1,250.00	
Remove Conc. Sidewalk		Sq. Yd.	\$5.00		15" R.P.C. Cl. III Pipe		Lin. Ft.	\$44.00	
Remove Conc. Drive Entrance		Sq. Yd.	\$6.00		18" R.P.C. Cl. III Pipe		Lin. Ft.	\$48.00	
Conc. Retaining Wall		Cu. Ft.	\$19.50		24" R.P.C. Cl. III Pipe		Lin. Ft.	\$63.00	
Stone Retaining Wall		Cu. Ft.	\$12.50		R.P.C. Cl. III Pipe		Lin. Ft.		
Storm Water Detention System		Lump Sum							

FIGURE 31.5 Arlington County performance bond unit prices.

Item	Quan.	Unit	Unit Price	Cost	Item	Quan.	Unit	Unit Price	Cost
4" Water Line D.I.P. Class 52		Lin. Ft.	\$45.00		8" San Sewer Pipe Less than 10' Deep		Lin. Ft.	\$55.00	
6" Water Line D.I.P. Class 52		Lin. Ft.	\$51.00		8" San Sewer Pipe Greater than 10' Deep		Lin. Ft.	\$160.00	
8" Water Line D.I.P. Class 52		Lin. Ft.	\$55.00		15" San. Sewer Pipe Less than 10' Deep		Lin. Ft.	\$80.00	
12" Water Line D.I.P. Class 52		Lin. Ft.	\$64.00		15" San. Sewer Greater than 10' Deep		Lin. Ft.	\$210.00	
16" Water Line D.I.P. Class 52		Lin. Ft.	\$78.00		4' I.D. San. Sewer M.H.		Vert. Ft.	\$300.00	
6x6 Tapping Sleeve and Valve		Each	\$2,200.00		5' I.D. San. Sewer M.H.		Each	\$350.00	
8x8 Tapping Sleeve and Valve		Each	\$3,150.00		San M.H. Adj. to Grade		Each	\$350.00	
12x12 Tapping Sleeve and Valve		Each	\$4,150.00		Tunneling 24" Carrier Pipe		Lin. Ft.	\$750.00	
16x12 Tapping Sleeve and Valve		Each	\$4,068.00		Traffic Signalization*		Lump Sum		
24x12 Tapping Sleeve and Valve		Each	\$8,050.00						
2' Blow Off Valve		Each	\$1,500.00						
Fire Hydrant		Each	\$1,325.00						
Fire Hydrant (relocate)		Each	\$1,000.00						

\*Contact the Department of Public Works Traffic Signal Engineer on 358-3575 for cost if required by this subdivision/site plan.

ADDITIONAL ITEMS

SUB-TOTAL \_\_\_\_\_  
 20% ADMINISTRATIVE, ENGINEERING & MISC. FEE \_\_\_\_\_  
 TOTAL QUOTE \_\_\_\_\_  
 REVIEWED AMOUNT APPROVED BY ARLINGTON COUNTY (FOR COUNTY USE ONLY) \_\_\_\_\_

BY \_\_\_\_\_ DATE \_\_\_\_\_

FIGURE 31.5 (Continued) Arlington County performance bond unit prices.

“Building Construction Data,” published by the R.S. Means Company; the “Dodge Reports,” published by the F.W. Dodge Company; and “Engineering New Record,” published by McGraw-Hill.

Other sources for cost guides include specialized trade associations such as the American Concrete Pipe Association and the Iron and Steel Institute, among numerous others. All of these guides are only that—the unit prices should never be used blindly since conditions vary from region to region, and in fact, from project area to project area within the same jurisdiction.

## ENGINEERING ECONOMY

Occasionally, the land development engineer is asked to assist the client in evaluating a series of alternatives (value engineering), which may involve a type of decision-making called engineering economic analysis. The intent of presenting the following information is not to provide a complete discourse on the subject but to summarize the formulas used in textbooks on the subject. The reader is directed to any of the engineering economy texts listed in the bibliography for a full explanation of the subject, as well as the necessary tables to apply the principles involved.

### Interest Formulas

While texts on engineering economics use different symbols, the following are the ones most commonly used and are taken from a well-respected text on the subject (*Principles of Engineering Economy*, Grant, E.L. et al. 1982).

#### Symbols

$i$  = interest rate per interest period

$n$  = number of interest periods

$P$  = Present sum of money

$F$  = Sum of money at the end of  $n$  periods. It is equivalent to  $P$  with interest  $i$  added.

$A$  = End of period payment or receipt in a uniform series continuing for  $n$  periods.

**Formulas.** The following information is provided to enable one to utilize the interest tables quickly when calculating the information needed to analyze engineering economy problems.

$$F = P(1 + i)^n \quad (31.5)$$

$$P = F \left[ \frac{1}{(1 + i)^n} \right] \quad (31.6)$$

$$A = F \left[ \frac{i}{(1 + i)^n - 1} \right] \quad (31.7)$$

$$A = P \left[ \frac{i(1 + i)^n}{(1 + i)^n - 1} \right] \quad (31.8)$$

$$F = A \left[ \frac{(1 + i)^n - 1}{i} \right] \quad (31.9)$$

$$P = A \left[ \frac{(1 + i)^n - 1}{i(1 + i)^n} \right] \quad (31.10)$$

The factors, which vary by interest rate and number of periods, are found in most engineering economic analysis texts. They are usually written as (Factor,  $i$ ,  $n$ ). For example, the uniform series present worth factor at  $i = 7$  and for  $n = 10$  would be written as ( $PWF$ ,  $i$ ,  $n$ ) or ( $PWF$ , 7%, 10), and is equal to 0.5083. This factor can be determined by looking at a table or by substituting the values into Equation 31.5. When solving complex engineering economics problems, it is a good idea to draw a cash-flow diagram. A simple problem will illustrate.

**EXAMPLE 4:** How much does one have to invest today to be able to achieve draws of \$2000 at the end of each year for 5 yr, assuming interest is at 7% per year?

First, prepare a cash flow diagram (Figure 31.6). To find  $P$  given  $A$  at  $n = 5$  and  $i = 7$ ,

$$P = A (\text{Present Worth Factor}) \quad (31.11)$$

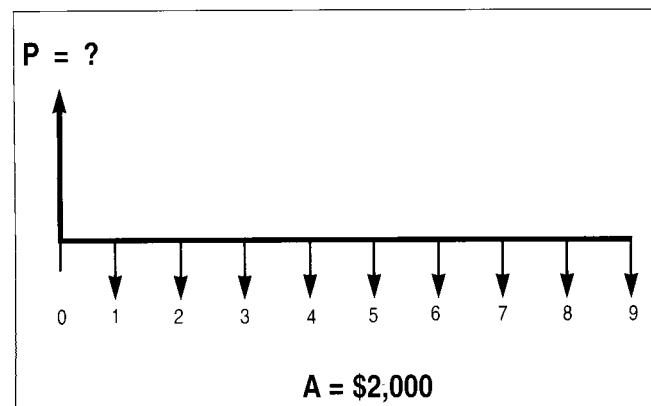
$$= A (PWF, 7, 5)$$

$$= 2000 (4.100)$$

$$P = \$8200$$

Example 5, below, is included as an example of alternative analyses that are commonly encountered in land development engineering. It uses the principles of engineering economics as described previously.

**EXAMPLE 5:** You are the engineer for a major landowner and developer. Your client needs to construct a storm drainage facility that extends under a state roadway. Your client has requested that you compare two alternative storm drainage pipe systems



**FIGURE 31.6** Cash-flow diagram for Example 4.



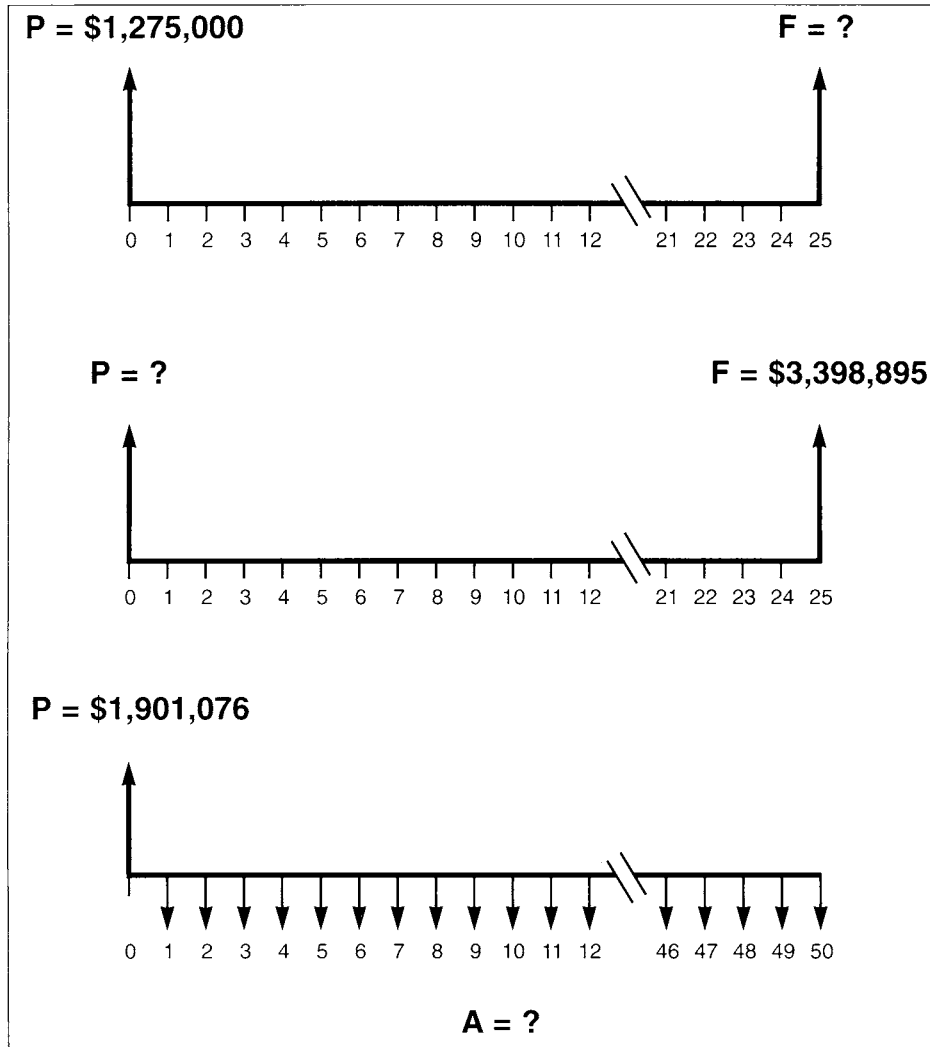


FIGURE 31.7 Cash-flow diagram for Example 5.

and has provided you with two bids from his contractor. Alternative A has a service life of 50 yr and a bid cost of \$1,575,800. Alternative B has a service life of 25 yr with a bid cost of \$1,275,000. Since your client will be the owner over the life of the facility, he wants to be sure he selects the least-cost alternative over the long-term.

You, as the engineer, investigate and find that the state highway department requires a service life of 50 yr, meaning that Alternative B could require replacement once during the service life of the project. You are aware that inflation is at 4% and interest rates are at 7%.

Assumptions/Facts	Alternative A	Alternative B
Bid Cost (P)	\$1,575,800	\$1,275,000
Life (n)50 years	25 years	
Interest (i)	7%	7%
Inflation	4%	4%

Your client has asked you to compare the life cost for each alternative on an annual cost basis.

For Alternative A, using the economic analysis equation for annual equivalent cost (see Engineering Economics Tables):

$$A = P \left[ \frac{i(1+i)^n}{(1+i)^n - 1} \right] = P(CRF, 7\%, 50 \text{ yr})$$

$$= 1,575,800 (0.07246)$$

$$A = \text{Annual Cost}_{\text{ALT A}} = \$114,182 \quad (31.12)$$

In Alternative B, the pipe is assumed to need replacement in 25 yr, since the service life is 25 yr. Here you assume, at an inflation rate of 4%/year, the cost to replace in 25 yr is:

$$S_{25} = P(1+i)^n = P(CAF, 4\%, 25 \text{ yr}) \quad (31.13)$$

$$= 1,275,000 (2.6658)$$

$$S_{25} = \$3,398,895$$

Next, find the present value of \$3,398,895 using

$$i = 4\%$$

$$\begin{aligned} P &= S \left[ \frac{1}{(1+i)^n} \right] = S (PWF, 7\%, 25 \text{ yr}) \\ &= 3,398,895 (0.1842) \\ &= \$626,076 \end{aligned} \quad (31.14)$$

Adding this value to the first installation cost you find,  $\$1,275,000 + \$626,076 = \$1,901,076$  for the total present value of Alternative B. Since you have been asked to compare the alternatives on an annual cost basis, you calculate:

$$\begin{aligned} A &= P \left[ \frac{i(1+i)^n}{(1+i)^n - 1} \right] = \quad (31.15) \\ &= P(CRF, 7, 50 \text{ yr}) \\ &= 1,901,076(0.07246) \end{aligned}$$

$$A_{\text{ALTB}} = \$137,752$$

Figure 31.7 is a diagram that shows Alternative B cash flows for each step.

Note that the whole computation process of finding the annual cost could have been done in one step:

$$\begin{aligned} A &= (P(CAF, 4\%, 25))(PWF, 7\%, 25)(CRF, 7\%, 50) \\ &\quad + P(CRF, 7\%, 50) \end{aligned} \quad (31.16)$$

Based on the above assumptions on an annual equivalent cost basis, Alternative A is \$114,182/yr and Alternative B is \$137,752/yr. In this example, no cost of removing the pipe at 25 yr is included. You advise your client that Alternative A, with the higher current bid cost, is the least-cost solution over the life of the facility (50 yr). You prepare a report to your client documenting your analysis and assumptions and put a copy in the project files.

#### REFERENCES

- Grant, E.L., et al., *Principles of Engineering Economy*, John Wiley, New York, 1982.
- Park, W.R., and D.E. Jackson, *Cost Engineering and Analysis*, John Wiley, New York, 1982.



## PART VI

# PLAN SUBMISSION AND PERMITTING



## CHAPTER 32

# SUBDIVISION SUBMITTALS

*Lawrence A. McDermott*

### INTRODUCTION

Approval of a rezoning application is insufficient to initiate construction. Detailed design and structural analyses are rarely incorporated to a sufficient degree in the rezoning process. Consequently, the development team can encounter numerous pitfalls in attempting to translate an approved zoning plan into a subdivision submittal. Engineering analysis performed for a subdivision sometimes suggests that the assumptions used in the rezoning submittal were inaccurate if not incorrect. This is because of the level of detail that each type of approval requires. The risk and expense involved in achieving this accuracy with the more speculative rezoning application may outweigh the benefit of greater exactness at that point in the process. Yet, some jurisdictions go so far as to require revisions to the approved development plans with the rezoning because of problems encountered during the subdivision analysis, and this can substantially alter the approved development plan. As with so many aspects of development review, the hesitancy to allow administrative corrections stems from fear of abuses and claims of unequal treatment.

The land development process is often lengthy and arduous. Previous chapters have discussed, in detail, comprehensive planning, zoning, subdivisions, building permits and a myriad of other elements of the development process. This chapter focuses on the subdivision submittal, review and approval process. It is at this point in the process that the developer is preparing to initiate construction and welcomes this critical element.

### PREPARATION AND PROCESSING

The subdivision submittal is the developer's actual representation of the proposed development. Using drawings

and other information, the submittal shows the arrangement of property lines, lots and other features that will occupy the site. The subdivision plat identifies the public and private facilities that are to serve the intended uses as well as information on how those facilities will be constructed, from grading of the property to connection of structures to water and sewer lines. The subdivision application is prepared, submitted and reviewed in a predetermined sequence of increasing accuracy and detail. Most jurisdictions consider subdivision approvals as an administrative act of the staff, without involvement of the legislative body or the public. However, even the most prescriptive ordinances allow and necessitate considerable judgment by development professionals concerning on-site layout and connection to off-site facilities.

Prior to expending significant time and money for the preparation of final layouts and construction plans, the developer seeks a comfort level that plans will be approved. By incorporating a series of checkpoints throughout the plan-design process, the development team is able to predict whether its interpretations of local policies, regulations and standards match those of local government authorities. Without this monitoring procedure, extensive redesign would be needed if, for instance, the lot layout did not comply fully with the requirements of the current zoning ordinance or the street and utility systems did not consider planned or existing networks properly.

The agency taking the lead role in performing subdivision plan review varies from jurisdiction to jurisdiction. In larger communities, the planning or public works department is given this responsibility. In some instances, a multi-purpose agency established to administer building and environmental regulations is sometimes created for this pur-

pose. In small communities, an individual such as the town manager or engineer may be responsible for the subdivision review process. The designated review agency also is generally responsible for coordinating the review of plans by other governmental and quasi-governmental bodies.

In a typical developing jurisdiction, the lead agency distributes copies of the subdivision plan to other agencies for review. Most agencies are asked to review submittals to identify the location, adequacy and availability of services and assess impact on systems for which they are responsible. These agencies will also review plans for compliance with ordinances and standards that they may administer. Their role is to ensure proper relationship between systems constructed as part of the new development and the community's existing and proposed patterns and networks. Some agencies receive copies to assist in monitoring developing activity, thereby aiding in planning and budgeting for manpower and facilities to service future users. Local ordinances frequently define the reporting relationships for this review, specifying whether agency comments are advisory or mandatory to the lead department. The distinction is important to the development team, as the different departments are responsible for individual programs that have varied and sometimes conflicting priorities. It is important to know who has the authority to resolve conflicts when competing demands confront the developer.

Agencies involved in rezoning review are also involved in subdivision review. While the reviews are similar, the focus is more detailed and is concerned with design, sizing, placement and connection.

- Long-range planning generally determines whether the street and system layout adheres to development patterns proposed in the comprehensive plan.
- Personnel in zoning administration will determine compliance with zoning ordinance requirements, such as lot size and density requirements. In addition, the submittal will be reviewed for compliance with the design guidelines and standards in the subdivision regulations for street and lot layout, environmental protection and other planning issues. Environmental reviews are often performed by personnel in both sections, as well as in other departments.
- The public works department typically is responsible for streets, storm drainage facilities, sewer and water systems. Other elements of a development may also fall within the department's purview, such as placement of survey monuments and markers. However, in some communities, streets may come under the jurisdiction of a separate transportation department or a state-level transportation agency. Similarly, water and sewer systems may be run by a regional authority serving several jurisdictions. A drainage district may also have a separate level of authority. A local or state health department can also have regulations concerning on-site

and off-site sewage disposal and the placement of septic drainfields and wells.

- Fire protection officials are frequently involved in the review of subdivision proposals to ensure that clear and adequate access is provided for emergency vehicles. Placement of water lines and fire hydrants may also fall under their purview, as well as the monitoring of development activity to assist in locating future station facilities.
- School districts typically perform an analysis of potential school enrollment generated by new development as a monitoring function.
- A local parks and recreation department will review subdivision proposals to evaluate recreational needs of both the new development and impact on existing facilities. In some jurisdictions, the subdivision regulations contain requirements for the provision of such facilities; however, such extractions usually take place during the rezoning process. This department may also be responsible for preservation and acquisition of important environmental areas, such as floodplains and wetlands, the control of which may be included in subdivision regulations.
- A building department typically is responsible for reviewing grading, construction and erosion and sediment-control plans that are included as part of the submittal package. This department will also be involved in project inspection as the project proceeds through the construction phase.
- The local gas, electric and telephone companies will be involved in confirming system adequacy, connections, the placement of utility easements and coordination of construction with street, water and sewer contractors.
- The soil conservation district or similar entity may review grading, storm drainage and other environmental issues.

Many communities have established a three-phase review process—sketch plan or pre-application preliminary plan and final plan. The purpose, focus of design and review, and requirements for each submission differ considerably. However, each of these phases is conducted in much the same manner. Plans must conform with ordinance specifications for the particular phase concerning detail, required information, scale, number of copies and supporting documentation. The package is presented to the coordinating office for distribution to the agencies that will conduct review. In some jurisdictions, a technician will check the submittal to ensure that it is complete, that is, has all the necessary information and studies called for in the ordinance. The importance of this step is discussed later in this chapter.

The coordinating agency will forward plans to the agencies on its distribution list, which may be set by ordinance or by office policy. In some municipalities, courtesy copies are sent to elected or appointed officials who represent the area in which the project is located. Public notice may or may not be required. If notice is required, as with rezoning applications, it may be accomplished by mailing notices to adjoining property owners and posting signs on the property describing the application. Each agency that receives a package is asked to review the submittal and return comments or a marked plan by a certain date. In some municipalities, the coordinating agency is required by law to complete its review and render a decision within a specified time period. In such a case, the agency would ask that these comments from other agencies be returned to allow sufficient time to comply with the mandated deadline.

Upon receipt of the comments, the coordinating agency typically meets with the applicant to discuss its own and other agency findings with appropriate staff in attendance. This meeting provides a forum for the land development team to explain the rationale behind its proposal and an opportunity to resolve conflicts between its design and agency requirements through negotiations or revision of the plan. One of three possible decisions may be rendered after the meeting. One, an approval means that the applicant is authorized to proceed to the next step and carry plan design to the next level of detail. Two, a conditional approval suggests that the plan is essentially acceptable, but modifications must be made prior to taking the next step. This poses some risks to the development team, since the modifications will be designed in the higher level of detail, without the benefit of the government's conceptual approval. Three, a denial means that the development team must reevaluate its proposal, consider the impact of the agency requirements and, upon redesign, resubmit the application at the same phase.

In some communities, an expedited procedure has been established for processing revised plans after a decision is issued. Often within a discrete time period, the land design team may be permitted the opportunity to work directly with an agency that had problems with the original submittal. Once the issues are resolved and the agency has notified the coordinator of its approval, the design process can move on to its next step. In communities with a large submittal work load, the availability of this process can save time for the developer. If not for this process, it would be necessary to enter the review process at the "bottom of the pile." The result of starting anew in the process is significant time delay which is both costly and frustrating to the developer.

### **Partial and Incomplete Submissions**

Communities that make a point of checking for completeness before accepting submissions do so for two reasons: (1) fairness to other developers; and (2) reduction of their own increasing workload. Under pressure to get submittals

into and through the community's development review process quickly, developers and their consultants often found it convenient to submit partially completed applications. Knowing that an existing backlog of submittals might mean a several-week delay in review, this practice provides the development team additional time to perform or finish a needed analysis or design. By virtue of the date of submittal, the plan is guaranteed an earlier review than plans submitted subsequently, even if the latter plans were complete. The practice often meant that purchase-contract or loan-commitment requirements could be fulfilled or that construction crews could proceed with minimal interruption.

In some instances, the practice insured that a plan would be effectively "grandfathered" from a new regulation or policy. In some jurisdictions, a plan that is designed using the requirements of one regulation often must be redesigned to comply with a subsequently adopted regulation. If that regulation is to become effective in the near future, the development team may need to demonstrate that some progress has been made through the review process. Redesign to insure compliance with the new rules would cause delays and substantive additional costs. Plans that are formally submitted "and accepted" by a certain specified date are often so "grandfathered." Hence, there is the rush to get a plan into the development review pipeline.

Last, some communities have found that the practice of submitting incomplete applications is an attempt to elicit design assistance from the review staff. Knowing that review personnel in various departments would clarify or resolve critical design issues, the development team would submit only partial solutions. Often, this land development equivalent of "run it up the flag pole and see how it flies" led to early abandonment of projects deemed too complex. Meanwhile, local government manpower and resources were tied up with frivolous projects, when their efforts could have been better spent on more serious proposals. Except where it is permitted and encouraged, the land design team should avoid the practice of making incomplete submittals. Some jurisdictions have institutionalized a partial-design phase to assist the development community in improving design quality. However, in most cases, the practice damages the reputation of all members of the design team, leading government reviewers to be skeptical of future proposals.

Fees are usually attached to the review process. The fee can be a flat fee for each type of plan or study that is submitted; or, the fee can be based on the acreage, number of lots or building floor area, linear feet of roads, sanitary sewers, storm sewers and water lines, cost estimates of public improvements or some other indicator. These fees, in part, pay for the government agencies' administrative costs in performing submittal reviews. Practice also varies as to when the fees are collected. Some jurisdictions require fees to be paid with each submission phase, while others require an upfront estimate of the entire development process and maintain an actual accounting of time spent throughout the life of project. Reimbursement of excess fees may be due



the developer, or additional payment may be required at project completion.

### Addressing Review Comments

Project plans are distributed to the various agencies for review and a determination as to whether they are in compliance with all appropriate codes. In addition to narrative comments, review agencies often write comments directly on the plan documents. Following all reviews, the plans are returned to the engineer for corrections. The consultant's task is to determine those comments that are actual requirements directly imposed by regulation versus those that are recommendations. Sometimes, the comments that appear are simply statements of local ordinance or policy, often without identifying whether the particular submittal is flawed. The consultant addresses the comments either by revising the design and changing the drawings or initiating discussions with the reviewer. Good access and rapport should be established and maintained with the lead reviewer at all times. Some comments may appear inconsequential, but if each and every comment is challenged, the relationship and future projects on which the consultant and reviewer may have to collaborate may be jeopardized. However, where some of the comments received appear to be improper interpretations of the regulations, it may be necessary to seek a decision from a supervisor or other authority.

After the land design team has addressed the comments by all of the agencies, the plans are resubmitted for a second review. The resubmission gives reviewers assurances that the initial comments have been addressed adequately and that changes have been integrated properly into the original design. When all of the review agencies have had their comments addressed, the plans are authorized for the next phase of the process. Noted here is the extent of the liability of the municipality and the engineer who sealed the plans. Although the reviewing agencies have approved the plans, the liability of construction rests with the engineer and not the review agency.

### Waivers

A common feature in subdivision regulation is the waiver process. Not unlike the zoning variance, a waiver allows for deviation from ordinance requirements in the event that strict compliance would impose an unreasonable hardship upon the developer. However, the difference between a zoning variance and a subdivision waiver is substantial. The latter often can be granted by the official charged with administering the regulations as an administrative act, absent public hearings. Requests for waivers are reviewed concurrently by the same agencies reviewing the subdivision plan. Typical situations that often involve requests for waivers include sites with unusual shape or topography that might preclude strict compliance with certain provisions of the ordinance. Required improvements that might duplicate ex-

isting public or private facilities may be waived. In addition, small projects whose contribution to a public system may be isolated or insignificant also may be granted waivers. In such instances, a cash payment may be rendered in lieu of the improvement as a condition of subdivision approval.

In practice, some jurisdictions require that the development team identify needed waivers early in the review process. The presumption is that a thorough analysis of the project and applicable development regulations has been performed, and the need for waivers has already been determined. This is an effort to discourage government personnel from having to assume the designer or consultant's role. Interim and frequent waiver requests are said to disrupt the review process and hamper the efficiency of reviewers. The results are often duplicative reviews that seriously encumber staff resources.

In preparing waiver requests (an example is shown in Figure 32.1), the development team should clearly present supporting rationale for the request. The request should include the identification of each ordinance provision for which a waiver is required, the specific reasons why compliance is not practical or feasible, alternate solutions and the proposed solution. There is a particular concern that frequent waivers weaken the future enforceability of the ordinance; therefore, the development team must stress the uniqueness of the circumstances and the particular project for which the waiver is being requested. In approving the request, the official often has the latitude to impose additional conditions to ensure that the public's interest is protected. Payment of fees for waiver reviews may be required to offset the additional cost of analysis performed by agency staff.

### Durability of Approvals

As noted earlier, completing and securing approval for one phase of the development process typically means that the development team is entitled to proceed to the next stage. However, many jurisdictions place a limit on the length of time for which an approval remains valid. This is done to prevent subdivision projects from spanning several years and being "grandfathered" from new ordinances and laws. In the course of administering its development regulations, local government often identifies flaws or deficiencies in its regulations. When this happens, it usually takes steps to amend those regulations, with new regulations being adopted to protect the health, safety and welfare of community residents. Plans in process are often protected against changes imposed by subsequently-adopted regulations. By providing an expiration date on its approvals, the local government can minimize the number of projects that are exempt or "grandfathered" from new policies and regulations. Another motive for providing for the expiration of development approvals is to enable the jurisdiction to monitor more accurately its rate and patterns of development. If approvals did not lapse, it would be extremely difficult



BASIS FOR REQUEST (explain why interests of adjoining property owners will not be adversely affected by this request): \_\_\_\_\_  
 \_\_\_\_\_  
 \_\_\_\_\_  
 \_\_\_\_\_  
 \_\_\_\_\_  
 \_\_\_\_\_

Four copies of the proposed screening plan attached (include plant list, location and spacing of proposed landscaping and barrier) [ ] YES [ ] NO

If applicable, four copies of the rendering of the architectural facade attached (illustrate appearance of site providing screening from adjoining property) [ ] YES [ ] NO

Four copies of a typical letter of notification to an adjoining or affected property owner of the proposed waiver/modification and a list of the owners notified. (Letter shall include the County requirements, as well as the proposed waiver/modification of those requirements.) [ ] YES [ ] NO

Separate letter of justification attached. [ ] YES [ ] NO

APPLICANT/AGENT SIGNATURE \_\_\_\_\_ DATE \_\_\_\_\_

<u>Process Sequence</u>	<u>Approval</u>	<u>Disapproval</u>	<u>As-Noted</u>	<u>Initials</u>	<u>Date</u>
Urban Forestry	_____	_____	_____	_____	_____
Branch Chief	_____	_____	_____	_____	_____
Deputy Director	_____	_____	_____	_____	_____
Distribution					_____
		<u>Contact</u>		<u>Initials</u>	<u>Date</u>
District BOS Member Notified	_____			_____	_____

Comments:

(3/93)

FIGURE 32.1 (Continued) Site plan transitional screening waiver.

to make reasonable estimations of growth. Local government would be unable to budget properly for the provision of public services, facilities or manpower.

While the subdivision and other regulations establish time limits, they also often provide for automatic extensions as well as extensions for extraordinary circumstances. Consequently, the land development team must closely monitor the project calendar. If circumstances indicate that a submittal will be delayed or that a subsequent approval will not be feasible within the time available, the team should communicate these concerns to the appropriate officials. A failed opportunity for an extension may result in an expired plan and require a totally new submission. This could expose the plan to any and all regulations and ordinance changes approved since the plan was initially submitted. Compliance with new regulations could necessitate a costly redesign and project delay. Additionally, the developer and the land development team should be cognizant of laws requiring that approved developments be recommended or that approved subdivisions be recorded within a specified period of time.

### Role of the Public

In theory, and in most jurisdictions, the general public has no official role in the review and approval of subdivisions. However, as noted in Chapters 2 and 28, citizens are often successful in demanding and securing a role, even if only advisory, to their elected leaders. Notices of pending subdivisions are often sent by the administrative staff to interested civic groups, which are then invited to comment on the application. Since the provisions of the subdivision regulations are rarely free from interpretive discretion, citizen comments often serve to influence the review staff, as is the case in the rezoning process. This does not suggest that the development team must employ the same intensity of community relations as for rezoning. However, the team should consider the potential impacts on adjacent property owners and attempt to reconcile them with the developer's own requirements. In practice, those familiar with the community must judge the level of involvement that the residents will seek. In the long run, it may be worthwhile to incorporate them early in the process. Frequently, careful consideration of the road systems, additional screening, setback or other aesthetic devices are sufficient to eliminate the opposition. Opposition can become a major obstacle or result in delayed project approval; therefore, it is prudent to resolve conflict with neighboring property owners early in the process.

Some jurisdictions have effectively eliminated the subdivision of land by right and have incorporated its review as a special exception. Where this process has been implemented, the public's concerns have a much more direct impact, and the subdivision process takes on an added dimension more closely paralleling the discretionary review of a rezoning application. The greater weight accorded cit-

izen comments in a public hearing places a greater obligation on the development team to identify concerns early in the design process.

### Description of the Preliminary Plan

Where the sketch plan was presented to discuss ideas and identify problems, the preliminary plan could be considered as the first formal submission. In some jurisdictions, the two plans can be the same document. The only difference is that the sketch plan is not "review and approved" and the preliminary plan the official document. The preliminary plan is perhaps the most important as it represents the parameters by which the site will be engineered and construction plans will be prepared. This plan will be reviewed by the agencies for compliance with all applicable codes and ordinances as well as compatibility with surrounding areas. Whatever is approved as part of the preliminary plan must be adhered to throughout the entire life of the project. To change what is shown on the approved preliminary plan may require starting the time-consuming process from the beginning, and this may be very costly to the developer. The preliminary plan does not give the developer any right or privilege to begin construction operations on the site. It is merely a method of communication by which the developer shows the local government and other regulating agencies, in more accurate detail, how development will be accomplished.

The preliminary plan is for the entire project to be able to show relationships and ties to each section. The final site plan breaks it down to the individual sections, and the final site plan is governed by the preliminary plan. One controlling factor may be the statutory period for which the preliminary plan is valid. Regulations often specify the minimum area that can be sectioned from a larger project for preliminary review. In addition, each section, along with previously approved sections, is usually required to form a self-sustaining project in terms of regulatory compliance. That is, in the event that the developer delays or abandons future sections, those previously approved must satisfy ordinance and functional requirements. The coordinating agency often maintains an accounting of critical elements of the project that are intended to support its population. Regardless of regulation, the section should be selected so as to insure that access and infrastructure can be constructed without long connections to existing facilities.

The preliminary plan is drawn to a scale sufficient to show the necessary detail. A preliminary plan typically includes the following information:

- Soils, geology and topography, using field-verified topo
- Adjacent property owners, zoning and use
- The lot layout of the project, showing area and dimensions and other property line information

- Information regarding height, bulk and area requirements
- Street layout, including off-site improvements, showing centerline alignment, profiles, typical cross sections, turn lanes, intersections and entrances, sidewalks and trails, traffic-control signs and devices
- Proposed street names
- Location of existing utilities and easements
- Location of proposed public utilities, including sizing and alignment of water, sewer and fire hydrants
- Sizing and location of proposed major drainage systems, including outfall and drainage-area analysis, accurate floodplain delineation, pipe sizing, stormwater management ponds
- Proposed easements, both temporary and permanent, for access, utilities, construction and maintenance
- Information regarding buffering, landscaping, clearing limits and impact mitigation
- Septic drainfield and perc information, and well sites for projects on individual and package-treatment systems

### Description of the Final Subdivision Plan

As the details of the data are refined, discussions about the project can be held with more certainty as the project evolves from the concept stages to preliminary design to final design. The final-design stage is represented by the construction drawings and specifications for the subdivision created by the engineers of the land development design team.

Prior to the sale of lots or the construction of buildings, the subdivision record plat must be approved and recorded in the municipality's land records. Typically, jurisdictions require that the public utilities, storm drainage system, and street system must either be completed or a performance bond executed by the developer to guarantee the construction of these public facilities prior to approval to allow recordation of the plat. The developer must have approved construction plans and specifications in order to obtain construction permits, without which construction cannot begin.

The approved construction documents are part of the final subdivision plan. The construction-plan set details the way in which the project is to be constructed. The construction plans generally will include:

- Detailed plan and profile of the roads
- Detailed plan and profile of water and sanitary lines
- Detailed plan and profile of storm-drainage system

- Grading plan
- Erosion and sediment control plans
- Enumeration of the specifications to be used for construction
- Stormwater-management plans and profiles

The size and complexity of the project, along with the scale needed to show accurate detail sufficiently, can result in a set of drawings of considerable size. Care must be taken to identify clearly the proper relationship of the various drawings. The organization of these plans depends on the particular ordinance or agency policy, the land design team and the project itself. Generally, the plan consists of the following sheets:

- Cover sheet, which shows title of the project and identification numbers for the project and other general pertinent information, such as, a vicinity map, developer's name and address, zoning/density notes and tabulation, sanitary and watershed districts, private utility information, sheet index
- General notes and symbol legend sheet
- Plan sheets, with match lines, that show the layout of the lots, streets, utilities, storm-sewer system and, in some cases, location of buildings
- Profile sheets that show street and utility profiles when they are within the right-of-way
- Additional profile sheets that may show storm sewer, sanitary and water lines systems throughout the project
- Storm-sewer computations sheet and drainage-divide plan
- Special detail sheets that show construction details of non-standard items
- Erosion and sediment-control plan that shows how the erosion from the site will be controlled
- Landscaping plan, if required, with a schedule of the types of vegetation and trees that will be planted; information showing how trees will be planted and how existing trees will be protected during construction is also often required

In addition to the construction drawings, the final plan submittal includes the final subdivision plat. This plat establishes a public record for the project, by which property will be sold or leased, and identifies the easements around the public utilities necessary for the site. It is important for the record plat to reflect the information on the development plan and construction drawings, such as location of streets and easements. The record plat contains extensive notes with respect to ownership, dedication, completion of

public improvements and restrictions. These affect the chain of title, use and enjoyment of the property, and they serve to notify future owners of their rights and obligations. Typical information required to be shown on the record plat includes:

- Subdivision name and section
- Parcel boundaries, including bearings and distances as well as the legal description
- Adjoining property lines, including ownership information
- Street right-of-way boundaries, including surveyed bearings and distances
- Street names, both existing and proposed
- Lot lines, including surveyed bearings and distances, lot and block numbers, lot addresses, building restriction lines
- Building footprint for certain types of uses
- Existing easements and easements created as a part of the development, including purpose and deed references
- Location of survey monuments and markers
- Conservation and other building-restricted areas, such as septic drainfields
- Ownership declarations and dedications
- Covenants and restrictions, including reference to separate documents in jurisdiction's public land records
- Approval blocks for signature(s) of authorized government officials

### Performance Guarantees and Bond Release

As discussed in Chapter 3, performance guarantees are usually called for in the subdivision regulations. However, the actual bonding process in most jurisdictions is a matter of policy and administrative procedure. The developer typically has two options with respect to the installation of the necessary public improvements. The first option calls for the completion of improvements such as streets, storm-drainage facilities and utilities, prior to approval and recordation of the final subdivision plat. Certificates of occupancy, allowing users to move in to homes or offices once they are built, will not be issued until the improvements are completed. In such a situation, the developer usually will be required to post a maintenance agreement and bond that remains in effect for a specified period, usually from 1 to 2 years. This agreement specifies that streets and facilities will be maintained by the developer and repaired in the event of premature deterioration or damage

caused by construction vehicles. Such agreements typically require the developer to keep public facilities in clean and operating condition, and often include snow removal as an obligation. Upon release of the bond, the local jurisdiction exercises the rights conferred by the developer's dedication and accepts the facilities into the public system. The municipality then accepts responsibility for and maintenance of the facility.

Under the second option, the developer is allowed to execute an agreement that provides for the future installation of improvements, within a specified period of time. A financial security, such as a bond, letter of credit or similar instrument, is posted with the jurisdiction. The amount of the instrument, the sum of estimated construction cost plus inflation factors, will be used by the jurisdiction in the event of default by the developer. The developer is then able to secure approval of final subdivision plats and start construction of public improvements. As structures are completed, occupancy permits can be obtained while the bond is in effect. The municipality usually has procedures for partial release of bonds as construction of improvements progresses. As with the first option, maintenance bonds and agreements are required after construction is completed.

With both options, the development team must produce "as-built" drawings upon completing construction. These drawings are usually modifications of the project's construction drawings, intended to reflect modifications made in the field by construction contractors. These drawings serve two important purposes. First and foremost, they present government agencies with the final opportunity to verify that construction complies with local regulations and standards. This is necessary to ensure that systems will function properly and not cause unanticipated impacts to the larger community. Second, the plans facilitate future maintenance and repair, particularly for underground utilities whose location, sizing and method of construction is otherwise hidden from view.

### REFERENCES

- David Jensen Associates, Inc., *How to Win at the Zoning Table*, National Association of Home Builders, Washington, D.C., 1984.
- Engineers and Surveyors Institute, *Improving Preparation, Review & Approval of Subdivision & Site Plans in Fairfax County*, Engineers and Surveyors Institute, Fairfax, Va., 1988.
- Institute for Participatory Management and Planning, *Citizen Participation Handbook*, Institute for Participatory Management and Planning, Laramie, Wyo., 1986.
- International City Management Association, *The Practice of Local Government Planning*, International City Management Association, Washington, D.C., 1979.
- National Association of Home Builders, *Subdivision Regulation Handbook*. National Association of Home Builders, Washington, D.C., 1978.
- Urban Land Institute, *Working with the Community: A Developer's Guide*, ULI-The Urban Land Institute, Washington, D.C., 1985.



## CHAPTER 33

# PLAN SUBMITTAL, REVIEW, AND APPROVAL PROCESS

*H.S. Hulme Jr., P.E.*

### INTRODUCTION

Some development projects can take place without the necessity for rezoning. However, more often than not, a rezoning process precedes the detailed design for construction. Approval of a rezoning application is insufficient to initiate construction. Detailed design and structural analyses are rarely incorporated to a sufficient degree in the rezoning process. The development team can encounter numerous pitfalls in attempting to translate an approved zoning plan into a subdivision or site-plan submittal.

The engineering analysis performed for a subdivision or site plan, as a part of the construction plan design, often suggests that the assumptions used in the rezoning submittal were insufficient for final design and sometimes are imprecise and in some cases incorrect. This is because of the differing level of detail each type of approval requires. Because of the risk and expense involved in achieving this accuracy with the more speculative legislative rezoning application, many land developers are reluctant to add the associated costs with greater exactness. Yet, some jurisdictions will require legislative approval of revisions to the previously approved rezoning development plans because of the changes made as a result of this subsequent detailed engineering plan analysis.

The rezoning process is not intended to be the approval process to begin construction. The rezoning process is intended to designate the allowed use of the land under the circumstances portrayed in the plans and other documents submitted in support of that process. It should be obvious that construction could not take place using the plans developed for the rezoning process. Much more detailed plans are necessary. These detailed engineering plans must also

be approved by public agencies prior to the issuance of a permit.

The rationale for plan approvals and associated codes and ordinances cannot be better stated than as found in older publications regarding this authority. The plan approvals in regard to the associated codes and ordinances “are essential to meet the needs of the community in the fields of public health, safety, and economic and social well-being. Provision of this public protection rests upon the police power of the government. Police powers are reflected in the enactment of master plans, zoning codes, building and building equipment codes, housing and sanitation codes and other regulatory ordinances. Good codes, well-administered, provide the only sound means for a balanced program to ensure the orderly development of a city, the prevention of slums and blight, and the safe-guarding of life, health, and property” (International City Manager’s Association, 1957).

Each jurisdiction has administrative requirements and procedures that govern the submittal of construction plans. Failure to follow these requirements can at best result in a delay in the plan’s approval. It is possible for the plan to be disapproved even though the plan has been designed in accordance with accepted design standards. It is the administrative requirements dealing with the particular way a jurisdiction desires to see the construction information that can cause a delay in plan approval. Most jurisdictions welcome a pre-application meeting with staff so that the types of plans needed and the application process is understood.

The plan-review process can be very time-consuming. This is especially true in developing urban areas. Some minor development plans take as long or longer to follow the review process than it has taken to develop the design. The



time schedule for the review process needs to be taken into account when planning a development project. This will avoid costly surprises when the design has been completed and the development client expects a construction permit in an unrealistic time frame. A somewhat typical review process is shown in the flow chart in Figure 33.1.

It is the purpose of this chapter to provide a better understanding of the administrative requirements and procedures for processing construction plans within jurisdictions. Topics to be covered will be the types of plans and how they are sequenced within the approval process, who reviews these plans and why, and some typical administrative processing problems and how to avoid them.

**PROFESSIONAL RESPONSIBILITY IN THE PLAN-REVIEW PROCESS**

The public works infrastructure codes and ordinances are generally viewed as minimal standards to be met; however,

in practice, they become the standard to which to design and construct. Conforming to these standards is essential to ensure continuity and compatibility with existing and future infrastructure. Public review and approval of engineering plans are necessary to protect the basic health, safety and welfare of the public. Maintenance expense of infrastructure is a major cost to public agencies. This is an important reason why public agencies take a keen interest in adherence to their adopted standards and specifications.

The private ownership of the internal infrastructure being designed can arguably be of less importance for public review and approval than that which will ultimately be the public's responsibility. Since it will be in private ownership, the quality and adequacy will be reflected in the value of the property. However, increasingly, the ultimate owner looks to the public review agency for quality control of the final product. Likewise, often the privately-maintained infrastructure systems have an effect upon the publicly-

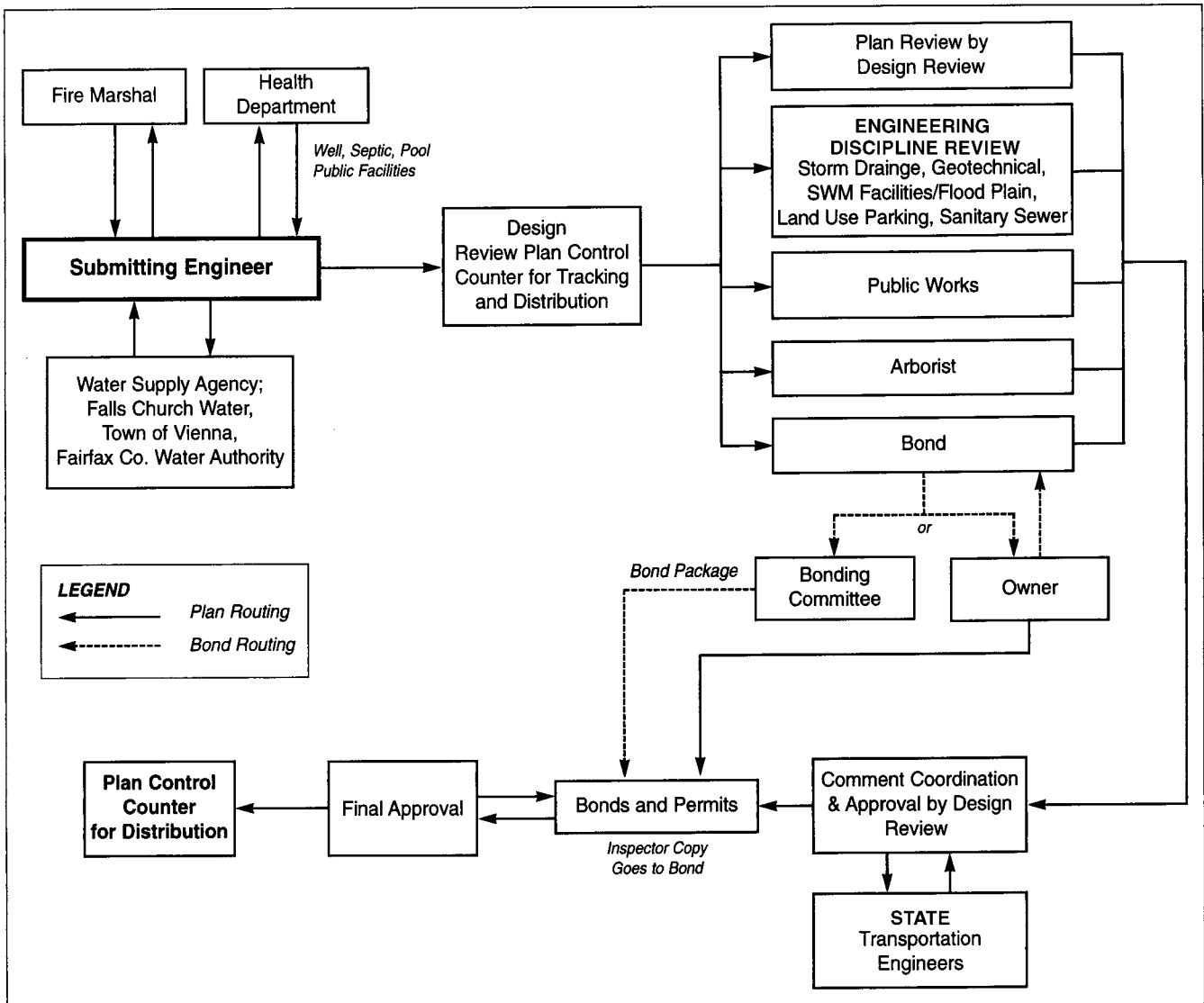


FIGURE 33.1 Typical plan review process flowchart.

maintained system. Then too, it is not uncommon for failed, privately-maintained systems to have to be taken over by the public. For these reasons, there is little difference in the review process or attention to detail for private or publicly-maintained infrastructure designs.

### Public/Private Sector Relationship

Most jurisdictions recognize the economic benefits of quality development. While it has not always been true, there currently exists a conscious effort on the part of administrators to assist those in the land development profession. Administrators recognize that responsive staff, regulations, standards, review processes and procedures that support and facilitate land development are more likely to attract the private sector to conduct business in that jurisdiction. A simple, cost-effective, and efficient regulatory process is seen as a pre-requisite for attracting development (City of Dallas, Texas).

Even so, there is a healthy adversarial relationship between public agency individuals charged with the responsibility to review and approve plans and those professional engineers designing and submitting plans. Many professional engineers believe that their plans need only be given superficial review in as much as they are taking full responsibility for the adequacy of their designs. They believe they have provided reasonable degrees of safety and adequate public protection (Bright, John K., 1992). On the other hand, the public-agency plan reviewer has been given the responsibility to decide when adequacy and reasonableness have been met. Detailed, specific design requirements adopted by state and local jurisdictions also require that technical standards be met accurately.

Does the engineer seal on a set of plans certify that administrative requirements and procedures for processing these plans have been met? There is no such thing as the perfect plan. Different engineers reviewing the same set of plans will find and miss different technical requirements. What, then, is the significance of the engineer's seal and signature?

The engineer seal on a set of plans certifies the adequacy of the design with regard to public good and safety and is best defined by the state licensing boards. State licensing boards are explicit about the engineer's responsibility to the public.

"The primary obligation of the professional is to the public. If the professional judgment of the regulant is overruled under circumstances when the safety, health, property and welfare of the public are endangered, the professional shall inform the employer or client of the possible consequences and notify appropriate authorities" (Virginia Board of Architects, Professional Engineers, Land Surveyors, Certified Interior Designers, and Landscape Architects, 1999.)

They also are explicit in stating the engineer's obligation regarding competency.

"The professional shall undertake to perform professional assignments only when qualified by education or experience and licensed or certified in the profession involved. . . . The professional shall adhere to all minimum standards and requirements pertaining to the practice of his own profession as well as other professions if incidental work is performed" (Virginia Board of Architects, Professional Engineers, Land Surveyors, Certified Interior Designers, and Landscape Architects, 1999.)

Less explicit is the professional responsibility regarding the administrative as well as technical requirements of each particular jurisdiction. While current practice allows less stringent professional standards of conduct in this area, it certainly behooves the engineer to be informed of the administrative and technical requirements of the jurisdiction in which the engineer is practicing. As with all ethical decisions, the engineers sealing and signing the plans must make the decision whether they are fulfilling their professional responsibility. Rarely are engineers cited for neglecting the detailed technical requirements of a design plan unless the public safety is clearly in jeopardy. This does not relieve the engineer of the responsibility to design and submit the best possible plan that to the best of the engineer's knowledge conforms to all codes and ordinances.

### LOCAL JURISDICTION PLAN REVIEW

#### General

The agency taking the lead role in performing plan review varies from jurisdiction to jurisdiction. In larger communities, the planning and/or public works department is given this responsibility. A multi-purpose agency established to administer building and environmental regulations is sometimes created for this purpose. In small communities, an individual such as the town manager or engineer may be responsible. Some communities contract with engineering design firms to conduct this review. The designated review agency also is usually responsible for coordinating the review of plans by other governmental and quasi-governmental bodies. However, this does not always extend to the review required by state or federal authorities.

#### Plan Distribution to Review Agencies

In a typical jurisdiction, one agency is usually designated as the lead agency, and this lead agency distributes copies of the subdivision or site plan to other agencies for review. Most agencies are asked to review submittals to identify the location, adequacy and availability of services and assess the impact on systems for which they are responsible. These agencies will also review plans for compliance with ordinances and standards that they may administer. Their role is to ensure proper relationships between systems constructed as part of the new development and the community's existing and proposed patterns and networks.

Some agencies receive multiple copies to assist in monitoring development activity. This aids in planning and

budgeting for public manpower and facilities to service future users. Local ordinances frequently define the reporting relationships for this review, specifying whether agency comments are advisory or mandatory to the lead department. The distinction is important to the development team. As indicated in the previous chapters, differing departments are responsible for individual programs that have varied and sometimes conflicting priorities. It is important to know who has the authority to resolve conflicts when competing demands confront the developer.

Agencies involved in rezoning review are also involved in construction plan review. While the reviews are similar, the latter's focus is more detailed and is concerned with design, sizing, placement and connection.

- A long-range planning department usually determines if the street and system layout adheres to development patterns proposed in the comprehensive plan. The comprehensive plan will normally cover such planned development topics as land use, transportation facilities, drainage and other utility facilities and school and recreation facility locations. The staff identifies lands proposed for development that may be designated in the plan for public use, such as highways, trails, schools, parks, libraries, fire or police stations. The review may trigger a request for reservation of needed areas.
- Personnel in the current (short-range) planning department will determine compliance with zoning ordinance requirements, such as lot size and density requirements. In addition, the submittal will be reviewed for compliance with the design guidelines and standards in the subdivision regulations for street and lot layout, environmental protection and other planning issues. Personnel in both departments, as well as in other departments, often perform environmental reviews.
- The public works department typically is responsible for streets, storm-drainage facilities, sewer and water systems. Other elements of the development may also fall within the department's purview, such as placement of survey monuments and markers. However, in some communities, streets may come under the jurisdiction of a separate transportation department or a state-level transportation agency. Similarly, water and sewer systems may be the responsibility of a regional authority serving several jurisdictions. A storm-drainage district may also have a separate level of authority. A local or state health department can also have regulations concerning on-site and off-site sewage disposal and the placement of septic drain fields and wells.
- Fire protection officials are involved in the review of subdivision and site-plan proposals to ensure that

an adequate and accessible water supply for fire suppression has been provided. This includes the placement of fire hydrants and sizing of water lines. Coordination of this aspect of the plan is necessary with the agency responsible for water supply. They also ensure that clear and adequate access is provided for emergency vehicles throughout the project. They also monitor development activity to assist in locating future fire-station facilities.

- School districts typically perform an analysis of potential school enrollment generated by new development. While long-range planning for school facilities is done through review of the comprehensive plan and rezoning, the actual assignment of pupils to individual schools is predicated upon projections of when actual development will have taken place.
- A local parks and recreation department will review subdivision and site plans to evaluate recreational needs of the new development and the impact on existing facilities. In some jurisdictions, the subdivision regulations contain requirements for the provision of such facilities. This department may also be responsible for preservation and acquisition of important environmental areas, such as floodplains and wetlands, the control of which may be included in subdivision regulations.
- A building department may be responsible for reviewing grading, construction and erosion and sediment-control plans that are included as part of the submittal package. They will also be involved in project inspection as construction proceeds.
- The local gas, electric, telephone and cable TV companies will be involved in confirming system adequacy, connections, the placement of utility easements and coordination of construction with the street, water and sewer contractor. Usually the design of these facilities is done by the utility company and may be incorporated in the final plan submittal.
- The soil conservation district or similar entity may review grading, erosion and sediment control, storm drainage, flood plains and other environmental issues.

Two of the most common unknowns to the submitting engineer are what agency needs what plans and when are the plans needed. New and revised regulations and new procedures often change the agencies needing to review and approve development plans. Engineers often do not submit enough plans to any one jurisdiction to develop a through knowledge of the administrative procedures. For this reason, it is beneficial to review the written procedures as well as to seek the advice of others that are knowledgeable of the process and practice prior to submitting a plan for review and approval. In some jurisdictions, these proce-

dures are typically included in a subdivision and site-plan ordinance or a public facilities and standards manual. As previously indicated, many jurisdictions welcome a pre-submittal conference.

Some larger jurisdictions are experimenting with the submittal of plans for review electronically. This could be a significant improvement during the review process. An electronic submission allows the reviewing engineer to make comments but not change the drawing. Comments can be sent back electronically to the designer as well as to other reviewing personnel. This can speed up coordination and design modifications. The expense and time of printing and distributing multiple copies will also be reduced dramatically.

Reviewing plans on a computer monitor requires the reviewer to make a major change in review approach and technique. It is difficult to change from reviewing plans displayed on a plan sheet to reviewing plans with a computer monitor. For instance, while it may be easier to review just the drainage design without the interference of other design elements, it is also more difficult to see the infrastructure integration. Clearly, reviewers and designers will need training and experience before the electronic review can be fully acceptable.

The legal aspects of electronic submittal are still under legal and methodology evaluation. There is the natural tendency to distrust the electronic submittal in favor of the current hard-copy originals with professional signatures. It may be some time before both the review process and final plan submittal will become a common, acceptable practice.

Computers can be used for plan review by scanning the submitted plan and dispersing the plan through the jurisdiction's computer data system. Reviewers can then review the plans from their individual computers and download and print only those plan sheets necessary for their individual review. This also permits the jurisdiction to integrate the proposed design into their GIS database, allowing easy access for follow-up questions and review. It is becoming more acceptable to communicate comments and questions electronically, thus easing the time requirement to reach someone by telephone or to transport comments by mail or hand pick-up by the designer.

### **Administrative Requirements**

Administrative requirements determine the manner in which plans are submitted for review. Typical administrative requirements will dictate the types of plans and number of copies needed, plan sheet size and standard scale, format of the sets of plans, agencies participating in the review process, the sequence of review, required supporting documentation and general information needed. Administrative requirements may also specify the need for a construction schedule and a cost estimate and may specify the time restraints associated with approvals. Since administration requirements change frequently, it is recommended that the

engineer check to see if the most current issue of the requirements is being used.

Administrative requirements facilitate the review process by standardizing information supplied with all plans. This is most helpful in the review process of large jurisdictions processing large numbers of plans from many engineers and surveyors. Since fewer plans are being processed for review in smaller jurisdictions, there is less need for detailed administrative requirements.

Some jurisdictions require the submission of a checklist indicating the completeness of the plan submission. The checklist will list all the required documents as well as some items that have been frequently omitted in prior submissions. The submitting engineer must check the items supplied and sign the checklist. This serves as a check for the engineer and facilitates the review process by ensuring that important documents that would delay the review have not been omitted.

Some engineers discount the importance of complying with the administrative requirements. They have devoted so much of their time to ensuring that the project is designed within the site constraints, complying with technical requirements and economic constraints, that the administrative requirements seem trivial. It is important to remember that failure to adhere to these requirements can delay the review process and frequently will result in plan rejection or disapproval. Not adhering to the time frames outlined in due diligence requirements can result in the expiration of the plan, necessitating a resubmission or even a new legislative approval.

The public agency rejecting a set of plans will comment as to why the plan is being rejected, noting things that need to be done to correct it. These comments are usually written, but they may be verbal. Meetings and conversations with the submitting engineer are helpful in establishing direction to secure plan approval. Once again, failure to follow the suggestions of the reviewing agency can result in the agency continuing to reject the plan. Obviously, the engineer's client can become dissatisfied with the engineer and question his or her competence. While there is no such thing as a "perfect" plan, following the suggestions of the reviewing agency and maintaining good professional relations will increase the likelihood of plan approval.

### **Bonding and Agreements**

Upon completion of the review and approval of the construction plans, additional administrative steps are necessary to ensure that not only is the infrastructure construction completed but that it conforms to the approved plans. These guarantees take the form of enforceable bonds and agreements. In most cases, the documents must be processed before the issuing of permits that would allow construction to commence.

The agreement is the legal, binding contract between the developer and the permit-issuing authority. The agreement specifies the manner and date by which the construction as

shown on the approved plans shall be completed. A bond or other forms of security approved by the local jurisdiction backs the agreement.

The performance bond assures compliance with the terms of the agreement. The bond is usually the full amount of the estimate for construction of the public improvements. The submitting engineer determines the quantities of items subject to bonding and certifies the accuracy of the estimate. For more information on bonding and cost estimation, please refer to Chapter 31 Construction Cost Estimating. The bond amount is a reasonable estimate of what one could expect to pay to have the improvements constructed within the period specified in the agreement. This estimate is reviewed and approved by the local jurisdiction as a part of the approval of the entire bond package. It is not unusual for larger developments with considerable infrastructure construction to develop the project in smaller sections to avoid the financial burden of bonding the entire project.

Upon completion of the physical site improvements and compliance with all other regulations, the bond may be released. Some jurisdictions allow for partial release as elements are completed and accepted. An inspection is conducted to ensure compliance with all previously-noted code infractions to ensure that all infrastructure construction is acceptable to the agency assuming maintenance responsibilities, and to ensure that all fees have been collected and the as-built construction plans have been approved.

Some localities require a conservation agreement and bond in order to preserve those parts of the development property that are to remain undisturbed. This is to ensure protection of trees, historic structures and the like. Accurate identification of these areas is necessary to avoid errors and misunderstandings. The conservation agreement also ensures that the erosion and sediment controls are installed and maintained.

### Permits

Based upon the approved construction plans, permits are necessary before the initiation of construction. Some typical types of permits necessary would be clearing and grading, siltation and erosion control, street construction, drainage facilities, sanitary sewer and water lines. Permits alert the locality that construction is about to commence so that inspection can be instituted. They also serve as a safety check that construction is being performed with plans that have not become outdated. They will also allow for any special conditions that may be necessary at that particular point in time. Generally, it is the client or contractor's responsibility to secure permits. The design engineer may be called upon to assist in this as needed.

### Plan Revisions and Record Documents

It is rare that plans do not need changing once construction commences. Changing field conditions, discovery of pre-

viously unidentified information affecting design, changes in the building design, changing economic conditions—all contribute to plan changes. The submitting engineer must prepare these changes and submit the changed plans to the local jurisdiction for approval. Since construction may be underway, quick review and approval are essential. Most jurisdictions give priority treatment to revisions of already approved plans. The procedure for filing the plan is the same as for the filing of construction drawings.

Local jurisdictions will want some assurance that the actual construction performed is in accordance with approved plans and revisions. For these reasons, a record document plan (sometimes inaccurately called "as-builts") is usually required prior to the release of the construction bond. Record document plans show the boundary of the site, location of all buildings, horizontal and vertical location and size of pipes and apertures, location of fire hydrants, location, width of streets, walks, trails and other improvements as constructed.

Since the engineer has created a revised plan in the form of the record document, he or she is required to certify record document plans in the same manner as design plans. Some agencies require that the engineer certify that the record document plans conform to the approved and revised design plans except as shown. Some difficulty may be experienced with this, depending upon what is shown on the record documents. Usually the record document plan is created from the original design documents. "It is the general contractor who is responsible for supervising the day-to-day work on the site; he or she is the only one in a position to identify and record changes made during the course of construction" (DeJarnette & Paul, Inc., 1991). These documents may contain information that was not a part of the record document survey requirement. In this instance, caution is necessary with respect to what the engineer is certifying. A suggested precaution to be stamped on each sheet of the record drawing, subject to review by attorneys, would be as follows:

"These record drawings have been prepared based on information provided by others. The Engineer has not verified the accuracy of this information and shall not be responsible for any errors or omissions which may be incorporated herein as a result" (DeJarnett & Paul, Inc., 1991).

As stated previously, record document plans must be approved prior to the release or partial release of the construction bond. For this reason, there is usually a critical time constraint on the development of the record documents.

### Meetings with Citizens

Frequently, jurisdictions will require design engineers to meet with citizens as a part of the plan-submittal process. Some jurisdictions have set up formal processes for this. Once plans have been submitted, they can be subject to review by citizen groups in the immediate area as well as

citizen groups set up for the specific purpose of reviewing development plans. These meetings are generally not in the regulated process and sometimes are seen as delaying tactics by those opposed to the proposed development. Jurisdictions see this process as being responsive to their citizens. It enables citizens to provide input to the project that could overcome objections if discovered later during or after construction.

The engineer can profit from these meetings by not only adjusting the design to fit better into the community, but by gaining support for the project. Often, engineers are reluctant to defend their design to non-engineers because they lack the communication skills to convey important design considerations. Engineers need to be better able to communicate their design to non-engineers in order to maintain credibility for the profession. This takes sincere listening and then responding to legitimate citizens' concerns. United States Senator J. William Fulbright's comments regarding student demonstrations during the Vietnam War is fitting for the engineer: "The student, like the politician, must consider not only how to say what he means but also how to say it persuasively" (Washington Post, 1992).

### Meetings with Review Staff

Jurisdictions that encourage communications between the design engineer and the review staff prior to plan development recognize the benefit in attracting quality development. The ability of the submitting engineer to understand the regulatory process and, subsequently, to move plans through the process in a timely manner benefits both the design engineer and the jurisdiction. The uniqueness of the specific plan, sequence of plan type submission, anticipated schedule and other important elements can also be determined.

A telephone conversation or meeting prior to the preparation or submittal of a plan can be most beneficial. In any event, whenever the engineer lacks knowledge about the process and other in-house professional staff cannot be of assistance, a request to meet with the reviewing agency should be made. It is far better to understand the process in advance rather than have to suffer through it by experience.

Public-agency reviewers must have a broad technical knowledge as well as knowledge of the review process. They are required to make recommendations on rezoning development plans, sketch plans, preliminary plats, plat check sheets, grading plans, public improvement plans, waivers to plans, subdivision plans, site plans, easement plats and record plats. They also review and make recommendations on the environmental impact of proposed development, the need for drainage studies, flood-plain studies, soil studies and CBR test results. They determine street design and alignment requirements. They must have a thorough knowledge of the subdivision provisions, zoning

ordinances, road specifications and other technical and procedural texts. Consequently, a reviewer can reasonably be expected to be able to respond to inquiries of the submitting engineer. However, this does not relieve the engineer from learning the technical and administrative requirements of the jurisdiction in which the engineer is practicing.

Following the initial review of plans, it is desirable to meet with the reviewing authority to understand the comments. This will enable the engineer to know specifically what needs to be changed in the plan in order to gain approval. Some jurisdictions arrange to have a representative from all of the reviewing agencies present at one meeting to coordinate comments better. This allows all questions and concerns to be addressed in one meeting with the appropriate officials (International City Manager's Association, 1986). Some jurisdictions make these post-submission meetings mandatory. In the Virginia suburbs outside of Washington, D.C., the mandatory "post-submission conference" agreed to by both public and private-sector engineers has been touted as contributing significantly to the reduction in the number of submissions necessary for plan approval. A specific format was developed for the conducting of the meeting (Engineers and Surveyors Institute, 1990).

These meetings between the submitting engineer and the public-review staff contribute to the healthy adversarial relationship mentioned earlier in this chapter. The reviewer has the responsibility to ensure that the desired plan changes are understood properly and that the design engineer is willing to make those changes. The design engineer has the responsibility not to compromise the design based upon his or her perception of potential harm to the health, safety and welfare of the public in violation of the professional engineer's plan certification. The design engineer is the client's representative at these meetings and should have a clear understanding of his or her limitations with regard to agreeing to design changes. Of particular concern would be those changes that will have a financial impact or significantly alter the project.

### PLAN SET FORMAT AND SEQUENCING

As indicated previously, design plans submitted for review and approval are the basis of the issuing of a permit or permits for construction. During the course of the design-review process, permits are issued for various phases of construction. They are also issued by various agencies signifying that the design meets specific codes and regulations. The approval of the plans themselves does not constitute authority to begin construction.

Each jurisdiction has its own list of requirements of what needs to be submitted to gain plan approval. They also have requirements as to what constitutes a set of plans and the format of the plan set. In general, construction-related information is assembled in logical order in the front section of the plan set, and the non-construction related data is placed in the last section. Specific plan sheets in a typical

set of plans submitted for approval fall into five sections: (1) cover sheet, (2) general information, (3) specific construction sheets, (4) additional construction sheets and (5) documentation information (Engineers and Surveyors Institute, 1990).

### Cover Sheet

The cover sheet typically will contain a location map, an index of all sheets, plan identification, submitting engineer data, information considered pertinent to someone seeing the plan for the first time and other general information. Some jurisdictions will require an *original* engineer's seal and signature on the plan cover sheet.

### General Information Sheets

These sheets contain all general notes not shown on the cover sheet. Typically, general information sheets contain the plan legend, typical sections, and modified designs. Information sheets will also reference the construction specifications to be followed.

### Specific Construction Sheets

Specific construction sheets consist of plans and profiles of the infrastructure (such as streets, curbs and gutter, sidewalks and trails) as well as grading details and grading plans. They also would include the plan and profile of water, sewer and storm-sewer lines, storm-water-management facilities, water meters, fire hydrants, street lights, street trees and other similar types of construction details.

### Additional Construction Sheets

Additional construction sheets would contain miscellaneous construction details such as utility and drainage profiles not previously shown, remaining profiles such as driveways, and special design items. Here also would be included the landscaping plans, siltation and erosion-control plans and geotechnical information. The landscaping sheets would include the plant list and landscape construction details. The geotechnical plan sheets would show geotechnical notes, foundation drains or other geotechnical construction details. A geotechnical report is sometimes necessary and included with the plan submission. The erosion and siltation sheets would include the erosion and sediment narrative and notes, erosion and sedimentation computations with sediment trap sizes and siltation divides and any details necessary for installation.

### Non-construction Data Sheets

Plans and documents needed for plan approval include the detailed calculations and methodology used to design the construction plans. This would include drainage area diagrams and storm-water routing calculations used to size the storm-water drainage and management system as well as flood-plain studies. Outfall narratives, ditch cross-section computations, and storm-inlet computations would also be included. Fire flow calculations would need to be provided to the fire and water authorities.

Plan approval documents would also include sight-distance calculations at intersections and critical driveways. Soils reports and other geotechnical information are also provided in plan form as well as in reports if these were not included in previous sheets.

In some jurisdictions, verification that appropriate public notices have been sent and permits from other agencies obtained is also needed. Other correspondence and supporting non-construction data sheets would include such information as fire-lane designations, overall sanitary sewer plan, condensed drainage analysis, construction waiver requests, record plats, easement plats, and other supporting documents such as preliminary plans and development plans.

Using these drawings and reports, the submittal shows the arrangement of property lines, lots and other features that will occupy the site. It also shows the location of public and private facilities to serve the intended uses. Plans included as a part of the submittal also contain information on how those facilities will be constructed, from grading of the property to connection of structures to water and sewer lines.

### Typical Plan Types

Not all plan types are subjected to the same administrative requirements. Examples of some typical plan types are listed in Table 33.1 The number of copies shown for each plan type is typical for the plan-review process. As can be seen, the number of copies varies considerably depending upon the number of agencies conducting a review of that specific plan as specified in the administrative regulations. The number of copies shown should not be taken as necessarily typical, since many jurisdictions require far fewer copies. The number of copies requested is based upon the number of review agencies and the method of transmitting and returning review comments. The limiting size of each of the listed plans may vary. The engineer will need to determine the required scale of each plan and plan the sheet layout accordingly. The use of CADD has enabled the engineer to reduce quickly the scale of plans to conform to single 24 in. by 36 in. plan-sheet size in order to see more of the project. This may reduce the size of important printed information so that it is difficult to read. Jurisdictions will also have a minimum size lettering to avoid this situation.

Because of the many interests involved in the review of plans, numerous copies are needed for the review process. Most jurisdictions have a listing of the plan types needed for a particular design plan as well as the number of plans needed for submission. This listing is usually incorporated into the codes and ordinances or in an administrative requirements and procedures document.

The number of copies of each plan being submitted is determined by the type of plan being submitted, the number of review agencies for that particular plan and whether those agencies wish to retain a copy of the plan being submitted. It is more efficient for concurrent review to be tak-

**TABLE 33.1 Number of Submittal Copies Needed for Various Plan Types**

PLAN TYPE	No. of COPIES
Drainage Study	2
Final Record Plat	16
Final Record Plat Redate	16
Flood Plain Study	2
Preliminary Plat	18
Preliminary Plat Redate	18
Preliminary Plat Revision	4
Public Improvement Plan	14
Rough Grading Plan	7
Sanitary Plan	7
Site Plan	16
Site Plan Redate	4
Sketch Plan	14
Soils Report	7
Subdivision Plan	14
Subdivision Plan Redate	4
Waiver Condition	2
Water Plan	7

ing place on a set of plans rather than one or two sets being reviewed by agencies in sequence. It is not unusual for 15 or more copies of a particular plan to be required for submittal.

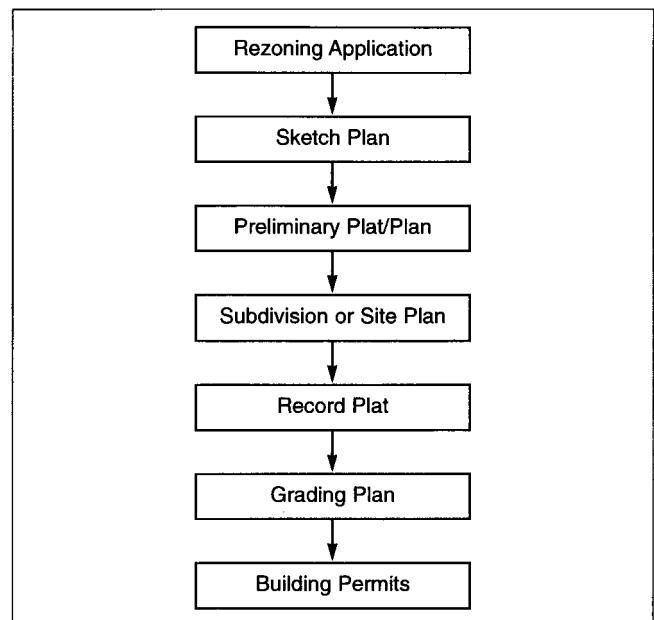
**Plan Type Submission Sequence**

Local government agencies require the previously-identified plans to determine if the design complies with applicable ordinances and standards. The plans and accompanying documents are prepared, submitted and reviewed in a pre-determined sequence of increasing accuracy and detail. This staging of analysis and design is important. Most, but not all, jurisdictions consider construction-plan approvals as an administrative act of the staff, without involvement of the legislative body or the public. Some jurisdictions require planning-commission review and approval of preliminary subdivision plans. However, even the most prescriptive ordinances allow and necessitate considerable judgment by design professionals concerning on-site layout and connection of off-site facilities.

Before expending significant time and money for the preparation of final layouts and construction plans, the developer wants assurances that plans will be approved. The various plans submitted during the process incorporate a series of checkpoints with the reviewing agencies. These checkpoints enable the development team to gauge whether its interpretations of local policies, regulations and standards match those of local government authorities. Otherwise, extensive redesign would be needed. For example, redesign would be needed if the lot layout did not fully comply with the requirements of the zoning ordinance, or if the street and utility system did not consider planned or existing networks properly. A typical sequence of plan submission is shown in Figure 33.2. Reports, studies, plans and documents are needed in a special sequence or concurrent with the design-plan submission. The sequencing is dependent upon the type of plan being submitted and upon the requirements of the individual jurisdiction. A typical plan sequencing in a local jurisdiction for a subdivision plan is shown in Figure 33.3. Plans not dependent upon information being reviewed on other yet-to-be-approved plans may be filed concurrently. Sometimes filing concurrent plans is necessary to provide for a more adequate review. Filing plans concurrently greatly speeds up the approval process. The engineer should attempt to file plans for review concurrently whenever possible.

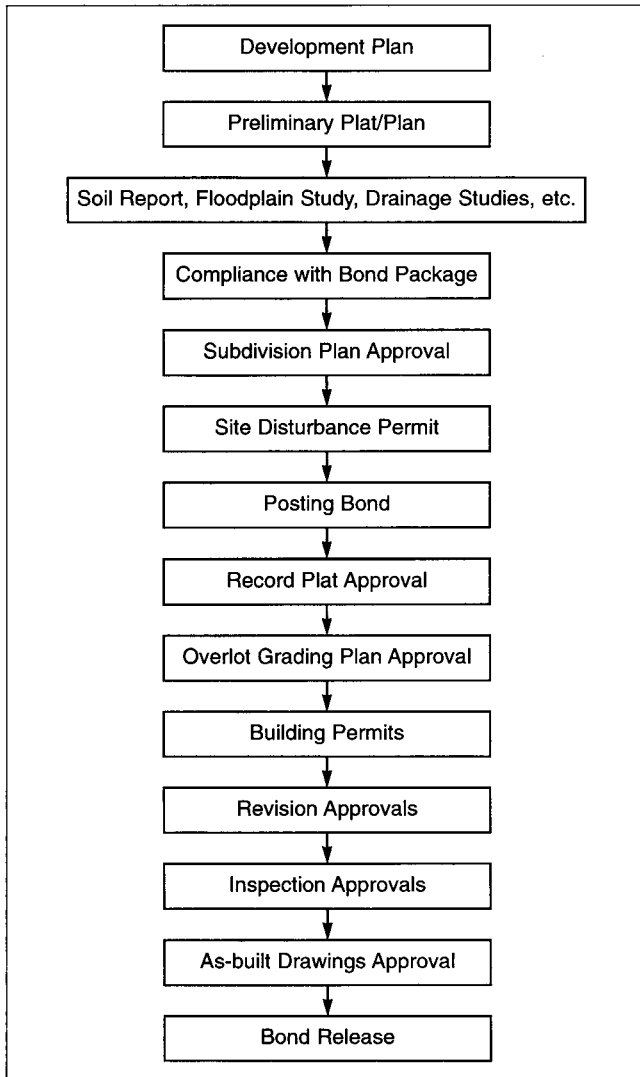
**FEDERAL REVIEW OF CONSTRUCTION PLANS AND PERMITTING**

Federal legislation is sometimes enacted that affects land development throughout the country. The Clean Water Act, Americans with Disabilities Act and Clean Air Act are examples of this type of legislation. Following the adoption and presidential signing of the legislation, a federal depart-



**FIGURE 33.2** Typical sequence of plan submissions.





**FIGURE 33.3** Typical plan sequence for a subdivision plan.

ment (usually spelled out in the legislation) is given the responsibility to adopt implementing rules and regulations. This can take many months and in some cases years from legislative enactment. Rules are first proposed and printed in the Federal Register with time for public comment. It is this process that enables engineers to anticipate impending changes that could affect the project design.

### Clean Water Act

**Wetlands.** The Clean Water Act probably is the most significant act in terms of its effect on land development activity. One of the provisions of this act provides for the protection of wetlands. Wetlands are those areas that are inundated or saturated by surface or ground water at a frequency or duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs and similar areas. Guidelines for the delineation of wetlands are

contained in the Wetland Delineation Manual published by the Army Corps of Engineers. The Federal Manual for Identifying and Delineating Jurisdictional Wetlands published in 1989 has been canceled, and the former 1987 Corps of Engineers manual is used in its place. The applicant may ask the Corps of Engineers to delineate the special aquatic site (Federal Register, 1991).

Prior to planning and constructing development within wetlands, the applicant must show that no alternative site exists, that ecological damage will be minimized and that any lost wetland area will be replaced. A permit may or may not be issued based upon the information presented in the permit application. An application may invoke the use of a Nationwide Permit instead of the individual permit (Form ENG 4345). The prospective permittee must notify the Corps of Engineers District Engineer as early as possible. Activity shall not begin until the District Engineer notifies the permittee that the activity may proceed or notifies that an individual permit is required. Alternatively, activity may begin if 45 days have passed since the District Engineer has received the notification and the permittee has not received notice from the District or Division Engineer.

There are 43 listed nationwide permits activities (44 are listed but one number is not currently used.) Commonly used nationwide permits in development projects are:

#12—Utility line backfill and bedding—may be placed through wetlands parallel to streams

#14—Road crossing—must be part of a single project, fill may not exceed one-third of an acre and the road must cross perpendicular to the stream channel

#26—Headwaters and isolated waters discharge—applies for filling of up to 10 acres of wetlands located above the headwaters of a non-tidal stream, water body or isolated wetlands. This permit allows for the filling of one acre of headwater wetlands without prior notification. Fills of 1 to 10 acres require submitting a Pre-discharge Notification (PDN) to the Corps of Engineers.

#39—Residential, Commercial, and Institutional Developments—authorizes up to 0.5 acres of impact from discharges of dredge or fill material into non-tidal waters of the United States excluding non-tidal waters adjacent to tidal waters, for residential, commercial, and institutional facilities. A Pre-Construction Notification (PCN) must be filed with the U.S. Army Corps of Engineers for fills over 0.10 acres. Discharges cannot cause a loss of more than 300 lineal feet of streambed, although the District Engineer can waive this restriction for intermittent streams.

All permits issued by the Corps under Section 404 must have a Section 401 water quality certification from the state or tribe having jurisdiction; many have not authorized, or authorized with conditions, Section 401 certification for

Nationwide Permits. States may incorporate the permit-application process into their adopted water-quality regulations. When this is done, it usually results in a simplified process. However, this field of regulation is subject to rapid change, and the submitting engineer is advised to seek out the advice of those professionals more attuned to the water-quality aspects of land development. Penalties for violating the Federal Regulations are severe.

### **National Pollution Discharge Elimination System (NPDES)**

Also of significance is the need for a National Pollution Discharge Elimination System (NPDES) permit. This permit is a requirement for land development even for a relatively small size. Until recently, it was assumed that storm water runoff from urban areas and industrial facilities was essentially clean water. However, many studies over the past 25 years have shown that runoff from urban and industrial areas typically contains significant quantities of the same general types of pollutants that are found in municipal wastewater and industrial discharge and cause similar water-quality problems (Virginia Water Control Board, 1992).

Most states administer the Federal program through delegation of authority from the Environmental Protection Agency (EPA). This is done under regulations adopted by each state and approved by the EPA. Land disturbing and development activities are required to create storm water pollutant-elimination facilities so as to improve the water quality of receiving streams. For many years, these facilities were required only to minimize soil erosion from entering streams. Nitrogen and phosphorus are pollutants that significantly harm receiving streams. Best-management-practices (BMP) facilities are now required in most jurisdictions to prevent this type of pollution. These practices can be either structural or non-structural. Refer to Chapter 22, Stormwater Management, for a more detailed discussion of these facilities. The degrees of land-disturbing activity, location of the storm water management facilities and their design are the subject of review and eventual issuance of a permit by the state.

### **Americans with Disability Act (ADA) and Final Fair Housing Accessibility Guidelines (FFHAG)**

Two federal regulations set requirements for accessibility. They are the Americans with Disabilities Act (ADA), which is applicable to non-residential buildings, and the Final Fair Housing Accessibility Guidelines (FFHAG), which is applicable to new multiple-family residential buildings. Most states have adopted regulations that require conformance to these federal standards, as have most local jurisdictions. A state or local jurisdiction can adopt a standard of protection greater than ADA. The state and local regulations then affect land development activity in that jurisdiction.

It is important to note that one cannot assume that a state or local jurisdiction has incorporated these regulations into their regulations. In that event, even though a federal

permit is not required, construction plans must still comply with the federal regulations. The ultimate purpose of the ADA and FFHAG is to bring those who have disabilities into mainstream American life. The FFHAG establishes standards for first-floor accessibility in multiple-family housing buildings. It also set standards for interior design to allow accessibility within specified numbers of units within the building. The engineer designing development plans for new construction has the opportunity to assist in this laudable goal.

ADA Accessibility Guidelines (ADAAG), issued by the Architectural and Transportation Barriers Compliance Board, have been adopted by the Justice Department as the standard to be applied in new residential and non-residential construction. These guidelines contain minimum accessibility requirements, technical specifications and requirements based on building use (Commerce Clearing House, Inc., 1992). Building sites must take into consideration:

- The addition of exterior ramps, chair lifts or elevators
- An increase in the number of accessible parking spaces
- An improved accessible route to the building entrance
- Altering the pavement to walking surfaces at abrupt changes in grade to warn the visually impaired
- Addition of signs to identify accessible routes
- Addition of a requirement for van spaces

### **National Environmental Protection Act (NEPA)**

Under the National Environmental Protection Act (NEPA), actions of federal and non-federal agencies that use federal funds or require federal approval are subject to an environmental impact review. In addition to government buildings and roads, a "major federal action" can include major private developments in federally protected wetlands.

The specific requirements are covered under 40 CFR 1500 et seq. Environmental review requirements for specific agencies are usually published in separate guidelines that are written to interpret NEPA requirements as they apply to a given agency's activities.

There are three levels of environmental review: (1) categorical exclusion, (2) environmental assessment, and (3) environmental impact statement. Projects that do not contain the potential for any significant environmental impacts can be granted a categorical exclusion. Exclusions are granted under a general waiver or with the submittal of minimal information.

Projects that do not fit the categorical exclusion definition will need to undergo an environmental assessment to determine if a full environmental impact statement is

needed. Projects that are deemed to have a significant impact upon the environment because of the environmental assessment are required to complete an environmental impact statement.

An environmental assessment will need to address the purpose and need for the proposed action, description of the present environment of the proposed project, alternatives and the environmental impacts. Typical subjects to be included in environmental impacts are socio-economic changes, historic landmarks, archaeological evaluation, air pollution, noise, hazardous waste, energy, water quality, floodplains, wetlands, threatened or endangered species, and flora and fauna habitat (Garfield, William W., 1992).

Because of the extensive requirements for the environmental assessment, a long lead-time will be necessary to secure acceptance of the report. The report is usually being prepared along with the preliminary plan design since information to and from both is needed.

A full environmental impact statement is a major undertaking. Specialists in this field usually perform it. Details of an environmental assessment are usually beyond the scope of the design engineer, and, therefore, a discussion is not included here.

### STATE REVIEW OF CONSTRUCTION PLANS AND PERMITTING

State legislation allows local jurisdictions to review individual development plans within their boundaries. State codes are usually enacted to detail certain requirements regarding the public health such as water supply, sanitation and state-wide uniform building codes. Likewise, some elements of the transportation system may require state approval.

Environmental issues in the area of water pollution, as noted previously, have led to enactment of several significant pieces of federal legislation. This in turn has led to the need for state legislation and state review of construction plans. These types of issues are much wider in scope than what prevails in one particular jurisdiction. The protection of streams from pollution and the protection of wetlands from destruction cuts across individual jurisdiction boundaries.

The processing of plans through state review agencies can be time-consuming, since plans submitted from throughout the entire state are being evaluated. Even with regional offices, the need for consistent treatment across the state requires time to complete the review.

### QUALITY CONTROL

It is unnecessary to say that quality control of the design of a project is the responsibility of everyone taking part in that design. The project manager usually assumes that role during the course of the design. However, recent trends emphasize the importance of quality control being installed into the product from the very beginning. It is not unusual for a Total Quality Management (TQM) program to be re-

quired by clients when entering into contracts with design engineering firms.

On the other hand, some clients demand unrealistic timelines on the design completion. This is usually motivated by businesses waiting to move into a new facility or because of the high cost of borrowing money on vacant land. Time becomes a very significant cost factor in these situations. It is this pressure of time that will make quality control seem like a nicety rather than a necessity. A hurried design can result in an inadequate design as well as not being in conformance with regulations. In more cases than not, time is not saved and the design is more costly in the end.

The typical approach to quality control regarding engineering plans concentrates attention on those plan elements relating to the ability of the project to be constructed efficiently and well. The design engineer's primary responsibility is to design the project to meet professional standards, and that requires almost singular concentration. The basic design of the infrastructure is the same regardless of jurisdiction. This results in the lack of attention to those plan elements peculiar to each jurisdiction that need to be correct in order to have the plan approved by the various reviewing agencies. There are many chances for error in the plan design and review process. The many different kinds of plans that are required for a specific situation increase the potential for error. In addition, so many persons and agencies are in the review process that the lack of an early identification of conflicting requirements causes problems late in the process. This adds to delay and expense.

Engineers in reviewing agencies in a jurisdiction are trained in the area of those elements of the codes and regulations specific to that jurisdiction. The reviewer sees many plans from that perspective and acquires skill in identifying areas of omission. One can readily see the juxtaposition of the design engineer concentrating on the quality control of construction elements while the reviewing engineer concentrates on those elements specific to that jurisdiction. The plan elements on which the reviewer concentrates often pertain more to the process than to the design product.

Plans submitted without undergoing a quality-control check for both construction and process elements clutter the review system and can result in an inadequate review because of omissions. Some jurisdictions institute a preliminary review of plans before accepting the plans into the process for distribution to the review agencies. This review usually is cursory, sometimes using checklists to ensure completeness in the set of plans submitted. The time taken to review for completeness is relatively short compared to the time necessary for the detailed review. Plans are rejected and returned to the submitting engineer when plan sets are incomplete.

There is a tendency to rely upon the reviewing agencies for the design quality control. When this is done, more time is needed to perform the agency review. It also reduces con-

confidence in the submitting engineer by the reviewing agencies that future designs will meet the jurisdiction codes and regulations. It is very tempting for a public-agency reviewer to comment on a set of plans with individual preferences as opposed to limiting comments that are supported by specific regulations. Likewise, a public-agency reviewer can become zealous in reviewing a development plan since in most instances the public agency will be the ultimate owner of the infrastructure being designed. Sometimes these comments can become burdensome and costly to incorporate into the design. The time necessary to incorporate comments into the design is also a cost item.

Design-engineering firms can improve upon the public-agency review process by instituting a good quality-control program that also emphasizes those plan design elements peculiar to the individual jurisdiction. This requires that engineers in the organization be trained to review plans in the same manner as do agency reviewers. They need to review plans as if the public agency is the client and their client is the ultimate owner of the infrastructure. They also need to review the development plans as if they were the enforcers of public agency codes and regulations.

Jurisdictions in the Washington suburbs of Virginia have taken an innovative approach to improving the quality of plans being submitted and reviewed by private and public sector engineers. A public/private partnership, Engineers and Surveyors Institute, has been formed to conduct education in the plan design and review process. It also conducts a peer review of plans prior to their submission to the jurisdictions as well as a peer review of the comments made by the jurisdiction reviewers. Design engineers of firms other than the one submitting the plan, as well as an independent engineer conduct this peer review.

Engineers completing the rigorous initial and annual continuing-education program and meeting experience qualifications are certified and entitled to an expedited re-

view process. Discrepancies in the plans being submitted can result in disciplinary action, including eventual disqualification for the expedited review. Plan completeness and quality have improved dramatically. Plans are being approved in one-third the time of previous submittals. The program encourages communication between submitting and reviewing engineers as well as among the various jurisdictions regarding the plan-review process and other technical concerns.

## REFERENCES

- Board for Architects, Professional Engineers, Land Surveyors, Certified Interior Designers, and Landscape Architects, *Rules and Regulations*, Commonwealth of Virginia, Richmond, Va., 1992.
- Bright, John K., *Reader's Write, Civil Engineering*, New York: 62 (8), American Society of Civil Engineers, New York, 1992.
- City of Dallas, Texas, Issue Paper: "Being User Friendly," Dallas, Texas, 1988.
- Commerce Clearing House, Inc., *What You Ought to Know About the ADA*, Commerce Clearing House, Inc., Chicago, Ill., 1992.
- DeJarnette & Paul, Inc., *The Professional Liability Perspective*, Richmond, Va., 1991.
- Engineers and Surveyors Institute, *Post Submission Conference Format, Recommended Plan Format for Fairfax County*, Engineers and Surveyors Institute, Fairfax, Va., 1990.
- Federal Register (56 (226), U. S. Government Printing Office, Washington, D.C., 1991.
- Garfield, William W., *Public Works*, 123 (9), Public Works Journal Corp., Ridgewood, N.J., 1992.
- International City Manager's Association, *Municipal Public Works Administration*, 5th ed., International City Manager's Association, Chicago, Ill., 1957.
- International City Manager's Association, April 14th Newsletter, Chicago, Ill., 1986.
- Virginia Water Control Board, Virginia Water Control Board, Richmond, Va., 1992.
- Washington Post, October 25 edition, Washington, D.C., 1992.



## CHAPTER 34

# ENVIRONMENTAL PERMITS

*Jeffrey M. Moran, PE*

### INTRODUCTION

The land development and construction industry is required to obtain permits for each phase of the development process, from land clearing and grading to final construction and occupancy. The majority of the permits required are related directly to the construction activity, building components and utilities. In addition, a parcel of land or construction activity may require one or more environmental permits because of the site location, site features, type of construction and/or the final use of the facility being constructed. Another factor may be the climate and the time of year that construction is underway. Individual permit requirements can vary significantly through state and local regulations, so the developer should be aware of the specific environmental laws in the areas where development is being performed.

An environmental permit refers to a permit required by an environmental law in order to regulate the emission or discharge of a pollutant or to have some authority over environmentally-regulated activities. Permits are used by federal, state, local, and municipal regulatory agencies to insure compliance with environmental laws. Most major, federal environmental laws require the issuance of a permit for specific activities.

Environmental permits, as presented in this chapter, are associated with air, water, and waste. It is the responsibility of the organization conducting the activity to insure that all necessary permits are obtained. The length and duration of permits issued under federal laws will differ depending on the type of permit and specific conditions imposed by the permit. Most environmental permits are issued for 1 to 3 and occasionally 10 years.

Obtaining environmental permits can be a difficult process. The first challenge is to understand the variety of environmental regulations. Once the potential permitting requirements are known, the next step is to complete the necessary studies required to obtain the permits and apply for them. Projects that have minimal impacts will take 3 to 6 months to obtain the necessary permits. Projects that have the potential of causing major environmental impacts will take significantly longer and may require significant study and documentation, such as an Environmental Impact Statement. The types of projects that may qualify for this level of documentation include roadways on new alignment or with numerous residential impacts, water-supply reservoir projects, and developments in sensitive areas that may harbor rare, threatened, or endangered species, developments within significant wetland areas, or with historic property impacts.

### PERMITTING PROCESS

The increased awareness of environmental impacts associated with land development resulted in increased permitting. If not incorporated into the development budget and schedule, these requirements can delay progress significantly or even stop work all together. Therefore, being aware of the environmental regulations and incorporating the requirements into the development plan is an essential part of the project.

The permitting process is constantly evolving, and regulatory agencies recognize the need to improve the efficiency and effectiveness of the process. Many complex issues must be addressed when pursuing reforms of environmental permits and significant challenges remain. Federal, state, and local regulatory agencies have reform efforts

in place to improve the process. Permit writers are finding new ways to improve environmental and economic results through innovative approaches to permitting. Some permits may contain incentives to reduce the amount of pollutants or allow for more operational flexibility; however, some can be labor-intensive during construction or operational phases of a project, requiring significant monitoring.

Although the requirements of an environmental permit may vary significantly from state to state, the permitting process is similar. Some permits may be relatively easy to obtain because of the simplicity of the activity and its potential for minimal effect on human health and the environment. For example, the permittee may simply announce the activity in the newspaper, complete the application, and within 30 days receive an approved permit. On the other hand, on a project where the potential impact to human health or the environment is significant, a much more detailed process may be required. This detailed permitting process can be divided into six distinct steps.

### Pre-application

Public involvement plays an important role for some permits. This is initiated by holding an informal public meeting, referred to as a preapplication meeting. The purpose of the meeting is to explain to the public the intended activity, including the specific proposed process associated with the pollutant and/or waste. The meeting is announced by placing a sign at the site, broadcasting on the television and/or radio, and placing an advertisement in the local newspaper. At the meeting, the organization (applicant) responsible for the activity explains the plans and provides the public with the opportunity to ask questions and make suggestions.

In some states, a pre-application meeting with the regulatory agencies is recommended once the preliminary plans for a site have been developed. This meeting assists both parties in understanding each other's concerns with the project and may save the developer significant time and trouble by pointing out the "red flags" of the proposed project.

### Permit Application

Following the pre-application meeting, the applicant fills out the permit application. Permits are often lengthy and include a description of the facility or development and address the following:

- A description of the project, including physical location, existing site conditions, distance to nearest water body, primary watershed, the outcome of previous coordination with the public or regulatory agencies
- How the facility will be designed, constructed, maintained, and operated to be protective of public health and the environment, including a demonstration of avoidance, minimization and mitigation

- How emergencies and spills will be handled, should they occur
- Where or how mitigation will be accomplished
- How the facility will clean up and finance any environmental contamination that occurs
- How the facility will close and clean up once it is no longer operating

### Receipt and Review Information

When the permitting agency receives a permit application, it notifies the public that it has received the application and places a copy of it in a public area for review. The permitting agency reviews the application to make sure it contains the information required by the regulations to determine if the facility can be built and operated safely. Some agencies do not post the project for public notice until they have made a preliminary decision to issue a permit or deny the permit. In these cases, what is posted is the tentative decision to issue or deny a permit.

### Revisions

If the permitting agency is not satisfied with the permit application, or finds the information submitted is insufficient, it may issue a Notice of Deficiency (NOD) or a request for additional information to the applicant. NODs identify and request that the applicant provide any missing information. During the application review and revision process, the permitting agency may issue several NODs. Each time the permitting agency receives a response from the applicant, it reviews the information and, if necessary, issues NODs until it considers the application to be complete. Given the complex and technical nature of the requested information, the review and revision process can take several years to complete, depending upon the type of project.

### Public Review

When the revisions are complete, the agency makes a preliminary decision about whether to issue or deny the permit. If the agency decides that the application is complete and meets appropriate standards, the agency issues a draft permit containing the conditions under which the facility can operate if the permit receives final approval. If the permitting agency determines that an applicant cannot provide an application that meets the standards, the agency tentatively denies the permit and prepares a "notice of intent to deny".

The permitting agency announces its decision to the public by placing a notice in a local newspaper and broadcasting it on television and/or radio. It also issues a fact sheet to explain the decision. Once the notice is issued, the public typically has 30 to 45 days to comment on the de-

cision. The public may request a hearing by contacting the permitting agency. The permitting agency may also hold a hearing at its own discretion. The agency typically gives 30-day public notice before the hearing.

### Final Permit

After considering public comments, the permitting agency reconsiders the draft permit or the notice of intent to deny the permit. The agency then issues a “response to public comments,” specifying any changes made to the draft permit. Finally, the agency issues the final permit or denies the permit.

Even after issuing a permit, the permitting agency may continue to monitor the construction and operation of the facility to make sure they are consistent with federal and state rules and with the application. Several additional steps can also take place after the permit is issued:

- *Permit appeals*—facility owners and the public both have a right to appeal the final permit decision. Administrative law judges usually decide upon the appeal.
- *Permit modifications*—if a facility changes its management procedures, mechanical operations, or the waste it generates or handles, then it must secure a permit modification. For modifications that significantly change the facility operations, the public must receive early notice and have a chance to participate and comment. For minor modifications, the facility must notify the public within a specified period of time prior to making the change.
- *Permit renewals*—the permitting agency can renew permits that are due to expire. Permit holders that are seeking a permit renewal must follow the same procedures as a facility seeking a new permit.
- *Permit terminations*—if a facility violates the terms of its permit, the permitting agency can terminate the permit.

### REGULATORY ENFORCEMENT

There are various methods used to monitor conditions imposed under environmental permits. Applicants may be required to sample or continuously monitor emissions or discharges and report the results to the permitting authority. In addition, they are usually required to notify the permitting authority when they are not in compliance with the requirements of the permit. The regulatory agency may also use one of their own inspectors to inspect periodically the activity being permitted as a means of monitoring compliance.

Federal laws provide federal and authorized state regulatory agencies with various methods of taking enforcement actions against violators of permit requirements. For example, the U.S. Environmental Protection Agency and state

regulatory agencies may issue administrative orders that require facilities to correct violations and that assess monetary penalties. The laws also allow federal and state agencies to pursue civil and criminal actions that may include mandatory injunctions or penalties, as well as jail sentences for persons found willfully violating requirements and endangering the health and welfare of the public and the environment. If any member of the general public finds that a facility is violating its permit, that member can independently start a legal action, unless federal or state regulatory agencies have already taken an enforcement action. EPA has an enforcement division dedicated to providing national direction, leadership, and consistency in case selection, case development, and resolution and appeal of civil, judicial and administrative enforcement actions pursuant to its statutory authority.

### ENVIRONMENTAL-PERMITTING PROGRAMS

Environmental permits can be categorized in a wide variety of ways, including environmental medium, regulatory agency administering the permits, or the activity. This section provides brief descriptions of some of the more common environmental-permitting programs associated with land development.

#### Air

Air pollution is caused by many different sources, including naturally occurring events and man-made sources. The cumulative effects of the air pollution caused by these sources can be significant. EPA or its delegated state authority has various programs in place under the Clean Air Act to minimize air pollution and protect and improve air quality. The specific standards for air quality vary depending on location and the activity being performed. The land developer is most likely to be faced with issues associated with fugitive dust, open burning, and construction noise regarding air quality. In some instances, a development proposed in an existing carbon monoxide or ozone non-attainment zone may have to complete complex air-quality modeling in order to verify that it will not increase these constituents significantly.

#### Fugitive Dust

Large construction projects can disturb significant areas of land, potentially leaving the soil exposed for long periods of time. These construction sites can be a significant source of fugitive dust from wind and soil erosion from surface-water runoff. When construction sites are located in or near cities, they can be major contributors to overall air-quality problems for the metropolitan areas. Therefore, these construction projects may require plans for meeting federal or state air-quality standards.

The goal of the contractor is to complete the construction project as quickly as possible and re-establish protective cover over the exposed soil. This goal is compatible



with fugitive-dust control because the shorter the period of exposure, the less opportunity for wind and water erosion to occur. However, contractors sometimes view additional steps for dust control as costly, time-consuming and, therefore, incompatible with their primary goal.

EPA or their designated local authority currently regulates fugitive dust on construction sites larger than 5 acres. Contractors must file a fugitive-dust emission-control plan to be in compliance. Smaller sites may be subject to local regulations.

Water is often used on construction sites to minimize fugitive dust. This can be effective if applied often enough. For sites that will not be paved, timely revegetation is very important. Placing sod is the quickest approach, but the cost is usually prohibitive for large areas. For areas that will be seeded, the use of mulch provides some immediate protection and improves the chance of getting a good grass stand quickly. Light mulches such as straw should be tacked in place, either mechanically or by application of a chemical tacking agent.

## Noise

Noise is considered a pollutant and can be a public nuisance. While its physical and emotional effects are difficult to define quantitatively, the noise level itself can be measured and, therefore, regulated through permits. Cities and local municipalities typically handle noise permits and ordinances, so they can vary significantly, depending on location. The characteristics and conditions that are generally considered when determining if a noise is a public nuisance include:

- Noise level
- Nature of the noise (natural versus unnatural)
- Origin of the noise
- Proximity to receptors
- The time of day (or night) the noise is generated
- Duration of the noise
- Whether the noise is recurrent, intermittent or constant

These characteristics typically are evaluated on the applications for noise permits. In addition, a noise-impact assessment may be required in the application for the permit. The impact assessment evaluates the potential impacts of the noise to local receptors such as residential buildings, schools, and offices. If it is determined that potential noise effects warrant evaluation of impacts and mitigation measures, an environmental impact statement may be required; this will identify the effectiveness of various mitigation measures that can be incorporated to obtain a noise permit if there are a significant number of receptors affected or if federal funds are involved.

## Water

For the purpose of addressing water-related permits, this chapter categorizes water into surface water, wetlands and ground water. All three are important natural resources that must be managed in a responsible manner to insure their quality and availability for the future.

**Surface Water.** Surface-water pollution can result in conditions that are unsafe for drinking, fishing, swimming, and other activities. Polluted storm-water runoff is a leading cause of impairment to the nearly 40% of water bodies surveyed in the United States that do not meet water-quality standards.

Mandated by Congress under the Clean Water Act, the National Pollutant Discharge Elimination System (NPDES) storm-water program is a national program for addressing the non-agricultural sources of storm-water discharges that adversely affect the quality of our nation's waters.

The Clean Water Act requires an approved NPDES permit to discharge pollutants through a point source to surface waters. The permit provides limits on what is being discharged, monitoring and reporting requirements, and other provisions. The program uses the NPDES permitting mechanism to regulate point sources that discharge pollutants into waters of the United States. Point sources are discrete conveyances such as pipes or man-made ditches. In most cases, the NPDES permit program is administered by authorized states.

Construction activities are considered an industrial activity, and as such an NPDES permit may be required. Specifically, an NPDES permit is required if the construction results in the disturbance of more than 5 acres of total land area. Construction activities that require a permit may include clearing, grading, and excavation. These activities typically are associated with road building, borough pit excavation, residential housing and office building construction, light industrial construction, or facility demolition. In some states, a permit may not be required if the runoff does not discharge into a waterway, as in the case where it evaporates from a catch basin or similar water body.

There are two permit processes applicable to construction activities: the Individual-Permit process and the General-Permit process. The Individual Permit, as the name implies, is a permit that is issued on a case by case basis. The General-Permit process, when available, is the easiest of the two processes. NPDES is a federal storm-water management program; however, many states have primacy, meaning that they are authorized to administer the NPDES program for facilities within their boundaries. To obtain primacy, the state must have enacted storm-water regulations that are at least as stringent as those enacted by the EPA. Permittees within states that have primacy must obtain their permits from the state NPDES program. If the state has obtained primacy and has also obtained a General Permit for Construction Activities from the EPA, then the state may

in turn issue the General Permits for Construction Activities. If the state has primacy but has not obtained a General Permit for Construction Activities from the EPA, then it may not issue General Permits, and the permittee must apply to the state for an Individual Permit. If the state does not have permitting authority, the permittee must apply directly to the EPA for either a General Permit or an Individual Permit.

The application for an individual permit is a rather extensive process, and the regulations should be reviewed in detail before beginning. The application for the Individual Permit is accomplished by submitting a permit application 90 days prior to construction. The permit application must include such information as:

- The results of quantitative testing data
- Information on significant spills or leaks in the area for the last 3 years
- A narrative description of the construction activities and a site map showing topography
- Site characteristics, and a description of effective management practices

For construction operators of sites located in the areas where the EPA's General Permits for Construction Activities apply, the steps for obtaining coverage are listed below.

1. Read the applicable Construction General Permit (CGP) fact sheet and permit requirements. The applicable CGP to use depends on the construction operator's NPDES permitting authority. The EPA's national CGP applies only in EPA regions.
2. Develop and implement a Storm-Water Pollution-Prevention Plan (SWPPP). SWPPPs are required of all CGP applicants. Several items are required in a SWPPP, including: site maps showing drainage and discharge locations, and the locations of control measures; a description of the site and of the pollution prevention Best Management Practices (BMPs); and inspection procedures and reports. Guidance on SWPPP development and the implementation of appropriate BMPs is available from the EPA. Note that SWPPPs must be developed and implemented in accordance with the CGP prior to submitting the Notice of Intent.
3. Complete and submit a Notice of Intent (NOI) for Construction Activity. This must be filled out and submitted to the EPA's NOI Processing Center at least 2 days prior to the commencement of construction activity. It is important to note that signing and submitting the NOI form obligates the permittee to comply with the terms of the Construction General Permit. Once the NOI is received, the NOI Processing Center will send the permittee notice of its permit number. Permittees may contact the NOI processing center to determine the status of a particular NOI. The EPA's NOI

requires certification that the industrial activity will not impact endangered or threatened species protected under the Endangered Species Act. This certification is unique to the EPA's NOI and is not a requirement of most NPDES-delegated states' NOIs. For a current list of these species and their locations nationwide (by county), refer to the Office of Wastewater Management's Endangered Species List. As with most NPDES programs, storm-water permits may be issued through one of the EPA's ten regions, or through an authorized state/territory NPDES permitting authority. Regardless of who issues the permit, the standards set forth must, at a minimum, agree with the federal version of the permit. States can create more stringent requirements or alter certain procedures associated with the permitting process. For more information about a particular state program or permit, contact the appropriate state.

To terminate coverage, a Notice of Termination (NOT) for Industrial Activity must be submitted to the EPA's NOI Processing Center. A permittee may submit a NOT when:

- Disturbed soils at the construction site have finally been stabilized and temporary erosion and sediment-control measures have been removed (or will be removed at an appropriate time)
- Storm-water discharges have been eliminated, or
- The permittee is no longer an operator of the site

Under the Phase II Rule, NPDES permitting authorities have the option of providing a waiver from the requirements to operators of "small" construction activity who could certify to one of two conditions:

- Low predicted rainfall (i.e., activity occurs during a negligible rainfall period), where the rainfall erosivity factor ("R" in the revised Universal Soil Loss Equation) is less than 5 during the period of construction activity
- A determination that storm-water controls are not necessary based on either:
  - Total Maximum Daily Load (TMDL) approved or established by the EPA that addresses the pollutant(s) of concern for construction activities; or
  - For non-impaired waters that do not require TMDLs, an equivalent analysis that determines allocations for small construction sites for the pollutant(s) of concern or that determines that such allocations are not needed to protect water quality based on the condition of exiting in-stream concentrations, expected growth in pollutant contributions from all sources, and a margin of safety

Waivers are not available for any construction activity disturbing 5 acres or greater, or less than 5 acres if part of a common plan of development or sale (or if designated for

permit coverage by the NPDES permitting authority). This regulation is slated to change in 2003, reducing the acreage from 5 acres to 1 acre.

### Wetlands

Wetlands can be significantly affected by land development. Their natural ability to purify water by reducing nutrient loading, chemical waste and turbidity makes them a valued resource to the natural environment. Section 404 of the Clean Water Act establishes programs to regulate the discharge of dredged and fill material into waters of the United States, including wetlands. Section 401 of the Clean Water Act establishes a program to regulate the effects of any discharges into waters of the U. S., including wetlands.

The simplest strategy for dealing with wetlands on a particular project is to know the location of the jurisdictional areas (i.e., those areas regulated by the United States Army Corps of Engineers [Corps]) and avoid them in the development process. However, in many instances, total avoidance is not practical, and some involvement with the regulatory process is inevitable.

The degree of involvement in the regulatory process depends on the amount of jurisdictional area likely to be affected. Most regulatory programs have minimum thresholds for allowable wetland impacts that, once crossed, propel one into the regulatory process. Generally, a zero threshold is established for tidal wetlands and non-tidal wetlands located below the headwaters. Any proposed fill in these areas requires permits. Headwaters are defined in the Corps regulations as the area upstream of the point where a stream's average annual flow falls below 5 cubic feet per second. Wetlands and their regulatory issues are discussed elsewhere in this book, but generally, minor impacts to headwaters and headwater wetlands are allowed under a general permit program implemented through the U. S. Army Corps of Engineers Nationwide Permit Program Regulations (33 CFR Part 330). Be aware however, that most states have implemented more stringent laws regarding fill, piping, or excavation in waters and wetlands.

### Ground Water

Ground water accounts for approximately two thirds of the world's available freshwater supply and is used for drinking water by more than 50% of the people in the United States. Ground water contamination can occur when products such as gasoline, oil, road salts and chemicals are released into the environment and enter the ground water, causing it to become unsafe and unfit for human use. Some of the major sources of these products, called contaminants, are storage tanks, septic systems, hazardous-waste sites, landfills, and the widespread use of road salts and chemicals. Ground water provides an estimated:

- 22% of all freshwater withdrawals
- 34% of agricultural use (mostly for irrigation)

- 40% of the public water-supply withdrawals
- 53% of all drinking water for the total population
- 97% of drinking water for the rural population

The Safe Drinking Water Act was established to protect the quality of drinking water in the United States. This law focuses on all waters actually or potentially designed for drinking use, whether from above-ground or underground sources. The Act authorized the EPA to establish safe standards of purity. State governments, which assume this power from the EPA, also encourage attainment of secondary standards (nuisance-related). The Comprehensive State Ground Water Protection Programs (CSGWPPs) established a partnership between the states, tribal governments, and the EPA to implement the EPA's ground water protection goal and principles.

### Waste

Waste is generated at construction sites where there is clearing, excavation, construction, renovation, or demolition of structures. This waste is considered construction and demolition debris. Some of this debris can be managed onsite, and the remaining waste must be properly handled and disposed of at the appropriate permitted landfill. For example, uncontaminated rocks, bricks, concrete, road demolition debris, and soil are not subject to solid-waste regulations and, therefore, do not have to be disposed of at a landfill. This debris may be buried onsite or may be removed from the site to be used for off-site fill as long as it is not placed in a waterway or wetland. The remaining construction debris must be characterized properly, transported and disposed of in accordance with solid and hazardous-waste regulations.

New construction, especially on previously undeveloped land, can generate vegetative waste that may be banned from solid-waste landfills. These wastes may be taken to a composting facility for disposal or dealt with on site by burning, chipping, grinding, or shredding. An air permit may be required if the waste is burned on site, and a registration process may be required if the waste is chipped, ground, or shredded, depending on the quantity of vegetative waste that was generated.

Landfills that receive construction and demolition debris are not required to be constructed to the same standards as solid-waste landfills under Subtitle D of the Resource Conservation and Recovery Act (RCRA). A subtitle D landfill requires a synthetic bottom liner and ground water monitoring. Therefore, disposing of waste in a Subtitle D landfill is more expensive. Hazardous waste is required to be disposed of at a Subtitle C landfill, permitted to receive specific types of hazardous waste. Disposing of waste at a subtitle C landfill is more expensive than subtitle D landfills.

The EPA has promulgated regulations (40 CFR 61, Subpart M) in the National Emission Standards for Hazardous

Air Pollutants (NESHAPS) requiring structures proposed for renovation or demolition to be tested for asbestos. The disposal criteria for asbestos material is determined by the regulating agency based on the quantity and type of asbestos found in the structure(s). Some types of regulated asbestos-containing materials may be disposed of in a construction/debris landfill. For example, roofing shingles and resilient floor coverings that are deemed to be in "good condition" (non-friable) may be disposed of in a construction/debris landfill.

Disposal of lead-based paint requires the classification of the waste. If the lead-based paint is generated from a structural renovation or demolition project, it is considered "debris" and can be disposed of in a construction/debris landfill or at a municipal solid-waste landfill. If the lead-based paint is a concentrated waste derived from scrapings,

sludge, dust, or blast grit, it may be required to be classified and handled in accordance with RCRA regulations.

### State Permitting Contacts

Determining the types of environmental permits that are required on a construction project can be a very complex and time-consuming process. Some permits are handled by federal agencies while others are the responsibility of the individual state or local municipality. To obtain additional information on federal programs, visit their websites. This can be accomplished by searching EPA or the names of the programs, most of which are linked. The table below provides a list of website addresses for permitting contacts, by state, should additional environmental-permitting information be required.

**TABLE 34.1 State Permits Contacts**

STATE	PERMITTING CONTACTS/LINKS
Alaska	Permit Questionnaire and Contacts <a href="http://www.state.ak.us/local/akpages/ENV.CONSERV/dsps/question3.htm">http://www.state.ak.us/local/akpages/ENV.CONSERV/dsps/question3.htm</a>
Alabama	ADEM Contacts <a href="http://www.adem.state.al.us/contacts.html">http://www.adem.state.al.us/contacts.html</a>
Arizona	Compliance Center <a href="http://www.adeq.state.az.us/do/compliance.htm">http://www.adeq.state.az.us/do/compliance.htm</a> Waste Program Division Contacts <a href="http://www.adeq.state.az.us/waste/hazwaste/index.htm">http://www.adeq.state.az.us/waste/hazwaste/index.htm</a> Water Quality Division Point Source <a href="http://www.adeq.state.az.us/water/permits/index.htm">http://www.adeq.state.az.us/water/permits/index.htm</a>
California	Permitting Assistance Center Contacts <a href="http://www.calgold.ca.gov/pac.asp">http://www.calgold.ca.gov/pac.asp</a> Regional Permit/Compliance Assistance Centers <a href="http://www.baaqmd.gov/enf/complnce/regpcap.htm">http://www.baaqmd.gov/enf/complnce/regpcap.htm</a>
Colorado	Environmental Customer Assistance Center <a href="http://www.cdphs.state.co.us/ic/ecashom.html">http://www.cdphs.state.co.us/ic/ecashom.html</a>
Connecticut	DEP Permitting Contacts <a href="http://dep.state.ct.us/pao/sourcinf.htm">http://dep.state.ct.us/pao/sourcinf.htm</a>
Delaware	Department of Natural Resources and Environmental Control Business and Permitting Services <a href="http://www.dnrec.state.de.us/busperm.htm">http://www.dnrec.state.de.us/busperm.htm</a>
Florida	Department of Environmental Protection Contacts <a href="http://www.dep.state.fl.us/officsec/ombud/permit__c.htm">http://www.dep.state.fl.us/officsec/ombud/permit__c.htm</a>
Georgia	Georgia Environmental Protection Division Contacts <a href="http://www.ganet.org/dnr/environ/pubnote/contacts/">http://www.ganet.org/dnr/environ/pubnote/contacts/</a>

**TABLE 34.1 State Permits Contacts (Continued)**

<b>STATE</b>	<b>PERMITTING CONTACTS/LINKS</b>
Hawaii	DOH Permitting Contacts <a href="http://www.state.hi.us/doh/permits/emd__permits.html">http://www.state.hi.us/doh/permits/emd__permits.html</a>
Iowa	Environmental Protection Division Roster <a href="http://www.state.ia.us/government/dnr/organiza/epd/roster/rosters.htm">http://www.state.ia.us/government/dnr/organiza/epd/roster/rosters.htm</a>
Idaho	Division of Environmental Quality Contacts <a href="http://www.state.id.us/deq/reg__map1.htm">http://www.state.id.us/deq/reg__map1.htm</a>
Illinois	Directory <a href="http://www.epa.state.il.us/quick-answer-directory/index.html">http://www.epa.state.il.us/quick-answer-directory/index.html</a>
Indiana	IDEM Offices and Staff <a href="http://www.state.in.us/idem/offices.html">http://www.state.in.us/idem/offices.html</a> Permit Assistance <a href="http://www.muncie.com/permit.asp">http://www.muncie.com/permit.asp</a>
Kansas	Department of Health and Environment Service Guide <a href="http://www.kdhe.state.ks.us/environment/servguid.html">http://www.kdhe.state.ks.us/environment/servguid.html</a>
Kentucky	Division of Air Quality Contacts <a href="http://www.nr.state.ky.us/nrepc/dep/DAQa.htm">http://www.nr.state.ky.us/nrepc/dep/DAQa.htm</a> Environmental Permitting Programs <a href="http://www.nr.state.ky.us/nrepc/dep/dep2.htm">http://www.nr.state.ky.us/nrepc/dep/dep2.htm</a> Division of Waste Management Contacts <a href="http://www.nr.state.ky.us/nrepc/dep/DWM.htm">http://www.nr.state.ky.us/nrepc/dep/DWM.htm</a> Division of Water Contacts <a href="http://www.nr.state.ky.us/nrepc/dep/DOWa.htm">http://www.nr.state.ky.us/nrepc/dep/DOWa.htm</a>
Louisiana	Department of Environmental Quality Contacts <a href="http://www.deq.state.la.us/organization/officdir.htm">http://www.deq.state.la.us/organization/officdir.htm</a>
Massachusetts	Department of Environmental Protection InfoLine <a href="http://www.state.ma.us/dep/infoline.htm">http://www.state.ma.us/dep/infoline.htm</a> Department of Environmental Protection Regional Service Centers <a href="http://www.state.ma.us/dep/service.htm">http://www.state.ma.us/dep/service.htm</a>
Maryland	Environmental Permits Service Center <a href="http://www.mde.state.md.us/permit/index.html">http://www.mde.state.md.us/permit/index.html</a>
Maine	Department of Environmental Protection Contacts <a href="http://www.state.me.us/dep/whoswho.htm">http://www.state.me.us/dep/whoswho.htm</a>
Michigan	Department of Environmental Quality Information Sources Guide-Licenses and Permits <a href="http://www.deq.state.mi.us/dept/info">http://www.deq.state.mi.us/dept/info</a>
Minnesota	Minnesota Pollution Control Agency Water Permits Contacts <a href="http://www.pca.state.mn.us/water/permits.html">http://www.pca.state.mn.us/water/permits.html</a> Pollution Control Agency Air Permit Contacts <a href="http://www.pca.state.mn.us/air/permits/index.html">http://www.pca.state.mn.us/air/permits/index.html</a>
Missouri	Department of Natural Resources Technical Assistance <a href="http://www.dnr.state.mo.us/technical.htm">http://www.dnr.state.mo.us/technical.htm</a>

**TABLE 34.1 State Permits Contacts (Continued)**

<b>STATE</b>	<b>PERMITTING CONTACTS/LINKS</b>
Mississippi	Environmental Resource Center <a href="http://www.deq.state.ms.us/newweb/erchome.nsf/pages/Directory">http://www.deq.state.ms.us/newweb/erchome.nsf/pages/Directory</a>
Montana	Montana Department of Environmental Quality Staff Directory <a href="http://www.deq.mt.gov/staffdir.htm">http://www.deq.mt.gov/staffdir.htm</a>
North Carolina	Air Quality Contacts <a href="http://daq.state.nc.us/Staff">http://daq.state.nc.us/Staff</a> DENR Regional Offices <a href="http://www.ehnr.state.nc.us/EHNR/files/regnofts.htm">http://www.ehnr.state.nc.us/EHNR/files/regnofts.htm</a>
North Dakota	Department of Environmental Health Contacts <a href="http://www.health.state.nd.us/ndhd/contact.htm">http://www.health.state.nd.us/ndhd/contact.htm</a>
New Hampshire	Department of Environmental Services Contacts <a href="http://www.des.state.nh.us/comments.htm">http://www.des.state.nh.us/comments.htm</a>
New Jersey	Division of Water Quality Permit Contacts <a href="http://www.state.nj.us/dep/dwq/gps.htm">http://www.state.nj.us/dep/dwq/gps.htm</a> Air Quality Permitting Program Contacts <a href="http://www.state.nj.us/dep/aqpp/contact.html">http://www.state.nj.us/dep/aqpp/contact.html</a> Pollution Prevention Permit Contacts <a href="http://www.state.nj.us/dep/opppc/info.htm">http://www.state.nj.us/dep/opppc/info.htm</a>
New Mexico	Environmental Protection Division Contacts <a href="http://www.nmenv.state.nm.us">http://www.nmenv.state.nm.us</a>
Nevada	Division of Environmental Protection Contacts <a href="http://www.state.nv.us/ndep/admin/phone.htm">http://www.state.nv.us/ndep/admin/phone.htm</a>
New York	DEC Regional Environmental Permit Offices <a href="http://www.dec.state.ny.us/website/dcs/EP_REGIONS/regional_offices.html">http://www.dec.state.ny.us/website/dcs/EP_REGIONS/regional_offices.html</a>
Ohio	Division of Hazardous Waste Management <a href="http://www.epa.ohio.gov/dhwm/distSCO.html">http://www.epa.ohio.gov/dhwm/distSCO.html</a> District Offices Contacts <a href="http://www.epa.state.oh.us/dist.html">http://www.epa.state.oh.us/dist.html</a>
Oklahoma	Department of Environmental Quality Inquiry Contact List <a href="http://www.deq.state.ok.us/EXTERNAL%20INQUIRY%20REFERRAL%20LIST.htm">http://www.deq.state.ok.us/EXTERNAL%20INQUIRY%20REFERRAL%20LIST.htm</a>
Oregon	Permits Handbook <a href="http://www.deq.state.or.us/od/permithndbk/permit.htm">http://www.deq.state.or.us/od/permithndbk/permit.htm</a>
Pennsylvania	Department of Environmental Protection Contacts <a href="http://www.dep.state.pa.us/dep/DEPPHO2.htm">http://www.dep.state.pa.us/dep/DEPPHO2.htm</a>
Rhode Island	Department of Environmental Management Contacts <a href="http://www.state.ri.us/dem/phone/topics.htm">http://www.state.ri.us/dem/phone/topics.htm</a>
South Carolina	Environmental Permit Types & Contacts <a href="http://www.state.sc.us/dhec/eqc/admin/html/permtypes.html">http://www.state.sc.us/dhec/eqc/admin/html/permtypes.html</a>

**TABLE 34.1 State Permits Contacts (Continued)**

STATE	PERMITTING CONTACTS/LINKS
South Dakota	Environmental Permitting and Regulation Guide Department Directory <a href="http://www.state.sd.us/state/executive/denr/enviro/deptlist.htm">http://www.state.sd.us/state/executive/denr/enviro/deptlist.htm</a>
Tennessee	Department of Environment and Conservation Contact List <a href="http://www.state.tn.us/environment/contact.htm">http://www.state.tn.us/environment/contact.htm</a>
Texas	Natural Resource Conservation Commission Contacts <a href="http://www.tnrcc.state.tx.us/admin/directory/phone/index.html">http://www.tnrcc.state.tx.us/admin/directory/phone/index.html</a>
Utah	Department of Environmental Quality Contacts <a href="http://www.eq.state.ut.us/eqoas/org/deqqlnce.htm">http://www.eq.state.ut.us/eqoas/org/deqqlnce.htm</a> Environmental Permit Contacts <a href="http://www.eq.state.ut.us/eqoas/org/deqqlnce.htm">http://www.eq.state.ut.us/eqoas/org/deqqlnce.htm</a>
Virginia	Virginia Department of Environmental Quality Directory <a href="http://www.deq.state.va.us/info/direct.html">http://www.deq.state.va.us/info/direct.html</a>
Vermont	Environmental Board <a href="http://www.anr.state.vt.us/fguide/map__reng.html">http://www.anr.state.vt.us/fguide/map__reng.html</a>
Washington	State Agency Contacts <a href="http://www.wa.gov/ecology/sea/pac/handbook/low/contacts.html">http://www.wa.gov/ecology/sea/pac/handbook/low/contacts.html</a>
Wisconsin	Department of Natural Resources Contacts <a href="http://www.dnr.state.wi.us/org/caer/cs/subjectlist">http://www.dnr.state.wi.us/org/caer/cs/subjectlist</a>
West Virginia	Division of Environmental Protection Contacts <a href="http://www.dep.state.wv.us/phone.html">http://www.dep.state.wv.us/phone.html</a>
Wyoming	Department of Environmental Quality Contacts <a href="http://deq.state.wy.us/contacts.htm">http://deq.state.wy.us/contacts.htm</a>

**REFERENCES**

- U.S. Environmental Protection Agency, Office of Solid Waste, *The Hazardous Waste Permitting Process*, EPA530-F-96-007, U.S. Environmental Protection Agency, Washington, D.C., June, 1996.
- U.S. Environmental Protection Agency, Office of Policy, Economics, and Innovation, Washington, D.C., May, 2001.
- U.S. Environmental Protection Agency, Office of Air Quality Planning and Standards, Washington, D.C.
- U.S. Environmental Protection Agency, Environmental Permitting Clearinghouse, Washington, D.C., January, 2001.
- U.S. Environmental Protection Agency, *Section 404 of the Clean Water Act, An Overview*, U.S. Environmental Protection Agency, Washington, D.C., May 25, 1999.
- U.S. Environmental Protection Agency, Region 9, *Solid Waster Program: Construction and Demolition Debris*, U.S. Environmental Protection Agency, Washington, D.C., November, 1999.
- Indiana Department of Environmental Management, March, 2001.
- U.S. Environmental Protection Agency, Office of the Administrator, *Final Comprehensive State Ground Water Protection Program Guidance*, EPA 100-R-93-001, U.S. Environmental Protection Agency, Washington, D.C., 1992.
- New York Department of Environmental Conservation, Division of Environmental Permits, *Guidance for Assessing and Mitigating Noise Impacts*, New York.
- Agricultural Publication G1885, *Fugitive Dust: Nonpoint Sources*, October, 1999.
- Tennessee Department of Environmental Conservation, *Solid Waste Management Permit By Rule*, Tenn., March, 2000.
- South Dakota Department of Natural Resources, Waste Management Program, *Solid Waste Permit*, S.D., July, 2001.
- U.S. Environmental Protection Agency, Office of Water, *National Pollutant Discharge Elimination System Permit Program*, July, 2001.

**PART VII**

# CONSTRUCTION





## CHAPTER 35

# CONSTRUCTION STAKEOUT SURVEYS

Mary Munson Rouse Root, L.S

### INTRODUCTION

While land planners and engineers propose a man-made order of interweaving utility networks, roads, and buildings, surveyors bring the plans to reality. The surveyor executes the mathematical calculations associated with the layout of the site and the field surveys that accurately reflect these calculations. Legal and practical standards demand precision in two directions simultaneously: the horizontal and the vertical. To fulfill these requirements, a surveyor must research the existing land records, apply coordinate geometry, and prepare a boundary and topographical survey before any site construction can commence. In construction stakeouts, surveyors align the design intentions with the field conditions and provide a “last line of defense” against potentially costly design errors. A licensed professional surveyor must interpret and implement various federal, state and local laws and restrictions, satisfy legal and ethical State Board requirements, and produce trustworthy data. Once the plans are finalized and construction contracts are awarded, surveyors then begin the construction-layout process.

Three frequently used terms relating to construction surveying are *location*, *stakeout* and *as-built*.

*Location* refers to the type of survey where a specific, existing feature is measured both horizontally and vertically from known control points. The data is then computed, plotted and incorporated into the design.

*Stakeout* is the field process in which the surveyor provides in-ground reference markers for the builder that physically delineate the placement of utilities, streets, and buildings. Construction stakeout, or layout, is necessary in every phase of construction, from the initial clearing of the site through the setting of final property corners.

*As-built* refers to a post-construction survey of a site, usually performed after the utilities are installed or the buildings are erected. This survey is used to determine if the structure is within the easement and if it is built according to plan. The accuracy and quality of new sewer lines, waterlines, and streets, as well as their adherence to approved construction plans, must be proven to the municipality before it will assume responsibility and control of the new system. As-built surveys also confirm that the constructed project complies with applicable codes and ordinances.

Typically, the construction-stakeout sequence begins with clearing and rough grading stakeouts, followed by stakeouts for the sanitary, storm and water lines, as well as the curb and gutter, building footprints, power, pavement, and property corners.

This chapter outlines the typical construction-stakeout procedures and technologies most typically employed by today's surveyors. However, it should be noted that advancing technologies have the potential to alter the construction-stakeout process radically. For example, the Global Positioning System (GPS) can establish positions on the earth's surface with reasonable accuracy. Many surveyors are now using GPS to conduct construction stakeout, although certain factors, such as the cost of the equipment, and limitations, especially in vertical accuracies, have slowed what would otherwise be widespread use. Undoubtedly these disadvantages will soon be overcome.

### Construction Stakeout Fundamentals

**Stakeout Markers.** For construction purposes, the marker of choice usually consists of a hub, a tack, and a stake. A hub is a 2-in by 2-in wood peg, approximately 8

in long, with a pointed end that allows it to be driven easily into the ground. The hub is driven flush with the ground. The measurement point is indicated by a surveyor's tack, which is driven flush into the hub. The vertical point of reference coincides with the highest part of the hub. In data collection, the tack is located horizontally and vertically, and this is duly noted in the surveyor's notes. A stake, made of plastic or wood, is a slat with one pointed end to be driven into the ground. It bears written information expressed in symbols, numbers and abbreviations, and it marks or "guards" the true horizontal and vertical point, which is the hub. The guard stake is placed such that the side with the offset distance faces the item being staked. Other types of markers used in stakeout include steel bars, spikes, nails with flagging or washers, chiseled marks in concrete, drill-holes, and so forth.

Stakeout markers and accompanying information are used to indicate points of construction to the contractor, but they may also be used as reference control points for subsequent surveys. Job sites are replete with construction activities, and, as a result, the hubs and guards are often buried, dug up or otherwise disturbed. Frequently, this cannot be avoided. Cooperation between the surveyor and contractor is necessary to schedule the construction stakeout for a specific phase and to place the stakes in areas where they are not disturbed prematurely by other construction activities but remain in areas convenient to the contractor (see Figure 35.1).

**Offsets.** If the stakeout markers were placed on an actual centerline, they would be lost because of excavation and/or grading and general construction traffic. Therefore, stakeout markers are set along lines parallel or concentric

to actual centerlines to preserve vital reference points. The distance from the actual centerline to the stakeout line is the offset distance. The offset's length is controlled by depth of excavation, width of the excavating machinery, the location of construction materials, traffic, and sometimes by governing authorities. The offset distance is marked on the guard stake, and the position of the offset is labeled left or right as determined by the direction of the increased stationing.

The offsets must remain undisturbed but not too far for convenient use. Quite often a contractor will request a specific offset distance. When the work is ordered, it is wise to confer with the foreman upon arrival at the site. The plan and profiles should be consulted to determine the amount of cut, as that determines the top width of the trench and the amount of excavated earth that will be piled adjacent to it (see Figure 35.2).

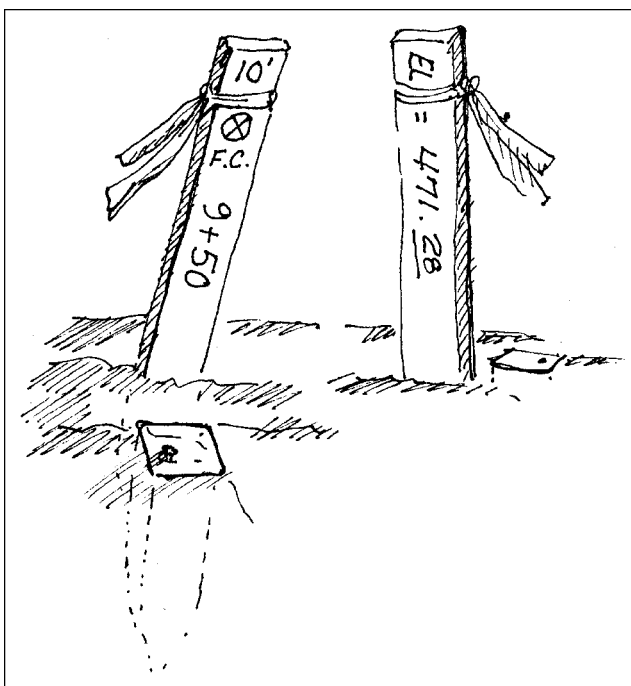
Offset stakes typically are labeled in this order: distance of offset, identification of object being offset, and the station. Vertical information is written on the back of the guard stake. Symbols and abbreviations typically are used. Some examples are provided in Table 35.1.

**Stationing.** Stationing is a method of designating distances from known points on the site, usually measured in multiples of 100 ft. The beginning point typically is designated as station 0, written as 0+00. A point 100 ft away would be station 1, written as 1+00. However, to avoid the use of negative stations (e.g. -1+50), the beginning station is sometimes designated as an arbitrary full station other than 0 (e.g. 10+00). Stations at even multiples of 100 ft are considered full stations. Intermediate points between full stations are called pluses; for example, a point 34.23 ft from station 1 is station 1+34.23 (pronounced as station "one plus thirty four point two three"). Stationing is the common method used on centerlines of streets and utilities and along the baselines of streams. For sanitary-sewer systems, storm-sewer systems, and streams, a common practice is to orient the stationing system such that the stations increase in the upstream direction.

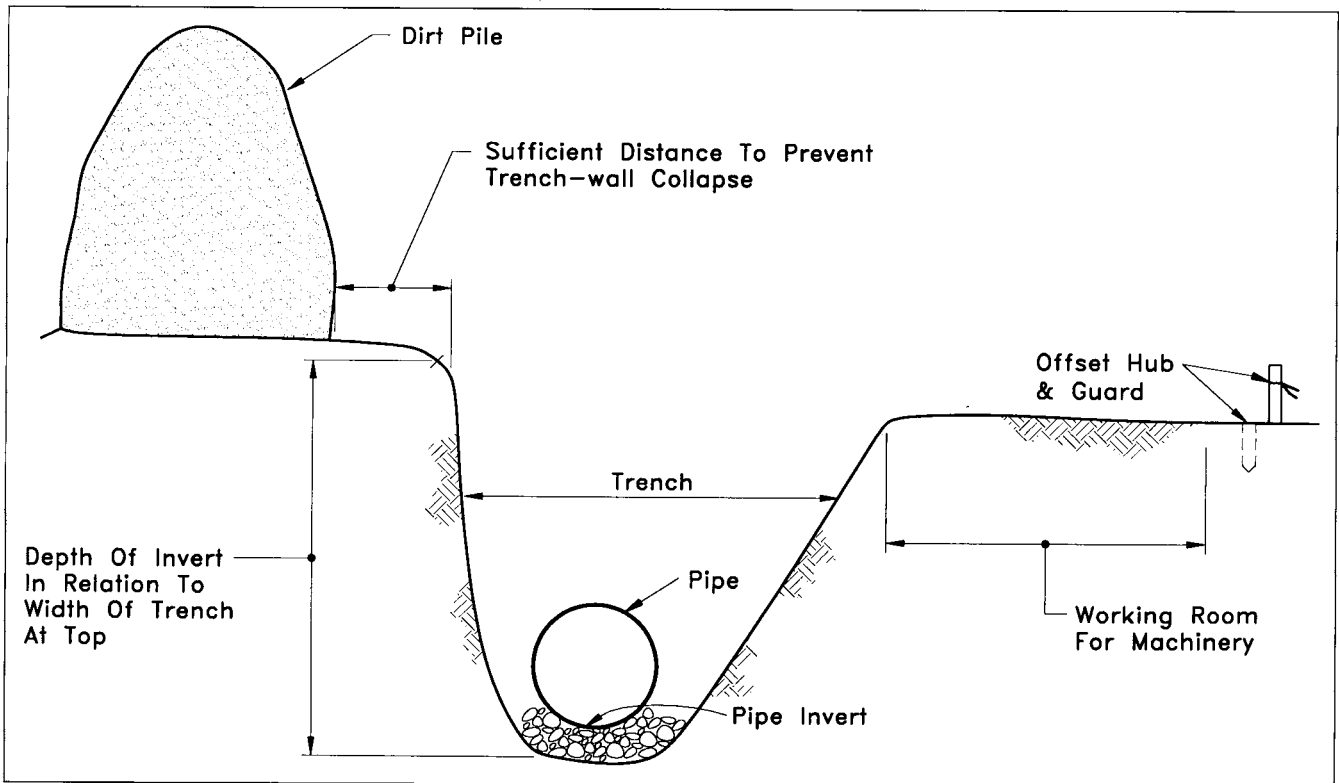
An equality is used at points of intersections or to designate a change in the station numbering system. For example, a point of concern occurs when a waterline crosses over a sanitary-sewer line. An equality is used to correlate the stationing of the sanitary line with the waterline. That is, the equality consists of equating the waterline station to the sanitary-sewer station at the point where they cross each other. Equalities are also used at street intersections. For example, a station equality is shown at the point where the two streets intersect, and this identifies (i.e. equates) the station street to the station of the intersecting street. An example of a station equality of two intersecting streets is 9 + 51.22 CL Main Blvd = 2 + 07.10 CL Crossing Drive (see Figure 35.3).

### Leveling Methods

**Differential Leveling.** After the offset hubs are set, their elevations are established to create a vertical reference for the contractor's use. Most construction-stakeout elevations



**FIGURE 35.1** Hub and tack with guard stake.



**FIGURE 35.2** Open trench, pipe invert and offset line.

are obtained by differential leveling. This procedure consists of running a level circuit between a minimum of two control points and obtaining rod readings on the hubs as the circuit progresses. The level circuit itself is a series of adding and subtracting rod readings to obtain differences in elevation. Once the height of the instrument is established, any subsequent rod shot need only be subtracted from the H.I. to determine the elevation of that point (see Figure 35.4). In construction-stakeout surveys, the observations of a number of offset hubs may be made from one instrument set-up, but these elevations are technically unproven since they are side-shots and not a contiguous part of the run. Therefore, key points of the stakeout are *turned* upon in the course of the level circuit. A *turn* consists of reading the rod on a stable point, changing the height (and location) of the instrument as needed, and reading the rod again. As part of a continuous-level run, the turn is an integral arithmetical component and is proven by the final check-in at another known benchmark (see Figure 35.5). Successful turns should be balanced, roughly using the same distance for backsight and foresight, to eliminate any adjustment error in the instrument. Also, when reading high on the rod, the rod can be rocked gently to and away from the instrument. This allows the instrument operator to read the least graduation on the rod, eliminating the error created by an out-of-plumb rod.

Leveling notes are kept in columnar form, using standard survey field-book paper. Each column has a set function in the notes, and the facing page, which is gridded, is

reserved for sketches and/or additional descriptions. (See Figure 35.6 Field notes.)

When jobsite benchmarks are established, they must be set in practical locations safe from harm. They must also possess an accurate elevation, proven by inclusion as a turning point in a level circuit, which should produce an acceptable closure. Recommended benchmark locations include a nail or spike driven into a save-tree, a corner of an existing storm structure, a flange bolt on a fire hydrant, or a re-bar or pipe driven flush into the ground.

**Three-Wire Levels.** Three-wire levels are used when greater accuracy is necessary. Many leveling instruments are equipped with three horizontal cross hairs. The top and bottom cross hairs are often called stadia hairs (or wires). The middle hair, at the center of the scope, represents the horizon line of the instrument. On most internal focusing instruments, the spacing of the stadia hairs creates a proportion of 1 ft to each one hundredth of a foot intercepted on the rod. In three-wire levels, the rod is read at each intercept: top stadia hair, middle cross hair, and bottom stadia hair. The readings are made to the thousandth of a foot by visually dividing a hundredth on the rod into ten parts. The readings are added together and divided by three to obtain the mean value. The mean should equal the middle cross hair reading, or be within one thousandth of a foot difference, to be valid. Good results may be obtained if only the first section of the rod is used (because of possible movement in the rod section clamps), each shot is balanced, and distance from level to rod does not exceed

TABLE 35.1 Surveying and Mapping Symbols and Abbreviations

AHD	Ahead	HSE	House	RAD	Radius
$\sphericalangle$	Angle		Instrument,	RF	Right Front
$\sphericalangle$ PT	Angle Point		Instrument Operator	RR	Right Rear
	Area in Square Feet	INTX	Intersection	$\emptyset$	Rod/Chain Person
@	At	INV	Invert	SAN	Sanitary Sewer
BK	Back	LF	Left Front	SAN LAT	Sanitary Lateral
BM	Benchmark	LR	Left Rear	S/O	Stakeout
	Benchmark		Notekeeper	STA	Station
$\text{C}$	Centerline	#	Number	STM STR	Storm Structure
COR	Corner	%s	Offset	ST	Street
$\sphericalangle$	Cut	$\otimes$	Offset		Square Point
$\Delta$	Delta	- -	Planetable Setup	TAN	Tangent
EP	Edge of Pavement	PC	Party Chief	TBM	Temporary Benchmark
ES	Edge of Shoulder	PC	Point of Curve	TB	Toe of Bank/Top of Bank
FC	Face of Curb	PI	Point of Intersection	TS	Toe of Slope/Top of Slope
$\pi$	Fill	PRC	Point of Reverse Curve	TC	Top of Curb
	Grade	PT	Point of Tangent	$\triangle$	Traverse
GB	Grade Break	POC	Point on Curve		Traverse Line
$\sphericalangle/2$	Half Delta	POL	Point on Line	TP	Turning Point
HI	Height of Instrument	PCC	Point of Compound Curve	VERT $\sphericalangle$	Vertical Angle
Hor	Horizontal	PCTC	Point of Curve, Top of Curb	W/L	Waterline
		$\underline{R}$	Property Line	WS	Water Service

80 ft. The notes are kept in the same fashion as for differential leveling. (See Figure 35.7 Three-wire level.)

**Trigonometric Leveling.** Trigonometric levels can be accomplished using a total station and prism pole, a total station and data collector, or a traditional level and rod. It applies the properties of a right triangle to determine differences in elevation. As shown in Figure 35.8, the vertical angle between the instrument's horizon line and an intercept of the rod or prism is measured. The slope distance and/or horizontal distance is also measured. With this information, the opposite side can be calculated and the prism height or rod-intercept height applied to obtain the

difference in elevation between the point where the prism/rod is being held and the instrument's horizon line.

### Notekeeping

Note keeping is a vital component of the survey crew's work. A set of field notes record the date, the crew's identity, and the work performed. Depending upon the purpose of the survey, the information and data may be used by designers, engineers, contractors, and other surveyors.

As-built locations, profiles and topographical survey notes provide designers and engineers with three-dimensional descriptions of existing terrain and features.

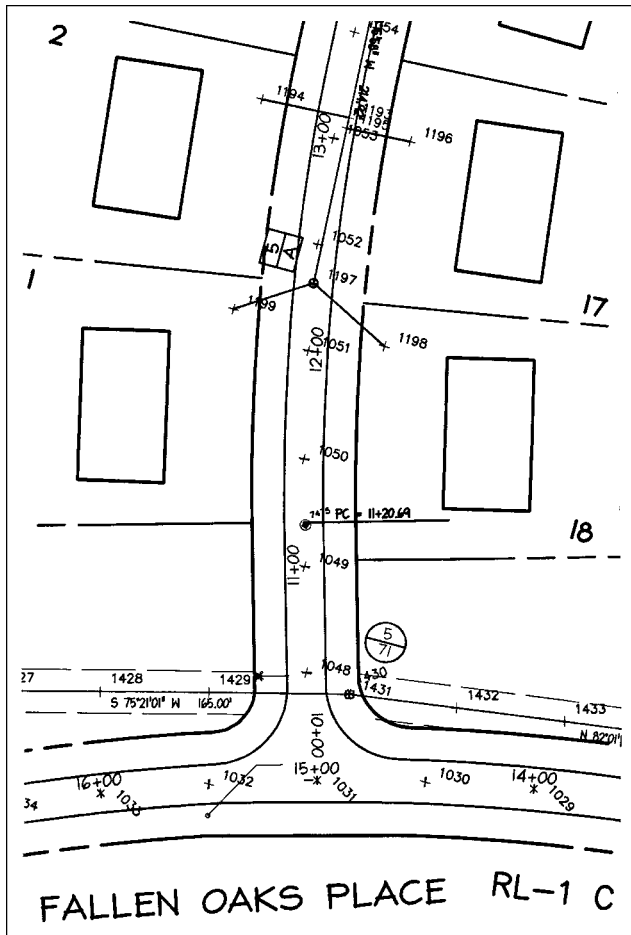


FIGURE 35.3 Example of street stationing.

Stakeout level notes and location survey sketches supply contractors and surveyors with the horizontal and vertical data necessary to create an accurate cutsheet. Sketches of traverse and centerline street references and TBM (temporary benchmark) notes may furnish the starting point for many of the subsequent surveys.

The field note-keeper functions as the “eyes” for the office personnel; their accurate sketches and observations convey vital horizontal, vertical, and other descriptive information. Moreover, these notes should provide the survey office with the means to check the accuracy of fieldwork before structures are built.

Good notes should be clear, complete, and self-explanatory. Proper notation should be used, along with terminology and symbols standard to the profession. Sketches can convey a great deal of information if they are organized and show all pertinent measurements and observations. An important aspect of a construction stakeout is to keep good records of the work performed. A surveyor must always be prepared to defend his work in this litigious world, and proper documentation is the key. As stakeout markers are easily and frequently disturbed or accidentally

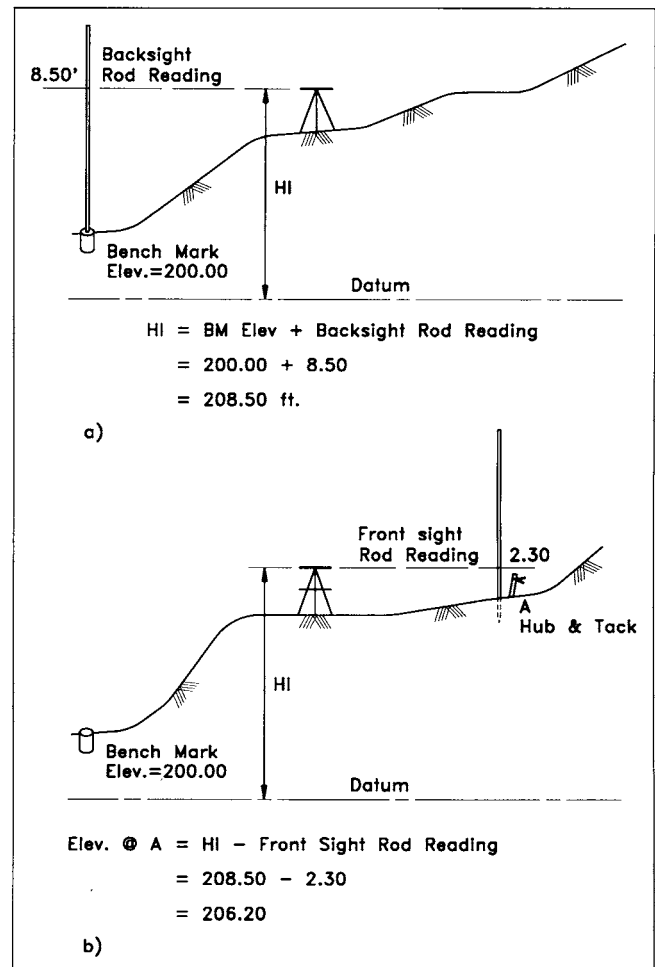


FIGURE 35.4 (a) Establishing height of instrument (b) Determining elevation at a point.

removed, records of notes and drawings of the work performed make it easier to reproduce the lost positions and resolve any discrepancies during and after construction.

The use of total stations, equipped with data collectors, does not eliminate the need for field notes and sketches. A jobsite fieldbook should be maintained for handwritten notes, which accompany the digital data. Field sketches could include outlines of buildings and structures, dimensions, measurements, types of materials, pipe sizes, descriptions, data-collection point numbers, and other pertinent facts. In the office, handwritten notes and sketches can be scanned from the fieldbook and added to the jobsite database along with the raw and adjusted digital-data files.

## BASIC PROCEDURES

### Office Preparations

Office preparation for construction stakeout includes conferring and scheduling with the client, obtaining up-to-the-minute plans, computing necessary stakeout points, drafting the layout, and preparing the jobsite folder. The

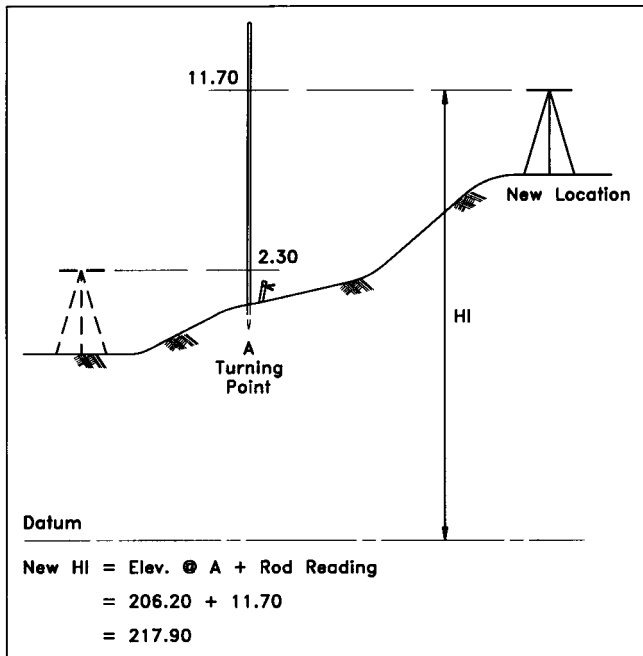


FIGURE 35.5 Establishing HI at a turn.

client has specific needs and deadlines and the field crew's schedules must be organized accordingly, with the parameters of the work understood by all. The need for special surveys, such as blasting stakeouts, burn or borrow pit stakeouts, grid stakeouts, or multiple offsets, will be dictated by the terrain of the site, the local utility requirements and restrictions, the equipment and materials used in construction, and any problems that may be encountered along the way.

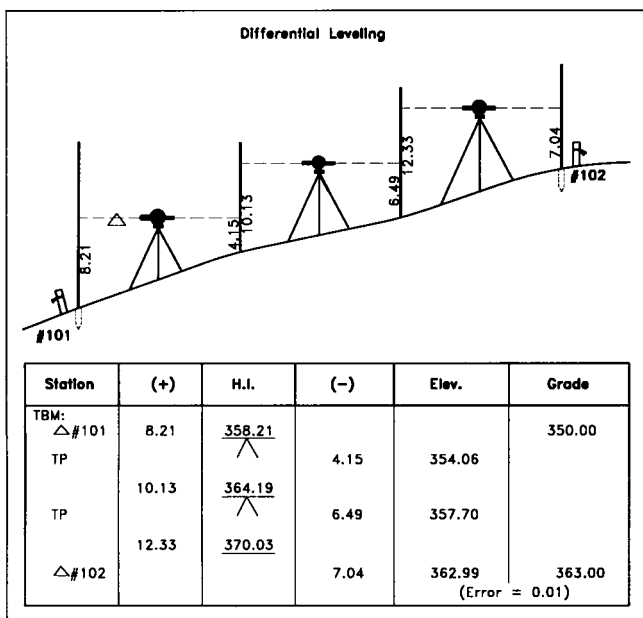


FIGURE 35.6 Field notes.

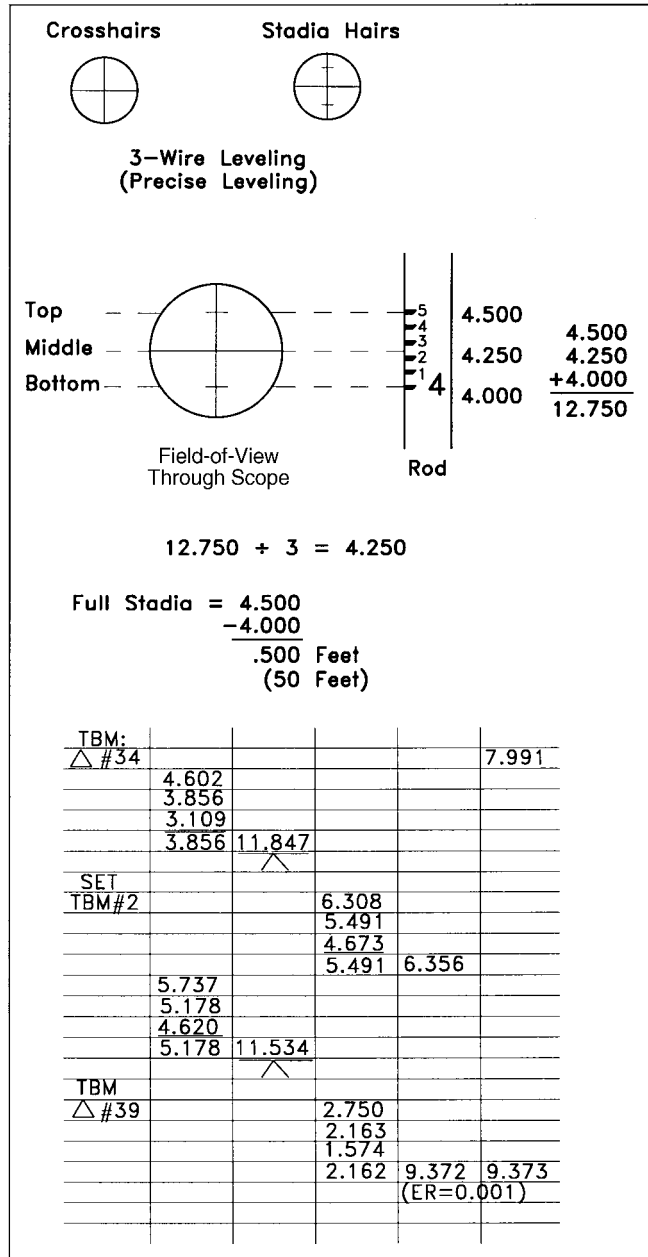
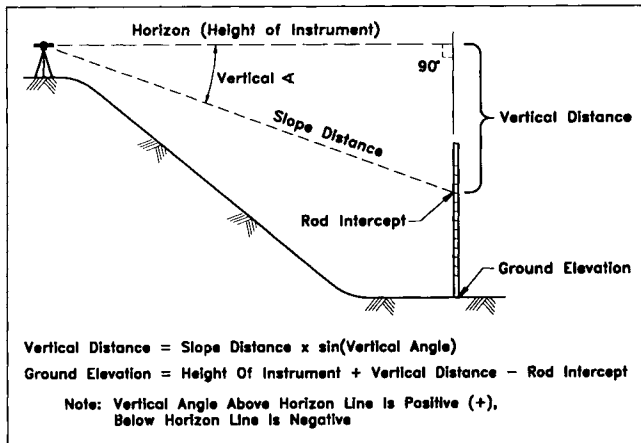


FIGURE 35.7 Three-wire level.

Every construction stakeout involves three important sources of information: the surveying layout plan, the record plat, and the approved construction drawings. The surveying layout plan has the boundary, traverse and other control points plotted on it. When the client calls for a stakeout of a particular item (e.g., sanitary sewer), the pertinent information is transferred from the construction drawings onto the stakeout-layout plan. Bearings and distances from control points to the stakeout points are computed and shown. As construction progresses and more stakeout work is requested, new information is added to the hard copy. The original drawing is kept in the office and is updated with this additional stakeout information. A



**FIGURE 35.8** Trigonometric leveling.

copy of the layout plan is made to accompany the field crew. A list of the coordinates may also be sent along with the original, in case the field crew needs to perform any calculations in the field.

When total station and data collectors are used, the coordinates and elevations can be uploaded into the instrument to facilitate the stakeout. This reduces the need for a detailed layout plan and minimizes the field calculations.

The plat of record contains bearings, distances, restrictions, notes, easements, curve information, and approvals of the subdivision or individual tract of land. The construction drawings impart details necessary to the proposed construction. A typical set includes an index, a legend of symbols, typical street cross-sections, site plans and typical layouts, sanitary plans and profiles, storm sewer computations and profiles, waterline profiles, and street plans and profiles, all of which are vital to the survey stakeout team. The surveyors have the opportunity to recognize any problems, or potential problems, between design concepts and actual construction methods as the layout plan and stakeout ties are produced in the office. When necessary, a solution can then be determined in conference with the site-design engineers, local government officials, and the client.

Two jobsite folders are prepared: one for office referral and one for the field. The office file contains house templates and plotted overlays, originals of special surveys, locations of structures, utilities, test holes, burn pits, borrow pits and monarch trees, encroachment checks, and easement plats. The field folder might be divided into sections like TBMs and REFs (Temporary Benchmarks and References), COORDINATES, DEFLECTION SHEETS, HOUSE STAKEOUTS, CUTSHEETS, and MISCELLANEOUS. Along with this folder and up-to-date copies of the layout plan, record plat, and construction drawings, the construction party chief is equipped with the viable horizontal and vertical data needed.

### Field Procedures

On site, the survey party will perform a reconnaissance of the area, looking for the traverse stations, property corners,

and any other known horizontal-control points. A minimum of two positions must be intervisible to begin the survey, so that the instrument can occupy one and backsight the other. Other control points must be recovered to check the work. As the instrument is set up at new positions, it is recommended that an angle and distance be measured to a known control point to verify field computations.

The backsight serves to orient the surveying instrument. A pre-determined azimuth is established between the instrument's station and the backsight station, and this enables the instrument operator to execute new angular directions. A late-model geodetic surveying instrument is capable of turning an angle to the nearest tenth of a second, but for most construction layout purposes, a 20-second instrument is adequate. (Note: trigonometrically, 20 seconds of angular error projected to a distance of 100 ft yields only 0.01' of error, a displacement well within the acceptable limits for almost all types of construction. There are methods that, when applied, can further refine the accuracy of a 20-second instrument.) Advancements in electronic technology will undoubtedly make total stations and data collectors more affordable and more precise, making conventional surveying instruments virtually obsolete.

The method of distance measurement used by the surveyor is dependent upon the desired level of accuracy. The variety of construction stakeout duties requires skills in pacing, chaining or taping, and in working with total stations and data collection systems to measure distances, depending upon the type of survey and the degree of accuracy required.

After a reconnaissance is complete, the party chief must form a logical sequence for the survey, based upon the points recovered, stakeout desired, on-site obstacles, and the distances involved. While a timely survey is desirable, accuracy is a necessity. The party chief should have a firm grasp of trigonometry and coordinate geometry, because of the number of computations that are often solved in the field, such as circular-curve deflections, radial bearings and chords, inverses using coordinates or bearings and distances, and intersections using bearings and/or distances. This knowledge is necessary to trouble-shoot possible problems in the plans, create alternative ways to perform a survey, and to provide mathematical checks and corrections to work in progress.

First and foremost is the need to check the work completely. There are routine checks in mathematical computations and there are routine checks in "laying out" and "staking". A minimum of three control stations are needed to layout and check a survey. Ideally, the instrument occupies one control station while another control station is sighted. Then the surveyors measure a computed angle and distance to the beginning point of the stakeout. The instrument is moved ahead to the new point, the last station occupied is backsighted and a new angle and distance is measured, this time to a third control station. If this checks, then the accuracy of the beginning point is established. For



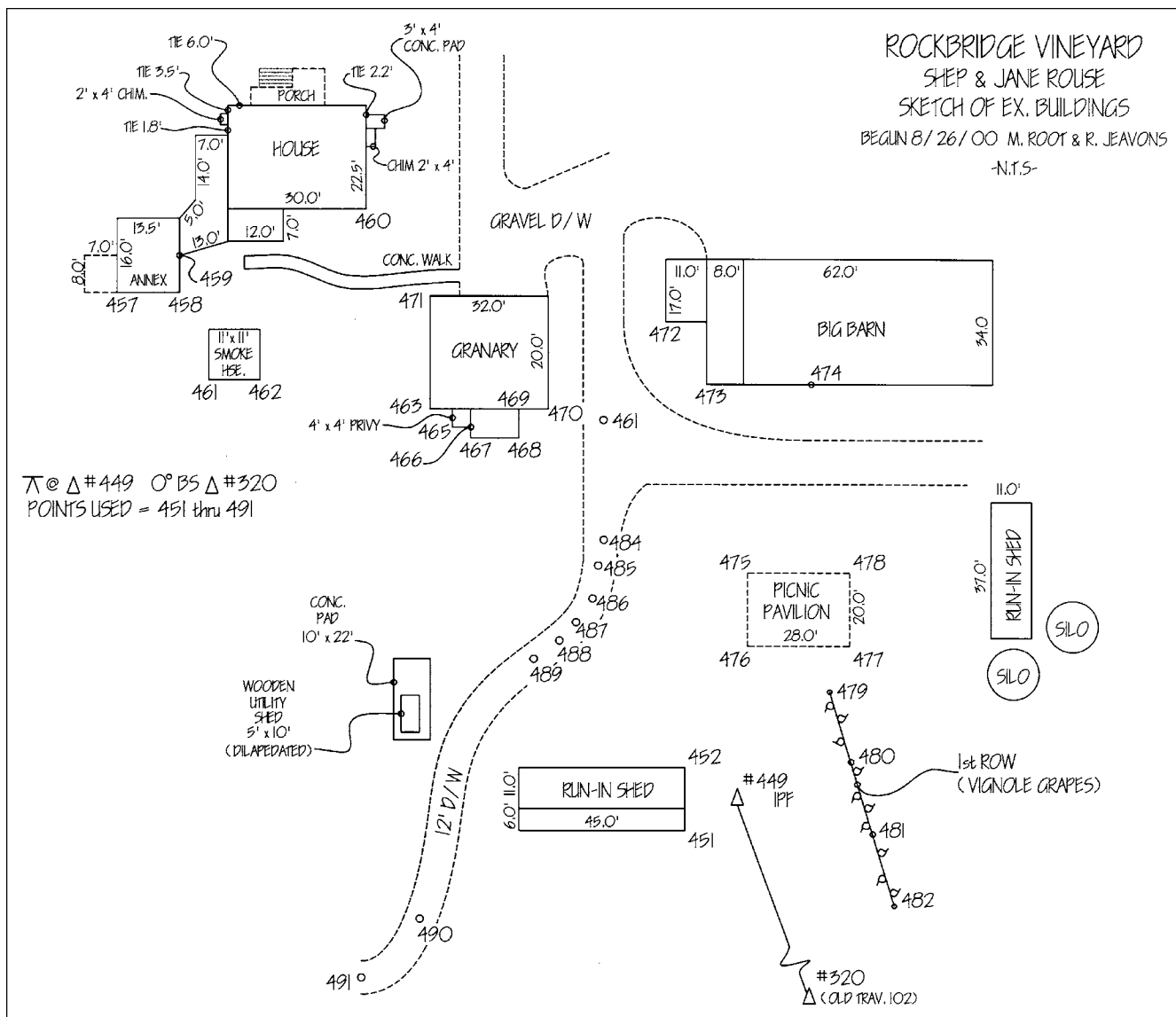
a lengthy stakeout, the instrument is moved forward to each subsequently-set point, backsighting the last, so that the whole series is proven to a known control station at the final check-in (see Figure 35.9).

**Using Total Stations and Data Collectors for Stakeout**

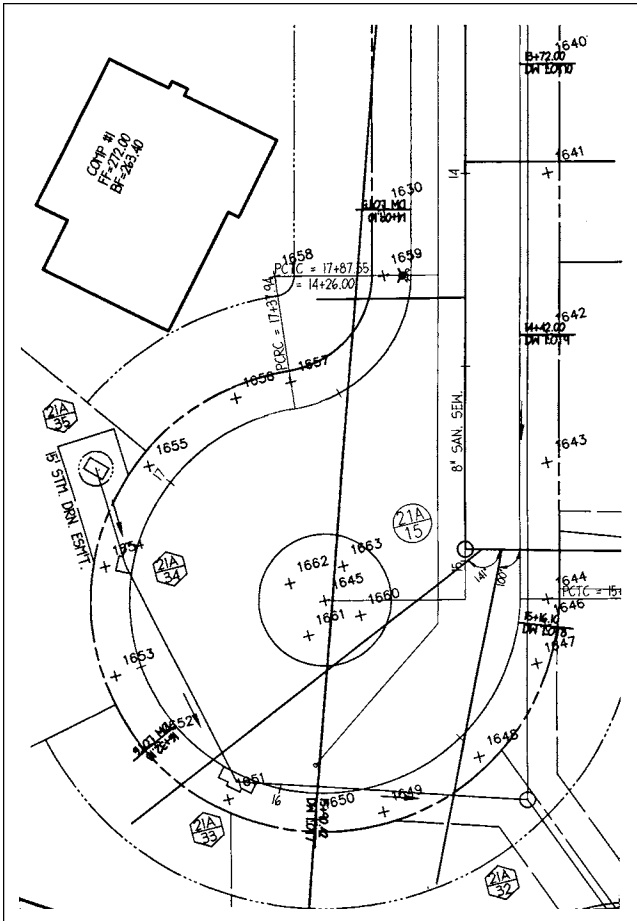
The data collector is capable of storing a jobsite's entire coordinate-geometry file. These "electronic field books" store and collect data and interface directly with a "total station" instrument and a computer. With this information, the data collector computes coordinates from distances and angles and vice versa, performs inverses, intersections and resections, determines stationing and offset positions, elevations and cuts and fills. The computed information can be stored for field or office reference and record-keeping purposes.

In the office, up-to-date plans are assembled, coordinates are generated, and point number plots are created. The data is loaded into the data collector and readied for the field. In the field, the station occupied, height of instrument, elevation of station, and backsight and foresight information are entered into the data collector. Once this is accomplished, the actual backsight and foresight are taken, using the total station and a prism target. The data collector can determine the horizontal and vertical position of the occupied station for comparison and check. Once the occupied position is verified, the "radial stakeout" can commence.

A "radial stakeout" is a method where many or all offset points are established from one instrument set-up. This method is best when used with a total station and data collector. However, a radial stakeout can be risky if not



**FIGURE 35.9** Field sketch to accompany data-collection.



**FIGURE 35.10** Surveying layout plan.

followed with a series of checks. Before the stakeout begins, the party chief should compare information from the plans and profiles with information computed by the data collector. Using the feature that computes inverses between points, the party chief checks street widths, offset lengths, distances between structures, and building dimensions. As the stakeout progresses, the backsight should be checked periodically, and before moving the instrument to a new location, to insure the accuracy of the instrument's orientation. Good communications between the instrument operator and the prism handler should be maintained to insure accurate entry of prism height for each instrument reading. Wherever possible, angles and distances to other known control points should be made from the key points of the stakeout. Distances between offsets and centerline of stakeout can be checked quickly by taping. Finally, a location of key stakeout points should be made from a known control point that was not used as part of the survey. This provides verification and a record of the points staked.

Back in the office, the data is downloaded into a computer. It is recommended that no more than one day's worth of work be allowed to accumulate in the data collector. The party chief is consulted for the initial edit of the data file,

where errors in coding, prism height, backsight or foresight entry can be corrected. After the data is downloaded into a computer, plottings can be made of the work.

### Errors in Stakeout Surveys

There will always be a certain amount of error in a stakeout because of weather, equipment, terrain and human fallibility. A survey party chief knows the tolerances of acceptable error. Stakes that are set for controlling earthwork activities need only to be within 1 ft horizontally of the plan location, whereas stakes set for house construction must be accurate to the nearest hundredth of a foot. Each type of stakeout must be analyzed in light of: what is its purpose, how will it be used, and can these points be used for another survey? With this knowledge, the amount of tolerable error can be determined, and if it is exceeded, the source of error must be found and corrected.

After the stakeout is complete, the survey crew should "step back" and look at the "big picture" of the stakeout. Too often the same attention to details and intricacies and the level of effort used to set individual points is not given to the overall task. Taking time to view the stakeout from a farther or different vantage point or comparing it to the construction drawings may disclose an obvious error or blunder. The cost of this short amount of time is inconsequential when compared to the cost of re-staking, reconstructing or replacing dirt, concrete, and asphalt.

### Referencing Points in Construction-Stakeout Surveys

References enable the original survey to be reconstructed with minimal effort. Those staked points that are either intended for later use or that are anticipated to need replacement because of disturbance should be referenced to other permanent points. In Figure 35.11, a street centerline was staked, and because of the survey accuracy, it became a desirable baseline for all future work. Since the street was under construction and was the only road for construction traffic, the entire initial survey would have been lost without the references. Note that each reference line is set perpendicular to the tangents of the centerline, and that the distances from centerline are a minimum of 50 ft, well out of harm's way. To reconstruct each centerline point, the instrument is set over the reference point nearest to the centerline, the farthest reference point is sighted, and then the line is projected to the street at the distance noted. Later, when the street has curb and gutter, the references can be chiseled into the concrete on both sides of the street serving as replacements for the original temporary references that could be destroyed during house construction. Every time a point is referenced, a sketch is drawn and placed in the jobsite folder, providing the next survey crew with the descriptive information long after the directives on the original stakes have become illegible. Figure 35.12 illustrates other ways to reference points in the field.

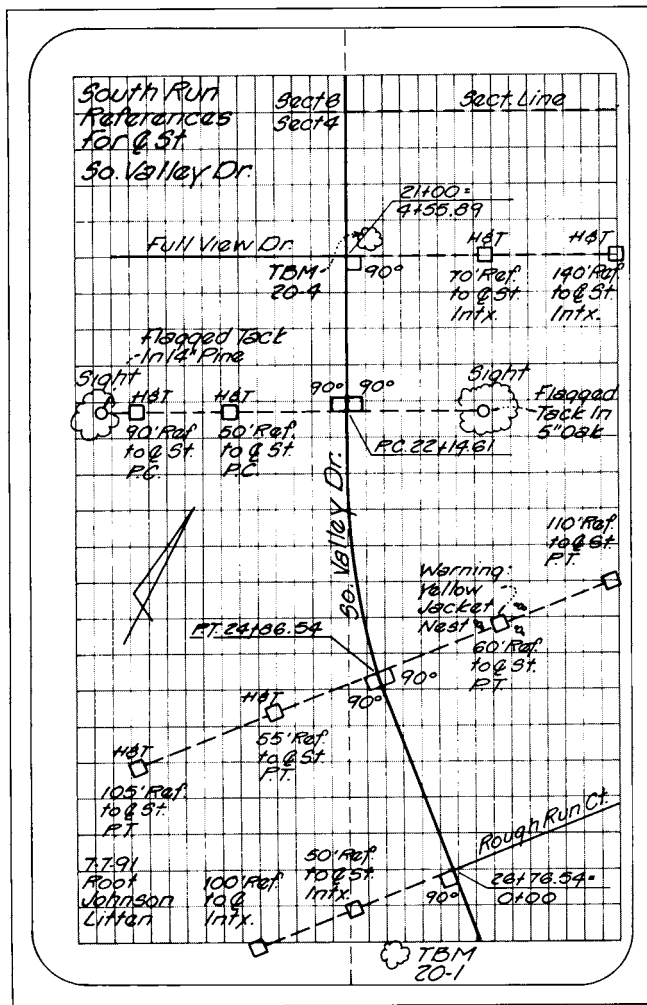


FIGURE 35.11 Example of point referencing.

**Cutsheets**

The cutsheet is a method for organizing and presenting stakeout calculations. For surveyors, it is a record of what was done in the field. However, contractors use it to establish line and grade for buildings and trenches, while municipalities may require it as part of the permit/inspections process. A typical cutsheet form is shown in Figure 35.13. Specific cutsheets can be tailored for a particular type of stakeout.

After the stakeout and elevations for the hubs have been established, the field notes and construction drawings are used to fill out the cutsheet. The column labeled STAKE EL. represents the offset hub elevation as determined in the field, while GRADE EL. is the proposed elevation of the invert at the pipe centerline or at the exact point of construction (e.g., top of curb, corner of bldg.). This elevation is taken from the plan or profile of the construction drawings. The difference between these two values is the amount of cut or fill.

Proposed grades for gravity flows are computed using the invert elevations and the slope of the pipe. Elevations to inverts for pipes carrying pressure flows are scaled from

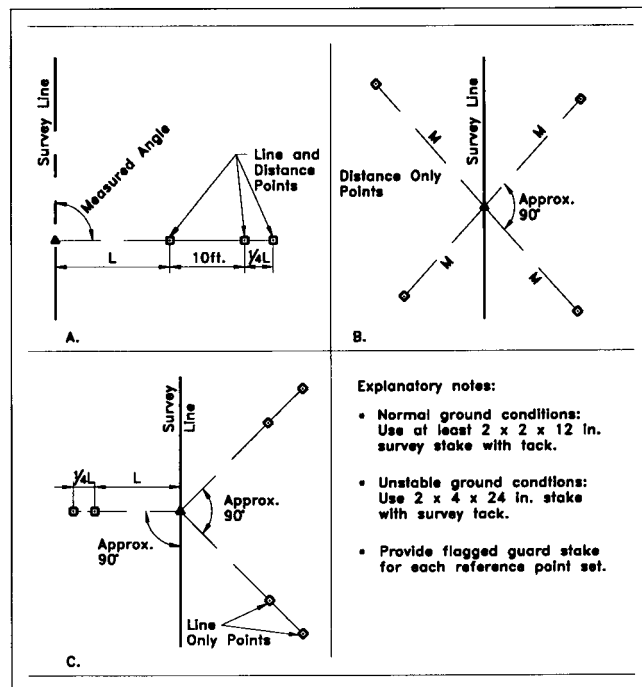


FIGURE 35.12 Reference-staking methods. Courtesy of Brinker, R., and R. Minnick, *The Surveying Handbook*, New York: Chapman & Hall.

the profile or computed based on the finished grade elevation and the minimum cover to the pipe. In street and parking areas, the top of curb elevation or edge of pavement typically is used in the cutsheet calculations. Top of curb elevations in streets are easily determined from the longitudinal grades and the typical section of the street. However, computing top of curb elevations in parking areas is not as straightforward. Because parking pavements have variable and non-uniform longitudinal slopes and cross slopes, the proposed elevations are computed from given spot elevations or by using judgment. When the plan is too vague to determine a workable elevation, the surveyor should immediately consult with the design engineer.

The cutsheet is one of the last points in the design process where errors and conflicts can be caught before they become costly to rectify. The individual computing the cutsheet should watch for conflicts where pipe and other utility crossings occur, check to see that minimum depths are adhered to and be attentive to other possible engineering, field and construction conflicts. The contractor needs the cutsheet information to begin construction. Typically, it should take less than a day or two from the completion of the field stakeout to prepare the cutsheet and return it to the contractor.

Descriptions and examples of cutsheets for particular stakeouts can be found throughout this chapter.

**CLEARING STAKEOUT**

On a wooded site, the first stakeout usually requested establishes the clearing limits. Trees enhance a property's



In the past, a clearing limit was accomplished using a plane table and alidade. The main disadvantage of this method was that alidades have no horizontal circles; therefore, there would be no record of the instrument angles produced, should someone question the work later. At best, the distances used could be written next to each main point, but this might not have provided enough evidence to prove field accuracy.

Presently, most clearing stakeouts are performed using a total station and prism pole, either with or without a data collector. On a small site, the clearing limit can be scaled and protracted directly on the site plan (see Figure 35.14). A straight-edge is used to draw spokes from the instrument-occupied station to each main point of the clearing limit. Next, a protractor is aligned on the plan, with the zero degree mark placed on the backsight station, and the angle to each spoke is noted and recorded to the nearest half degree. Then each distance is scaled to the nearest foot and the results written next to the angle of that line. This method is desirable as it leaves a written record of the performed stakeout.

On a large site, a data collector is generally used. Coordinates are computed for points along the clearing line and the data is uploaded into the data collector. In the field, the data collector provides the angles and distances from the occupied station to any point desired. The points that are actually set can be located and stored in the data collector and plotted in the office later, serving as a check of the work.

In the field, control points are recovered and referenced if they appear threatened by the clearing operation. As each point of the clearing limit is established, the rodderson ties a streamer of flagging before moving onto the next logical point. The person at the instrument must make certain that the points are spaced at intervals so as to be intervisible through the woods and to guard against the loss of accuracy caused by spacing the tie points too far apart in a curve (following a chord instead of an arc). After the key points are set, they are connected by running flagging between them. It is tied high and secured well so it will remain undisturbed by the weather and out of reach from any children who might wander onto the site. In areas where van-

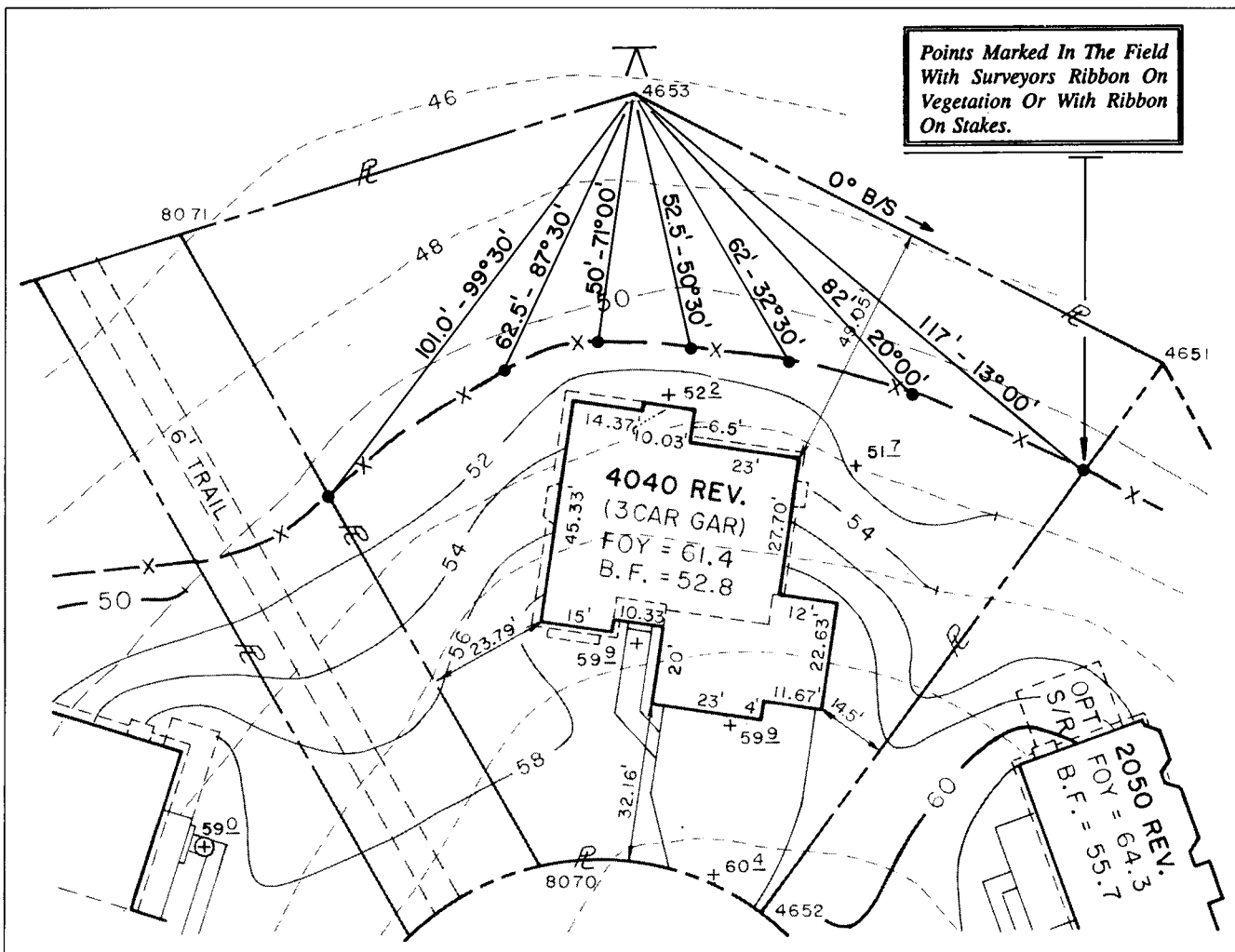


FIGURE 35.14 Protracted and scaled-clearing stakeout.

dalism may be a problem, the contractor may request that the trees be marked with paint as well. Flagged trees along a limit line are left standing, a fact that must be considered before tying or painting a dead or damaged tree.

The party chief makes sure there is sufficient width flagged on the utility easements to accommodate the depth of the trench and working room for the machines. In addition, the party chief may decide to flag a slightly larger area if the clearing limits seem tight in relationship to the proposed buildings. Finally, the party chief should locate any specimen or monarch tree that may be a borderline case for removal. In an effort to save such a tree, the field location notes are plotted by the survey office personnel and sent to the planners and engineers to make a determination.

If rights-of-way or easements alone are requested for a clearing stakeout, another procedure can be used. The centerline of the street, right-of-way, or easement is staked every 50 or 100 ft. Again, it is good practice to measure an angle and distance to a known control point to verify the stakeout. Once this is accomplished, distances to the clearing limits can be measured from right and left of the centerline points.

## GRADING

### Street Grading

The centerline of streets is generally used to control the horizontal and vertical limits of their construction. For subdivision streets, the stakeout consists of setting hubs at the centerline's main points, such as PCs and PTs, and at 50-ft intervals. As a check, ties (measuring angles and distances) are made to control points from selected PCs and PTs. The ties are made when control points can be sighted and measured without too much difficulty from the selected points on the centerline. However, it is a rare situation when ties can be made frequently as the stakeout progresses. Typically, early control points are near the project's boundaries. Therefore, ties are made as the street stakeout nears the boundaries.

The outside limits of the grading for the street can then be established by measuring, radially or perpendicular from the centerline hubs, the necessary distance. In most cases, the horizontal accuracy required for setting the outside limits for the rough grading is approximately 0.5 ft. The stakeout for the street is set on the actual centerline rather than offset. Although this might obstruct the equipment operators as they grade around these points, the close proximity of the points allows the contractor to set the street grades accurately, even in the rough-grading phase. Evidence of the saved points within the grading area are apparent on construction sites, where one can see numerous nearly cylindrical mounds with guard stakes and surveyor's flagging on top. When the street is brought to grade, these mounds are easily removed.

Reference points to selected centerline hubs may be set if the stakeout proves accurate. Being able to re-establish the centerline from reference points rather than from control points several thousand feet away saves time and money.

Elevations are determined for all the hubs, starting at the point where the street construction is to begin. On most projects, street construction begins where the intersection of the proposed street meets an existing street near the boundary of the site. From here the construction proceeds towards the interior of the project. In addition to providing a beginning point where the elevation of the proposed street is assured of matching the elevation of the existing street, this practice provides a logical order for surveyors to check the work against the plans and supplies the contractor with a cutsheet starting at the most accessible beginning point. If the cutsheet provides cut and fill depths to the finish grade of the street, the contractor is responsible for deriving the subbase elevations.

### Slope Staking

Engineering projects such as road and rail beds, ditches, and canals often use slope stakes to define the limits of excavation and grading. The slope stake is referenced to the "catch point"; the intercept is where the finished side slope meets the original ground surface. Slope stakes are especially helpful to the contractor in areas of cut where the catch point is less apparent than in areas of fill.

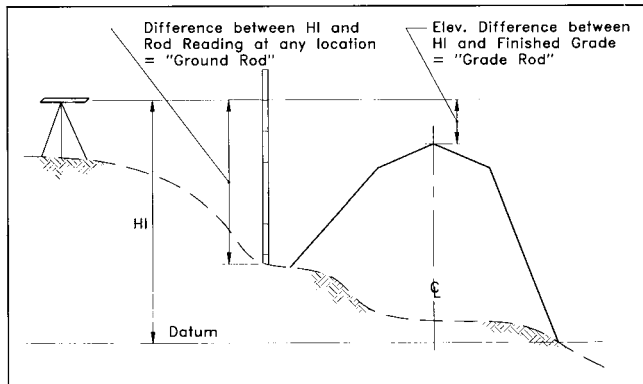
To begin the survey, the centerline of the road or ditch is staked at regular intervals and elevations are determined at each station. The next portion of the survey, setting the slope stakes for each station, uses a level, leveling rod and a measuring tape. The objective is to find the catch point left and right of each station, and this is achieved by using Eq. (35.1) and a series of trials:

$$d = \frac{w}{2} + hs \quad (35.1)$$

where,  $d$  is the measured horizontal distance from centerline to the catch point,  $w$  is the plan width of the roadbed or ditch,  $h$  is the cut or fill at the centerline, and  $s$  is the side-slope ratio.

These parameters are shown in Figure 35.15. As shown, the inherent horizontal error in Eq. (35.1) is because of the variation in depth,  $h$ , at the centerline and the actual depth at the catch points  $h_L$  and  $h_R$ . For this reason, setting of slope stakes in the field becomes a trial-and-error process.

To begin the process, the level is positioned and an H.I. established by taking a rod reading on the centerline station previously established. From the plans, the final road-grade elevations along centerline and the typical cross-section side-slopes are extracted for the calculations. The difference between the H.I. and the finished-grade elevation is known as the "grade rod". The difference between the rod reading at a trial location and the H.I. is known as the "ground



**FIGURE 35.15** Slope staking.

rod". The ground rod is variable and depends on the trial location, whereas the grade rod is constant for a given centerline station. The cut or fill at any trial point in the section is figured by subtracting the ground rod from the grade rod. The equation can then be expressed as

$$d = \frac{w}{2} + s(\text{grade rod} - \text{ground rod}) \quad (35.2)$$

The objective in the field is to find a ground rod such that the right side of Eq. (35.2) is equal to the distance from the centerline at the location of the rod reading.

**EXAMPLE** in this example, the road width is 24 ft, the side-slope ratio is 2:1, the finished-grade elevation at the centerline is 29.0 ft, and the H.I. is at elevation 42.0 ft.

1. The "grade rod" is  $42.0 \text{ ft} - 29.0 \text{ ft} = 13.0 \text{ ft}$ . Using Eq. (35.1), the first trial distance from the centerline is 38 ft.

$$d = \frac{24}{2} + 2(13) = 38 \text{ ft} \quad (35.3)$$

At a trial distance of 38 ft, the rod reading is 4.2.

$$38 \equiv 12 + 2(13.0 - 4.2) \quad (35.4)$$

2. Use this information and Eq. (35.2) to see if the right side of the equation balances the left.

Obviously  $38 \text{ ft} \neq 29.6 \text{ ft}$ . The left side of the equation must decrease and the right side must increase, so the rodperson is directed downslope or closer to the centerline.

3. The second trial, at 29.6 ft from centerline, yields a rod reading of 4.3, which when placed in the equation shows:  $29.6 = 12 + 2(13.0 - 4.3)$ , producing a value of 29.4 ft for the right side of the equation ( $29.6 \neq 29.4$ ).

4. Moving the rod to a location of 29.5 ft from centerline produces a ground rod of 4.3 ft. This final trial yields  $29.5 = 12 + 2(13.0 - 4.3)$ ; thus,

$29.5 \approx 29.4$ . This is within 0.1 ft of where the rod is being held, so the catch point is recorded at 29.5 ft from the centerline and the depth of cut is recorded as 8.7 ft ( $13.0 - 4.3$ ). The catch point is then offset, away from centerline, and the stake is marked to express the information to the contractor. The front of the stake indicates the offset distance to the catch point, the cut or fill from the catch point to the finished centerline grade, and the station. The back of the stake indicates the offset distance to the centerline and the cut or fill from the offset elevation to the finished centerline grade (see Figure 35.16).

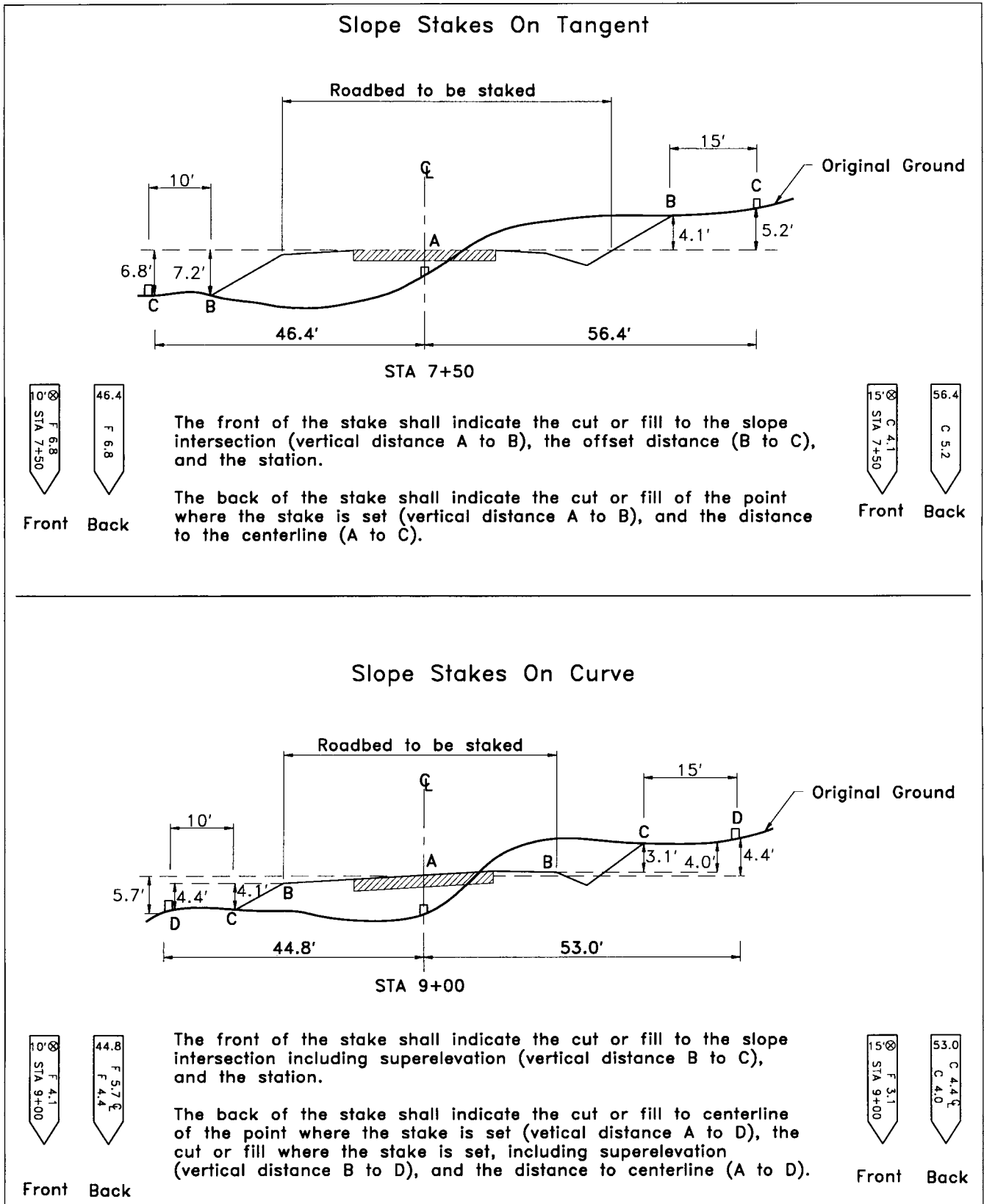
Construction plans for many subdivision streets contain a contour-grading plan or street cross sections. If the topographic maps used to create these are reasonably accurate, the catch points can be scaled directly from these drawings.

### Overlot-Grading Stakeout

On many well-designed projects, the earthwork will balance; that is, the volume of cut will equal the volume of fill (see the Grading and Earthwork chapter). However, in many projects, the cut areas are a considerable distance from the fill areas. Consequently, the dirt must be hauled from one place on the site to the other. Overlot grading is one phase of grading where the site is mass-graded to bring the entire sight closer to the finished grade. In residential developments, this usually includes transporting dirt from one lot to another lot to minimize grading operations during house construction. Typically, such grading operations are associated with commercial and high-density residential developments.

For projects that require a large amount of earthwork, an overlot-grading stakeout may be requested. Overlot grading can be accomplished with a grid stakeout or a contour stakeout. The need will dictate the means.

**Grid Stakeout.** Grid stakeouts are performed when the earth base must be relatively accurate. To insure a reasonable degree of accuracy, the grid points are usually staked at a close interval. Depending upon the size of the project, 25 to 100-ft grid intervals can be used, and cuts and/or fills are determined for each grid point. When establishing the grid in the field, a matrix numbering system helps identify each grid point. One method is to label each grid intersection with a number and a letter. The number represents the line in one direction, and the letter represents a line perpendicular to it. This identification is written on a stake and placed at the intersection of the two lines. The grid is superimposed atop the grading or site plan. The amount of cut or fill is determined from the difference between the existing and proposed grade at each grid point. The grid-point elevations are interpolated between contours, spot elevations or building corner grades. A copy of the grid layout is furnished to the contractor along with the cutsheet.



**FIGURE 35.16** Setting and marking slope stakes. Courtesy of Virginia Department of Transportation, Location & Design Division, *Survey Manual 1991*.



**Contour Stakeout.** Contour stakeouts typically are used for ponds, berms, and slopes. A contour stakeout transfers selected contour lines from the plan to the field to guide the earth-moving operations. The key points to be staked are determined from the plan using such methods as protractor and scale or grid coordinates. When selecting these points, their spacing and location must take into account the maneuverability of the grading equipment. The stakeout markers should be highly visible to the equipment operators, sometimes necessitating the use of 4-ft-tall stakes guarding the hubs. Tall stakes also provide ample room for cut, fill, and grade markings, the proposed contour, and the elevation of the hub.

The stakeout can be achieved with transit and tape, or a total station/data collection system. After the hubs are set, elevations are obtained and a formal cutsheet is issued supplying the cut and fill information.

### SANITARY SEWER STAKEOUT

The design, construction and operation of the sanitary sewer is based upon gravity-flow, hence the importance of pipe invert elevations. Proper sewerage can use a force main system, but it is costly and prone to mechanical failure. Most engineers use the time-honored “water runs downhill” approach. A manhole is constructed at each horizontal change of direction or vertical grade break in the pipe. Each manhole, existing and proposed, has a minimum of three important elevations: the top, the invert elevation(s) of inflow pipe(s), and the invert elevation of the outflow pipe.

Some types of manholes are built with the centerline of the cover offset from the centerline of the main barrel, and this may be 1 to 2 feet. The point used to locate the manhole depends on the purpose and intent of the survey. For as-built surveys and construction purposes, the center of the barrel structure is crucial to pipe slope, pipe length, and easement calculations. For topographic surveys, the center of the manhole cover is often sufficient.

The construction drawings and the record plat contain the information pertinent to the stakeout preparation. Bearings, distances and widths of easements are shown on the record plat and plotted onto the stakeout hard copy. Ties are computed from known control to the centerline of manhole structures. After the centerline of the manholes are set and checked, the offsets are then staked.

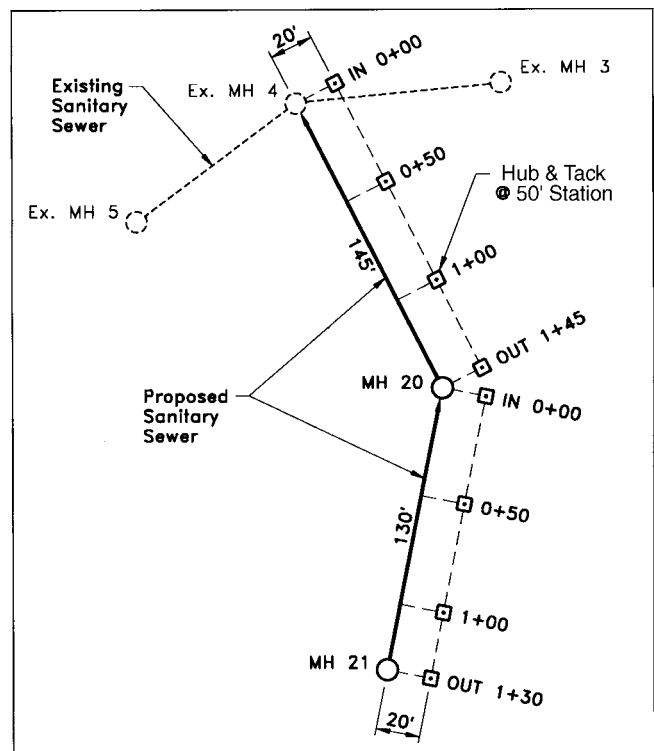
Occupying the centerline of one structure, the next structure centerline is sighted. A 90° angle is turned for the offset. The offset distance from centerline is based upon the sewer depth shown in the profiles. Accommodations must be made to allow for the distance required by the interrelation of trench depth and width, and the presence of the dirt pile. Twenty feet is often sufficient.

The stakeout proceeds in the direction opposite that of the proposed flow (i.e., upstream). The backhoe begins at the lowest point of the stakeout, usually an existing manhole, and digs progressively up the line, “out” of the lower manhole, “in” to the higher manhole. The lowest manhole

invert of the line to be staked is the beginning, its offset hub is called the IN, and its station designation is typically 0+00. If the length of the sanitary line to the next manhole is 100 ft, then the offset station for that manhole is 1+00 and is called the OUT. Another hub is set for this same manhole and designated as the IN (see Figure 35.16). This hub is 90° from the bearing of the pipe line entering the manhole. At this hub the stationing reverts back to 0+00 again before proceeding to the next manhole. In this manner, the actual distances between manholes are self-evident and can be readily compared to the construction plan, the stakes and the cutsheet pluses.

Once the manhole offsets are staked, the rest of the offset line can be set. The transit occupies a manhole offset hub, backsights the manhole centerline and turns 90° to foresight the next manhole’s offset. This check assures that the offset distance is the same as the length of the line between manholes. Intermediate stations are staked, the stakeout length is checked for compatibility to the profile, and the distance is scaled on the plan view as a check. The offset line stations between the OUT and the IN are generally set every 50 ft, but if the terrain is steep, 25-ft stations may be required to insure that the pipe crew has enough horizontal and vertical checks as the line progresses.

After the offset hubs are set, elevations are run on the stakeout. The existing manhole where the stakeout began is accurately located with the top rim used as a turning point in the level run. The existing pipe inverts are measured down from the top elevation. If the proposed sani-



**FIGURE 35.17** Typical sanitary-sewer stakeout.

tary-sewer line is tying into an existing sanitary line between two manholes, both existing manholes must be located vertically and horizontally. The distance between them is measured to determine the slope of the existing pipe.

When the level circuit is run, the IN and OUT hubs are used as turning points to assure accuracy. Two benchmarks are used to prove relative accuracy. If the stakeout is in an easement, the existing ground should be profiled at the centerline to provide a topographical check.

### Sanitary Sewer Lateral Stakeout

The sanitary-sewer lateral is the relatively short length of sewer line that runs from the collector line to the property line. In most municipalities this is considered as part of the public sewer system; thus, the municipality is responsible for its maintenance and repair. The remaining part of the sanitary line that connects the lateral to the house is considered privately owned, and maintenance is the responsibility of the plumbing contractor doing the pipe work for the building.

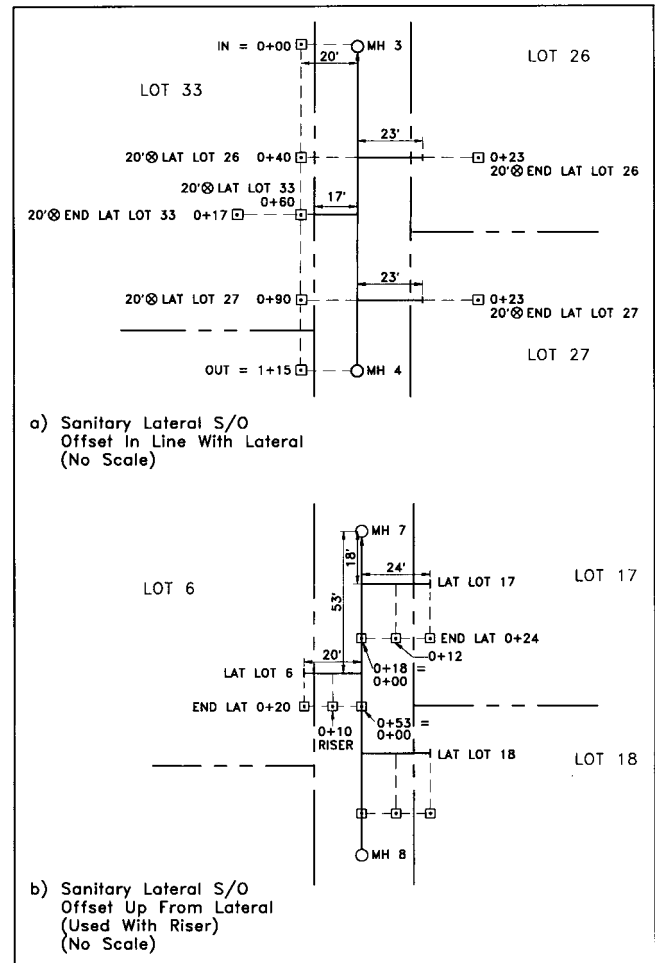
In most cases, the existing sanitary sewer and/or the sanitary stakeout is used as a control for staking out the laterals. Before setting any stakeout hubs, the party chief compares the location of the lateral, shown on the profile sheet, with the location shown on the plan. Discrepancies are resolved and duly noted, or in extreme situations, the engineer is consulted before proceeding with the stakeout. The lateral's length on the plan view is checked by scale, and the presence of risers (vertical pipes) in the lateral section is noted, as these must also be marked with offset hubs. Finally, each lot is checked to insure that there is a lateral provided.

For each lateral, a hub is placed at the appropriate station along the main section of the sanitary-sewer system. The transit is set over one of the hubs, a  $90^\circ$  angle is turned from the direction of the of the sanitary main in the direction of the lateral. Along this direction, a distance is measured equivalent to the length of the lateral plus the offset distance. The length to the riser, plus the offset distance, is also measured and staked. This method places the stakes on the lateral line extended. Another way to offset the lateral is up the main sanitary line, at the offset distance. Both of these methods are illustrated in Figure 35.18.

Elevations are run on the offset hubs, with the end of the lateral hub either used as a turning point or the rod inverted upon it as a check of the reading. This second method serves as an immediate check, verifying the reading of the footmark on the rod. Both readings should equal the total height of the rod when added together.

### Cutsheets for Sanitary Sewers and Sanitary Laterals

After sanitary-sewer laterals are staked and the readings verified, cutsheets are prepared from the stakeout information. Before the cutsheet is started, the person preparing the cutsheet checks the field notes and certifies that they are cor-



**FIGURE 35.18** Sanitary lateral stakeout (a) Offset in line with lateral (b) Offset up the line from lateral.

rect with his initials. If there is an error in the field notes, the party chief is consulted, and a site visit may be necessary to resolve any discrepancies.

Next, copies of the current profiles (see Figure 35.19), record plat, and surveying hard copy are gathered for reference. The cutsheet is usually a pre-printed form with columns laid out for each piece of information (see Figure 35.19). The heading is filled in with the name of the jobsite, along with any reference numbers, file numbers pertaining to the construction drawings, the party chief's name, and the cutsheet preparer's name. The manhole designations and stations, with the elevations of each hub, are copied from the field notes and placed in their respective columns. The proposed inverts, taken from the profile sheet, are filled in next, along with the size of pipe. The pipe slope is calculated and compared with the profile, and the remaining inverts are computed. Invert elevations for the house or building laterals are calculated at the mainline, at the riser, and at the end of the lateral.

The vertical difference between the elevation of the hubs and the elevation of the sewer main and laterals are com-

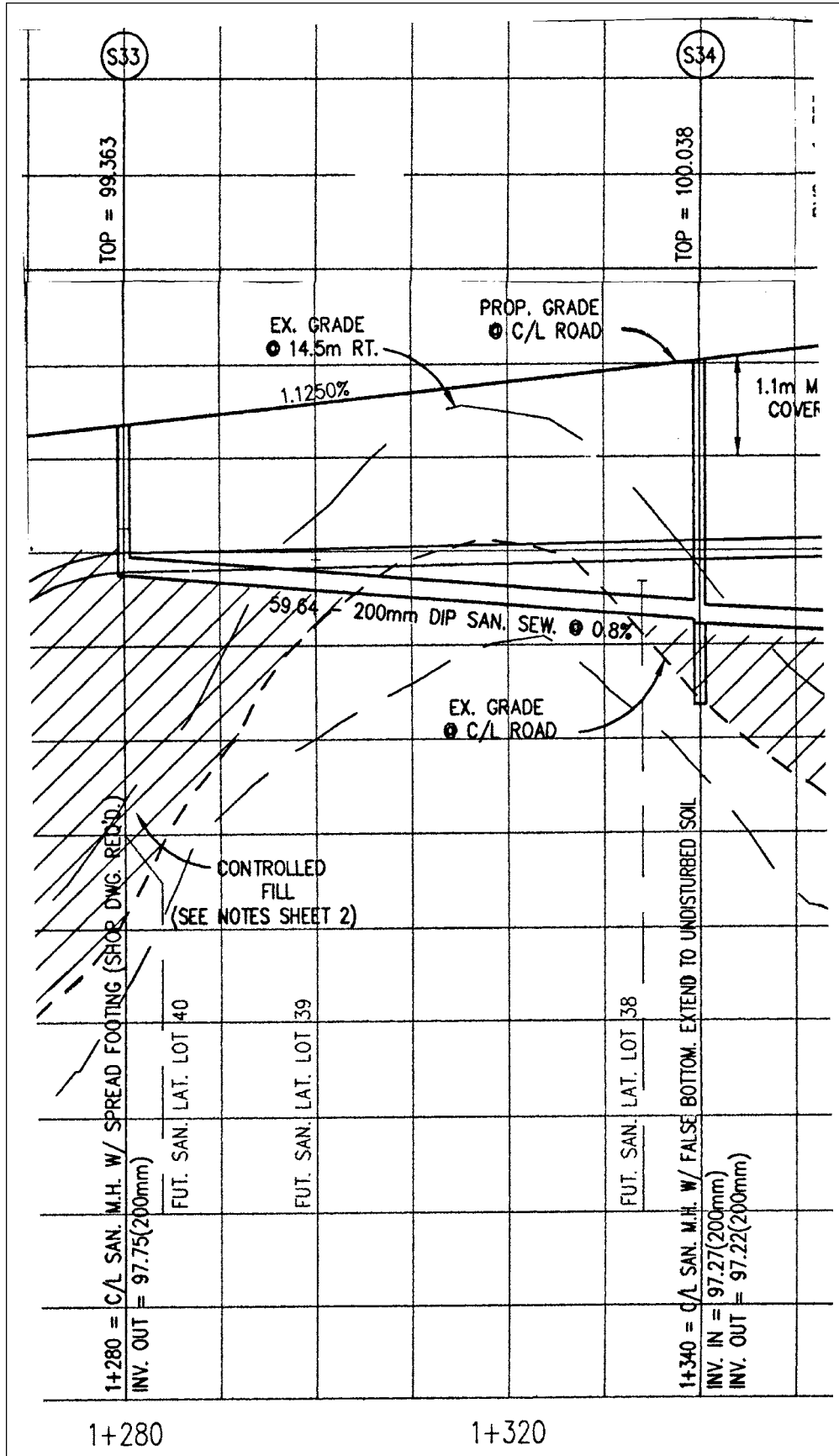


FIGURE 35.19 Sanitary-sewer profile (metric).

puted for each station and entered in the appropriate column. This vertical difference is the amount of “cut” that the backhoe will have to excavate to reach the proposed invert elevation. Final checks include scaling the location of the laterals on the plan and profile sheet, and checking the record plat and hard copy to verify that they coincide with the stakeout. Copies of the completed cutsheet are sent to the proper municipal agency, if required, and to the contractor.

## STORM-SEWER STAKEOUT

A legitimate concern for any jobsite is how and where storm water will drain. Even the most thoughtfully scrutinized plans can have a problem area, considering the unpredictability of Mother Nature, human error in construction, and the variable qualities of materials and equipment. At times, corrections are made in the field and in the cutsheets to insure positive drainage and no ponding. Because it is such a critical component, frequently storm-sewer stakeout is more exacting than other types of stakeout. Storm-sewer structures, tops, and throats must match the paving and the curb and gutter in the street, contours in graded lots, and existing swales in the outfall, in addition to carrying the required amounts of water.

### Storm-Sewer Structures in Street Right-Of-Way

Catch-basin locations are determined by street geometry such as stationing, width, and slope. Typically, the storm-sewer system is built before the curb and pavement of the street. To insure a proper match of catch basin to street geometry, accurate street centerline control is surveyed and verified using appropriate control points. Stationing points, directly related to the storm-sewer structures, are set along the centerline during this process. In a tangent section of street, the centerline of street point, opposite a storm structure, is occupied and a 90° angle is turned to set the centerline of the structure. When the structure is located in a street curve, a deflection angle is turned to set the stationing of the structure along the street centerline, and a radial line then determines the centerline of the structure.

The two critical dimensions used to determine the location of the structure are the centerline of street to face of curb and face of curb to centerline of structure. For the former, the party chief consults the plan view along the entire street length, watching for dimension contradictions or street transitions, and scales the dimensions shown on all the drawings. At street intersections, the side street’s plan view is checked in a similar fashion to determine the width of the street. Specifications for the structure type must be consulted to determine the distance from the face of curb to the centerline of the structure.

Many contractors request stakes that relate the storm-sewer structure to the face of curb. This is to insure accurate placement of the structures for eventual matchup to curb and gutter. Along tangent sections, offsets are set along the face of curb with the offset distances, relating to the centerline of structure, indicated on the stakes. In a curve, the

offsets are placed on a line perpendicular to the radial line. The hubs may be placed to the left and right of the structure to allow the contractor to string-line between, or they may be set to one side of the structure for a line the contractor can extend. Both of these methods are shown in Figure 35.21.

Checking the face of curb offset hubs is accomplished by calculating the hypotenuse distances from the centerline of the storm-sewer structure, or from the nearest centerline street station. When applicable, distances between face of curb hubs on parallel structures can be measured as an additional check.

The pipeline offsets are staked once the centerline of each structure and, if applicable, the face of curb hubs have been set. The offset distance is determined with the contractor and, again, by the depth of pipe shown in the plans and profiles (see previous discussion of dirt-pile and trench clearance in sanitary-sewer section). The issue of which side of the line to place the offsets is usually determined by on-site conditions and practical considerations. Obvious choices include placing the offset on the side away from the street, on the side most protected from construction machinery, and/or the “high side” of the construction area.

Similar to a sanitary-sewer stakeout, the hub for the pipe that discharges into the downstream structure is designated as the OUT, with a station 0+00. The IN hub is the inlet side of the same pipe, at the next upstream structure, whose station is designated by the distance between the two structures. Once the OUT and IN offset hubs are staked, the transit occupies one, backsights its corresponding structure, and turns 90° to foresight the next structure’s offset. This serves as a check that the offset distance is the same for both structures. Offset hubs along the pipe length are set at 50-ft intervals, while any sanitary sewer lines or other pipe and utility crossings are offset as well. The stations for any crossings are referenced from the profiles. To have crossings staked in the field, while appearing on the cutsheet as well, helps the contractor avoid contact with existing lines by maintaining the minimum amount of clearance. Elevations are run on the entire stakeout using two or more benchmarks and using the IN hubs as turning points, and a cutsheet is prepared (see Figure 35.22).

### Outfalls and Storm-Sewer Structures In Non-Paved Areas

In many instances, storm-sewer easements are parallel or nearly parallel to property lines. The bearings of the property line and the easement will be compared to see if they are parallel. After checking the record plat for the needed easement information, the party chief should check that the sewer line agrees with the plan and profile and note the distance between the centerline of the easement and the nearest property line. After insuring that the storm sewer complies with easement restrictions, the stakeout then proceeds as outlined above.

Storm-sewer outfall structures (see Figure 35.23) must match to the existing discharge system horizontally and vertically, whether they are swales, ditches, streams or ponds.

**Dewberry & Davis LLC**

Project: Dominion Valley Cntry Club  
 Job No: 98-030  
 Sec.10 Clubhouse/Recreation Center

Battlefield Business Park  
 10525 Battleview Parkway Manassas, VA 20109

Tel #: (703) 631-3430  
 Fax #: (703) 631-0234

Page: 1

**Sanitary Sewer Cutsheet**

Grade To: Invert of Pipe  
 Location: Ex.22 - 1 - 2 - 3 - 4 - B1

Prepared By: R. Root Date: 8/9/2001  
 Staked By: D. Cook Date: 8/8/2001  
 Checked By: Date:  
 File:

Description	Offset	Station	StakeElev	GradeElev	Cut/Fill	Comments
SSO1						
EXMH52/22(IN)	15RT	00+00	380.72	369.68	C- 11.04 '	171.86' - 8" PVC @0.55 %
	15RT	00+50	382.24	369.96	C- 12.28 '	
	15RT	01+00	383.80	370.23	C- 13.57 '	
	15RT	01+50	385.37	370.51	C- 14.86 '	
MH1(OUT)	15RT	01+71.86	386.04	370.63	C- 15.41 '	TOP=386.42
SSO2						
MH1(IN)	15RT	00+00	386.37	370.83	C- 15.54 '	145.90' - 8" PVC @0.55 %
	15RT	00+50	387.26	371.10	C- 16.16 '	
	15RT	01+00	388.31	371.38	C- 16.93 '	
MH2(OUT)	15RT	01+45.90	390.32	371.63	C- 18.69 '	TOP=390.85
SSO3						
MH2(IN)	15RT	00+00	389.97	371.83	C- 18.14 '	184.49' - 8" PVC @0.55 %
SLTCLUBHSE	15RT	00+21.40	390.88	371.95	C- 18.93 '	
	15RT	00+50	392.12	372.11	C- 20.01 '	
	15RT	01+00	393.11	372.38	C- 20.73 '	
	15RT	01+50	393.28	372.66	C- 20.62 '	
MH3(OUT)	15RT	01+84.49	392.68	372.85	C- 19.83 '	TOP=392.67
SSO4						
MH3(IN)	15RT	00+00	392.61	373.05	C- 19.56 '	378.50' - 8" PVC @0.55 %
	15RT	00+50	388.63	373.33	C- 15.30 '	
	15RT	01+00	387.21	373.60	C- 13.61 '	
	15RT	01+50	385.81	373.87	C- 11.94 '	
	15RT	02+00	385.48	374.15	C- 11.33 '	
	15RT	02+50	385.56	374.42	C- 11.14 '	
	15RT	03+00	385.32	374.70	C- 10.62 '	
	15RT	03+50	386.47	374.97	C- 11.50 '	
SLTREC	15RT	03+71.54	386.33	375.09	C- 11.24 '	
MH4(OUT)	15RT	03+78.50	386.25	375.13	C- 11.12 '	TOP=387.74
SSOEXB1						
MH4(IN)	15LT	00+00	385.73	375.33	C- 10.40 '	153.98' - 8" PVC @2.38 %
	15LT	00+50	386.69	376.52	C- 10.17 '	
	15LT	01+00	387.91	377.71	C- 10.20 '	
EXMHB1(OUT)	15LT	01+53.98	388.17	379.00	C- 9.17 '	TOP=388.73

USE A MINIMUM OF THREE (3) HUBS FOR LINE AND GRADE

FIGURE 35.20 (a) Sanitary sewer (b) Sanitary lateral cutsheet.

**Dewberry & Davis LLC**

Project: Braemar Ph.2 Sec.21A  
 Job No: 87-018  
 # 01-00150

Battlefield Business Park  
 10525 Battleview Parkway Manassas, VA 20109

Tel #: (703) 631-3430  
 Fax #: (703) 631-0234

Page: 1

**Sanitary Lateral Cutsheet**

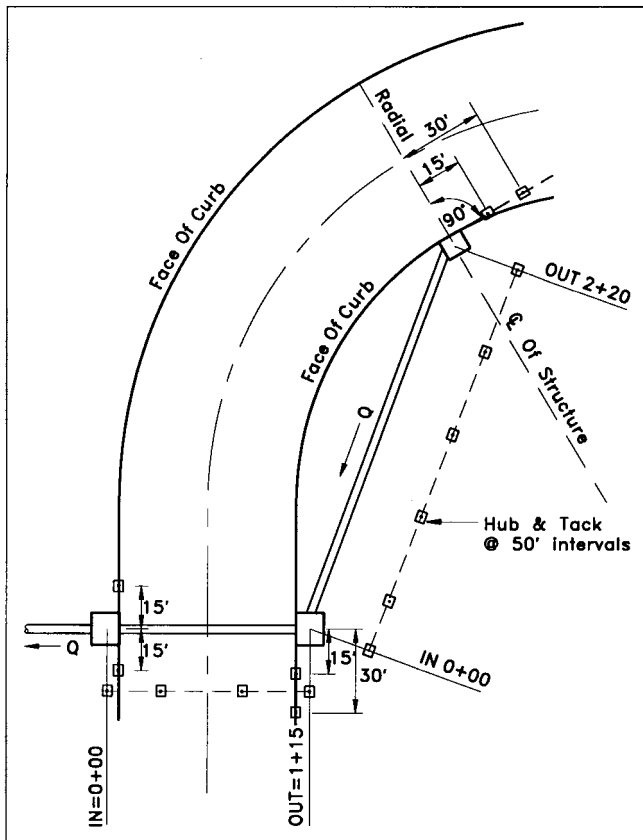
Grade To: Invert of Pipe  
 Location: MH4 - MH5  
 4" Laterals @ 2.08%  
 \* = RESTAKE

Prepared By: R. Root Date: 6/26/2001  
 Staked By: B. Glascock Date: 6/25/2001  
 Checked By: Date:  
 File:

Description	Offset	Station	StakeElev	GradeElev	Cut/Fill	Comments
						BR = BEGIN RISER
						ER = END RISER
						P/L = PROPERTY LINE
						E/L = EASEMENT LINE
SSSO5						
MH4(IN) 00+00	15'LT		234.761			
SLT22 00+60	15'LT	00+00	235.06	229.07	C- 5.99 '	TEE LOT22
	28.96'AHD	00+39.04				P/L
SLE22	15'AHD	00+53	234.17	230.14	C- 4.03 '	END
SLT37 01+15	15'LT	00+00	235.11	229.64	C- 5.47 '	TEE LOT37 *
	30'AHD	00+18	237.44	230.02	C- 7.42 '	BR
	28'AHD	00+20	237.44	232.06	C- 5.38 '	ER
	18.18'AHD	00+29.82				E/L
SLE37	15'AHD	00+33	237.44	232.33	C- 5.11 '	END
SLT21 01+25	15'LT	00+00	234.61	229.74	C- 4.87 '	TEE LOT21 *
	29.06'AHD	00+35.94				P/L
SLE21	15'AHD	00+50	234.77	230.79	C- 3.98 '	END
MH5(OUT) 01+55	15'LT		235.821			

USE A MINIMUM OF THREE (3) HUBS FOR LINE AND GRADE

**FIGURE 35.20** (Continued) (a) Sanitary sewer (b) Sanitary lateral cutsheet.



**FIGURE 35.21** Face of curb offsets for storm-sewer stakeout.

Swales and ditches may exist naturally, but more often than not they are designed to transition the storm-sewer system into the natural system. Some are made of concrete and are designed to channel storm water directly into another structure in environmentally-sensitive areas. In the field, the party chief may have to adjust the stakeout to create a better fit. In such cases, the new layout is located and the information is then sent to the engineer for confirmation.

When the storm-sewer system discharges into a natural outfall, the survey crew takes the time to locate, vertically and horizontally, the outfall channel a minimum of 150 ft downstream from the point of discharge. The discharge point is designated as the OUT and, if it is assigned a station of 0+00, downstream shots are designated as -0+50, -1+00, -1+50 and so on. Any horizontal change of direction on the outfall should be noted and sketched, with a distance measured to the centerline of the bend and its elevation obtained (see Figure 35.23). This information serves to verify the location of the storm-sewer structure in relation to the limits of the easement and the natural drainage system. After all field stakeout work is complete, a cut-sheet is prepared and copies delivered to the contractor and other appropriate agencies.

### Storm Water Retention and Detention Ponds

Storm water retention and detention ponds are staked along the proposed grading contours. Hubs typically are set to

locate the toe of the proposed berm and also to mark the top of the proposed berm. Stakeout markers are set at horizontal bends and at regular intervals to delineate the size and shape of the pond. A limited number of stakes should be used, as setting too many markers may conflict with the operation of large earth-moving equipment. The stakes set along the proposed contours are generally set at every two to three contour intervals for the same reason.

Preliminary preparation for this stakeout includes plotting traverse, or other control points, on the site plan. Angles and distances to each pond point are protracted and scaled directly on the plan. A transit-and-tape, or total station and data collector, may be used to stake the points, which usually consist of a hub and a 4-ft-tall guardstake. The hub is the horizontal and vertical point, while the guard carries the necessary written information, such as the proposed contour or toe/top designation, the hub elevation, and the cut or fill to the desired elevation. A pond stakeout requires minimum accuracy: to the nearest foot horizontally and to the nearest tenth of a foot vertically (see Figure 35.24).

### WATERLINE STAKEOUT

Many waterline layouts are frequently designed to parallel (or closely follow) the centerline of street and are profiled coincident with the street stationing. Although waterline pipe is manufactured in straight pieces, it can be installed on curve-chord deflections. Other types of fittings are used for more abrupt changes of direction. Such fittings include tees, crosses, bends of various angles, and wyes. The dimensions and stations of the waterline should be scaled and compared to the written information in the plans and profiles, verifying the location of all valves, tees, blow-off valves, air-release valves, and fire hydrants.

Double offset hubs are used to stakeout tees and bends, while fire hydrants require multiple offset hubs. The stakeout of the rest of the waterline is achieved by converting the centerline of street into an offset line. This procedure is recommended if the distance between centerline of street and centerline of waterline is anywhere from 7 to 12 ft. The depth and width of the average waterline trench does not require a long offset.

The profile shows the horizontal and vertical path of the waterline and its relationship to the storm and sanitary lines. Crossings should be noted and staked, especially if there is minimum clearance either vertically or horizontally between the two utilities. If there are minimum horizontal clearances between the waterline and the sanitary lines, the waterline should be adjusted to maintain an adequate distance, especially if the waterline passes near the barrel vault of a sanitary structure.

In easement conditions, such as over terrain or through parking areas, the waterline will have its own stationing system shown in the plan and profile. This stationing must be adhered to as closely as possible, as this is the system approved by the local governing authorities. If there are inaccuracies, sometimes a station equality is placed on the

**Dewberry & Davis LLC**

**Project: Braemar Ph.2 Sec.21A**  
 Job No: 87-018  
 # 01-00150

Battlefield Business Park  
 10525 Battlevue Parkway Manassas, VA 20109

Tel #: (703) 631-3430  
 Fax #: (703) 631-0234

Page: 2

**Storm Sewer Cutsheet**

Grade To: Invert of Pipe or As Noted  
 Location: Str.30 - 31 - 32 - 33 - 34 - 35,  
 32 - 36 - 37

Prepared By: R. Root Date: 9/12/2001  
 Staked By: M. Cook Date: 9/11/2001  
 Checked By: Date:  
 File:

Description	Offset	Station	StakeElev	GradeElev	Cut/Fill	Comments
<u>SMSO36</u>						
STR32(IN) MH	15RT	00+00	261.80	259.30	C- 2.50 '	204.01' - 15" RCP @1.57 %
	15RT	00+50	263.82	260.08	C- 3.74 '	
	15RT	01+00	264.48	260.87	C- 3.61 '	
	15RT	01+50	265.96	261.65	C- 4.31 '	
STR36(OUT) DI-3B	15RT	02+04.01	267.57	262.50	C- 5.07 '	
FC@STR36	15LT	13+73	267.14	269.02	F+ 1.88 '	TOP OF CURB
FC@STR36	30LT	13+88	267.44	269.02	F+ 1.58 '	TOP OF CURB
<u>SMSO37</u>						
STR36(IN) DI-3B	15RT	00+00	267.10	262.60	C- 4.50 '	32.00' - 15" RCP @0.62 %
	15RT	00+16	267.16	262.70	C- 4.46 '	
STR37(OUT) DI-3B	15RT	00+32	267.04	262.80	C- 4.24 '	
FC@STR37	15RT	13+73	267.08	269.02	F+ 1.94 '	TOP OF CURB
FC@STR37	30RT	13+88	267.34	269.02	F+ 1.68 '	TOP OF CURB

USE A MINIMUM OF THREE (3) HUBS FOR LINE AND GRADE

**FIGURE 35.22** Storm-sewer cutsheet.



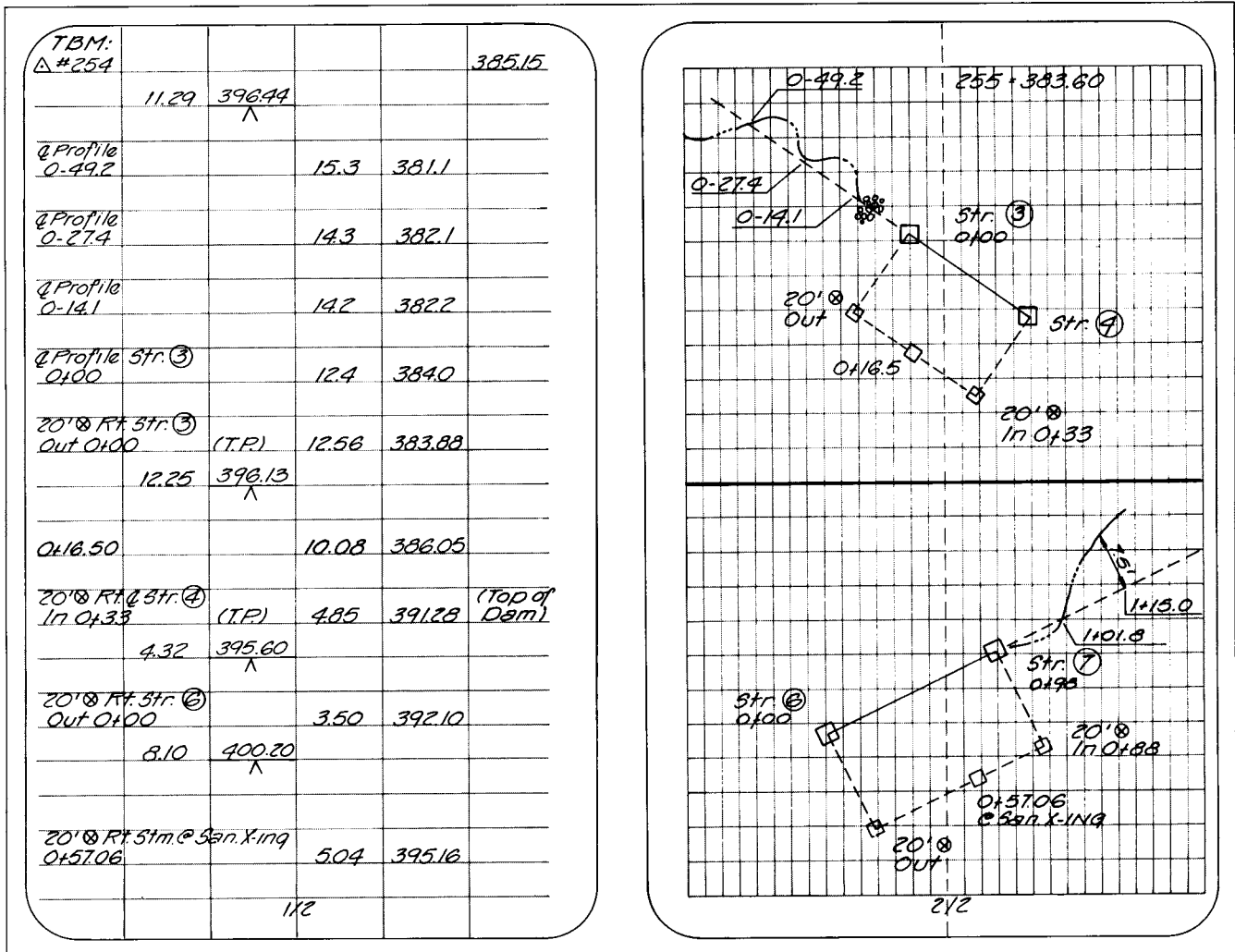


FIGURE 35.23 Storm-sewer outfall notes and sketch.

stakes, noted by the plan and profile station first, followed by the equal sign and the stakeout station. This allows for correction in the stationing, which matches the remainder of the stakeout to the plan stationing.

**Fire Hydrants**

Fire hydrants must be placed accurately horizontally and vertically. The stakeout is based on the relationship between the curb line (or edge of pavement/shoulder) and the main waterline. The critical dimension, the distance from centerline of street to the face of curb, is the reference point that controls the hydrant location. As shown in Figure 35.25, a hub marking the tee for the stub line leading to the fire hydrant is set along the offset line for the main waterline. Two additional offset hubs mark the extended stub line beyond the hydrant. Two hubs are then set marking the face of curb or edge of pavement. String lines stretched between the hubs marking the face of curb and from the hubs beyond the fire hydrant to the hub for the tee mark the intersection of the curb and the waterline stub to the fire

hydrant. The guard stakes indicate the elevation of the curb or edge of pavement, and from these references the contractor can place the fire hydrant.

Hydrant installation is expensive and, because of the poured concrete surge block, very permanent. Setting the face of curb and having the contractor install the hydrant to the code criteria involves a shared responsibility.

As an additional precaution, fire-hydrant stakeouts are tied to other street or traverse control. The notes for hydrant locations are later checked and the stakeout location verified. When elevations are run, the rod is read once directly in a natural position, and then flipped and read inverted. The two readings should add up to the total height of the rod, thus verifying the accuracy of the initial reading. This procedure is used for three of the offsets; the fourth offset is employed as a turning point in the level circuit.

**Valves and Other Waterline Fittings**

Tees are 90° fittings, typically occurring at connections to existing lines, stub lines to fire hydrants, and intersections

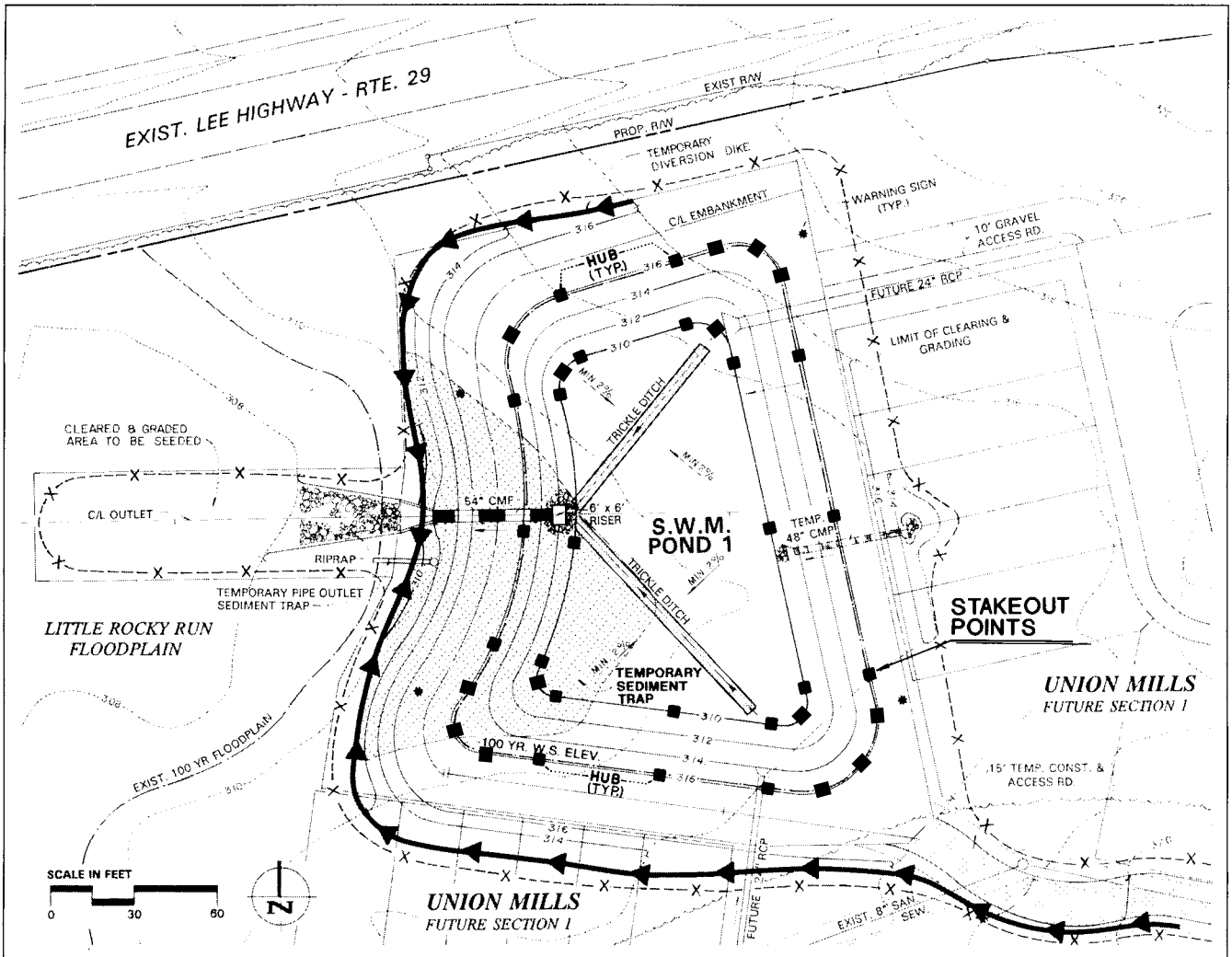


FIGURE 35.24 Pond contours and stakeout points.

with other waterlines. They are staked with double offsets, one indicating the line direction back, and the other indicating the line ahead.

Horizontal bends in the waterline are also staked with double offsets, each one set 90° from the direction of the line it is representing. Both bend offsets carry the same station, as they address the same point. To distinguish them, one is called the Bend Back, and the other is the Bend Ahead, based on the stationing system and its point of beginning (see Figure 35.26).

Blow-off valves are located at the end of a waterline. In some cases, they deviate from the street centerline. Therefore, their placement may be determined by scale and protractor from the site plan, if no other exact location means is available. The plan-view location is then compared to the profile. Intervals between offset hubs and tees, bends and valves is often 50 ft, but 25-ft intervals are sometimes employed in horizontal and vertical curves.

The level circuit begins at the existing waterline, with an elevation obtained on the top of the existing valve. Only

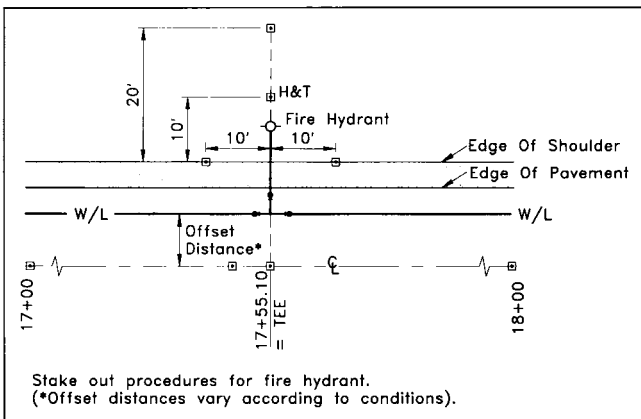
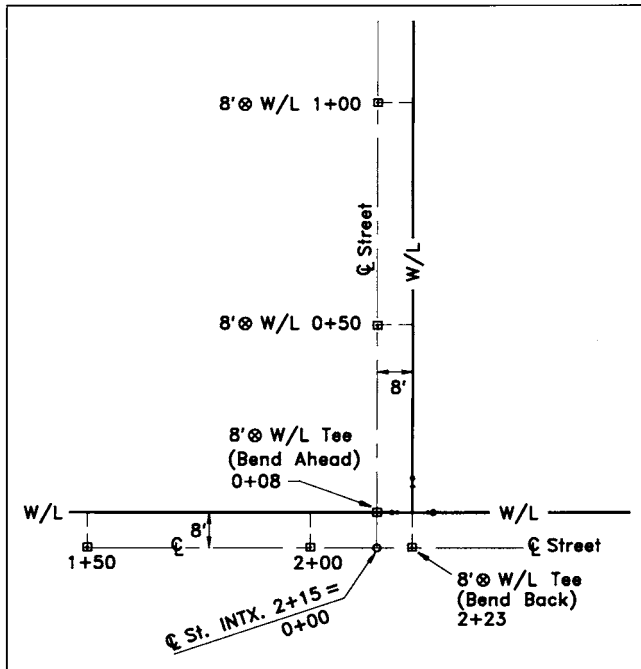


FIGURE 35.25 Stakeout procedures for fire hydrant.



**FIGURE 35.26** Waterline tee offsets.

the offset hub elevations are necessary when the waterline is in the street, but in easements the ground should be profiled to check the topography.

### Service Lines

A separate stakeout is made for water crocks, which mark the limits of public service and from which the private line extends to serve the lot. Water crocks contain the meters and cut-off valves for each service line. The crock location is marked either with paint on the curb or with hubs and guards. There is usually a code distance between the property line and the crock. Either the property line is marked at the point where the crock is to be installed, or the property line is referenced by offsets in order to preserve the stakeout. In either case, the stakeout is never the crock itself but is referenced to the property line at the crock. One offset with an elevation given for each crock is generally all that is necessary to build the crock and its lateral from the main.

Waterline laterals that service high-rise office and apartment buildings are staked with multiple offsets, indicating the tee location, intermediary station(s), bends and the connection to the building. The site of the existing building connection may differ from the plan, as architectural plans are not always finalized before the site plans are designed. A consultation with the contractor or plumber is recommended before staking building connections. At times, dimensions pinpointing the plumbing inside the building are measured and recreated outside to locate the connecting point. The stakeout then proceeds in the above-described fashion.

### CURB AND GUTTER STAKEOUT

Curb and gutter is expensive, therefore it is wise to exercise due diligence during all aspects of the construction stakeout process. Using a copy of the record plat, construction drawings, and the stakeout hard copy, the street's main control points are set and ties are made to existing traverse control or other street references as a check for accuracy. Before staking, the site plan, street profiles and typical street sections are thoroughly checked for street width(s), transitions, vertical curves, radius dimensions, and parking-space dimensions (where applicable). The following method provides sufficient guards to accuracy.

The main points, including point of curves (PCs) and point of tangents (PTs), are usually echoed in the layout of the curb when the curb is parallel or concentric to the centerline. Other points identifying stations where street-width transitions begin and end, intersecting streets, street entrances to commercial property, and stations corresponding to curb returns are set along the centerline as the stakeout progresses. The major points for the curb are transferred laterally from the centerline (90° in tangent, radially in curves) at the prescribed distance to the face of curb plus the offset distance (see Figure 35.27). Seven and 10-ft offsets are commonly used, as this is often the distance to the property line and convenient for use with a slip-form paver (curb and gutter pouring machine).

For a curb return (the point of curve or point of tangent at a street intersection), the 90° (tangent) or radial (curve) line from centerline also sets the radius of the return. When the intersecting street curb is staked, the validity of the radius point is checked by the same method (i.e., setting the stationing point on the street centerline corresponding to the curb return and then checking the angle and distance to this point, as well as the previously set radius point). Then the corresponding PC or PT offset may be set. Fifty-ft intervals are adequate along tangent sections, while 25-ft intervals are used through horizontal and vertical curves. Points along a horizontal curve on the curb-offset line are set the same way as staking out a horizontal curve.

To "prove out" a point between main stations, the distances from each end must be checked. On a curb and gutter stakeout involving the placement of driveway aprons, each distance between driveways should be measured and the sum of the increments compared to the total distance to eliminate any error. When staking a curve, the total deflection angle and distance is checked first, then the intermediate stations are set. The final check-in distance will prove the accuracy of the intermediate points set.

Other features should be checked as the curb and gutter stakeout progresses. Distances (and occasionally angles) are measured to storm structures, sanitary manholes near the curb line, fire hydrants, and the match point where existing curb and gutter meet the proposed curb and gutter. The plans are consulted frequently to see that the design location of such features have been constructed or located in the field properly.

### **Curb Offsets through Street Transitions**

Street transitions occur when the street changes width between intersections because of the addition of turn lanes. The transition occasionally occurs on one side of the street only or on one side of the intersection only. Transitions also occur with the addition of a lane to increase traffic capacity.

A smooth transition design can simply be two reverse curves. The beginning, midpoint, and end-of-transition points are set at their stations along the street centerline. Then each is offset radially behind the face of curb at the transition widths noted in the plans. If the curves are equal in length, the distance to the midpoint, which is the point of reverse curve, will be half the difference between the two ends. Once the key points are offset, deflection angles and chord distances are used to set intermediate stations along each offset line of the curve.

### **Cutsheets for Curb and Gutter**

Horizontally, curb offsets are always staked in relation to the face of curb. Vertically, the top of curb is frequently used as the reference point. Preparation of a cutsheet for streets with constant geometry is straightforward. The proposed street grades can be determined from the profile and the typical section. This grade is compared to the existing elevation, obtained from the level notes of the stakeout, and the corresponding cut and fill depths are entered onto the cutsheet.

Where the proposed curb design connects to an existing curb, a minimum of three top of curb elevations along the existing curb at 25-ft intervals are obtained as the level circuit is run for the stakeout hubs. This provides the cutsheet preparer with the information needed to check how well the newly constructed curb matches the existing curb. If the proposed grades do not match the existing grades, the cutsheet preparer should consult with the engineer. Otherwise, the cutsheet preparer, using good judgment, may make slight adjustments to the elevations to provide a smooth vertical connection between the existing and proposed curb and gutter.

A frequent problem when preparing cutsheets of curb and gutter for parking areas is a misinterpretation of the elevations listed on the site plan. Whereas the proposed top of curb elevations can be found in the street plans and profiles, curb elevations may not be as readily available on plans of areas where uniform geometry does not exist, such as parking areas or combination travel ways and parking frequently used in townhouse projects. Since parking areas are warped to facilitate drainage and to accommodate the site conditions, spot elevations are used in conjunction with grading contours in the design. Drainage problems or uneven curb and gutter sections may result if spot elevations are too sparse or are not interpreted properly by the preparer of the cutsheet.

In such cases, the person preparing the cutsheet must check the viability of the proposed top of curb elevations

throughout the stakeout area. A careful inspection is made of the proposed topography to insure that the new curb and gutter will actually channel storm water to catch basins and drains. Care must be taken to recognize different types of curb and gutter, and the transitions that link them. The cutsheet must provide for smooth vertical transitions along the curb line.

### **Curb and Gutter Stakeout in Townhouse and Parking Areas**

Townhouse and parking lot curb and gutter stakeouts are generally performed as previously described. However, there are a few additional items to consider. Chief among them are travel-lane widths, parking bays, islands and medians, and depressed curbs for ramps (see Figure 35.28).

In some areas, municipal code may dictate a minimum width for travel lanes, particularly to insure accessibility for fire and rescue vehicles. It is prudent to know the minimum standards and to check the site plan for discrepancies before proceeding with the stakeout.

Parking bays are designed to contain certain numbers of vehicles, usually noted on the site plan along with the width of each space. The number of parking spaces multiplied by the width of each space should equal the distance between the two corners of the bay. This should be compared to the stationing system to insure the distance is accurate. If there are any handicap parking spaces, the additional width should be added to the total distance as well.

Islands are often used to channel traffic, to create additional parking, or to accommodate central features such as tot lots, community mailboxes, light poles, fire hydrants, and entrance gates. Generally, islands that channel traffic are called medians and may consist of solid concrete or curb and gutter on each side with a planted strip in the center. Some may incorporate transitions to accommodate turning lanes. Depending upon the size and shape of the median, the contractor may request a single row of offsets down the middle, or two rows of offsets each dedicated to a side of the island. If the latter is requested, it is preferable to offset each side away from the center to avoid confusion of crossing offsets. Medians usually terminate in small curves, so the radius points should be clearly marked and graded for inclusion on the cutsheet (see Figure 35.29).

If an island incorporates utilities such as fire hydrants, storm-sewer structures, sanitary manholes, or light poles, checks should be made in the field to insure proper match-up and/or containment of these features both horizontally and vertically.

Finally, the location of depressed curb for handicap ramps, public trails or loading zones may be a required part of the stakeout. A careful study of the site plan will reveal the public-area locations, and the record plat will aid in the siting of trails within easements.

### **BUILDING STAKEOUTS**

The types of buildings typical in land development projects include single-family-detached residential units, single-

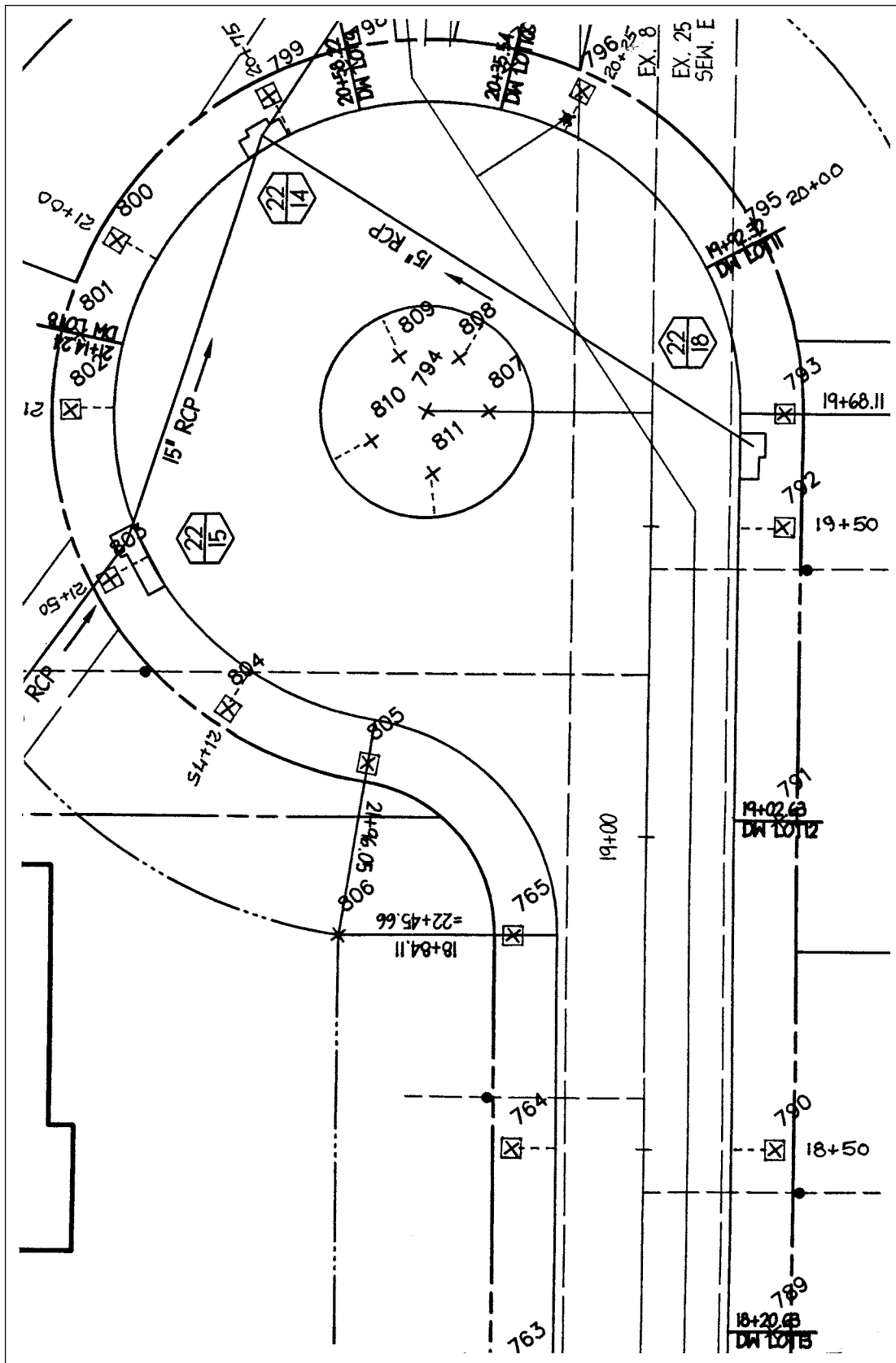
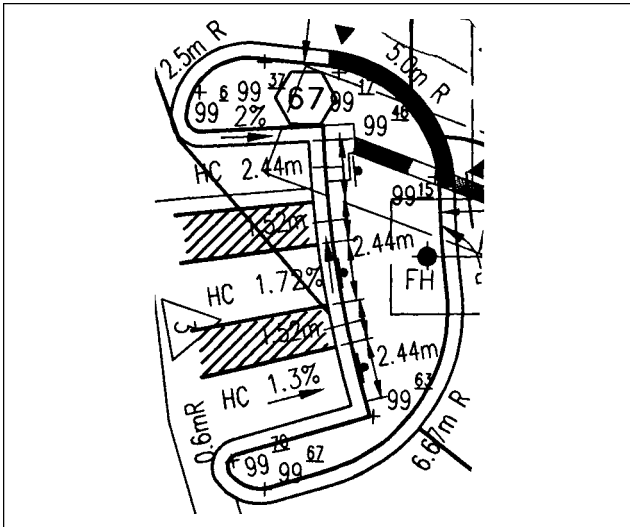


FIGURE 35.27 Curb stakeout offsets in a curve.



**FIGURE 35.28** Curb and gutter island with parking bay.

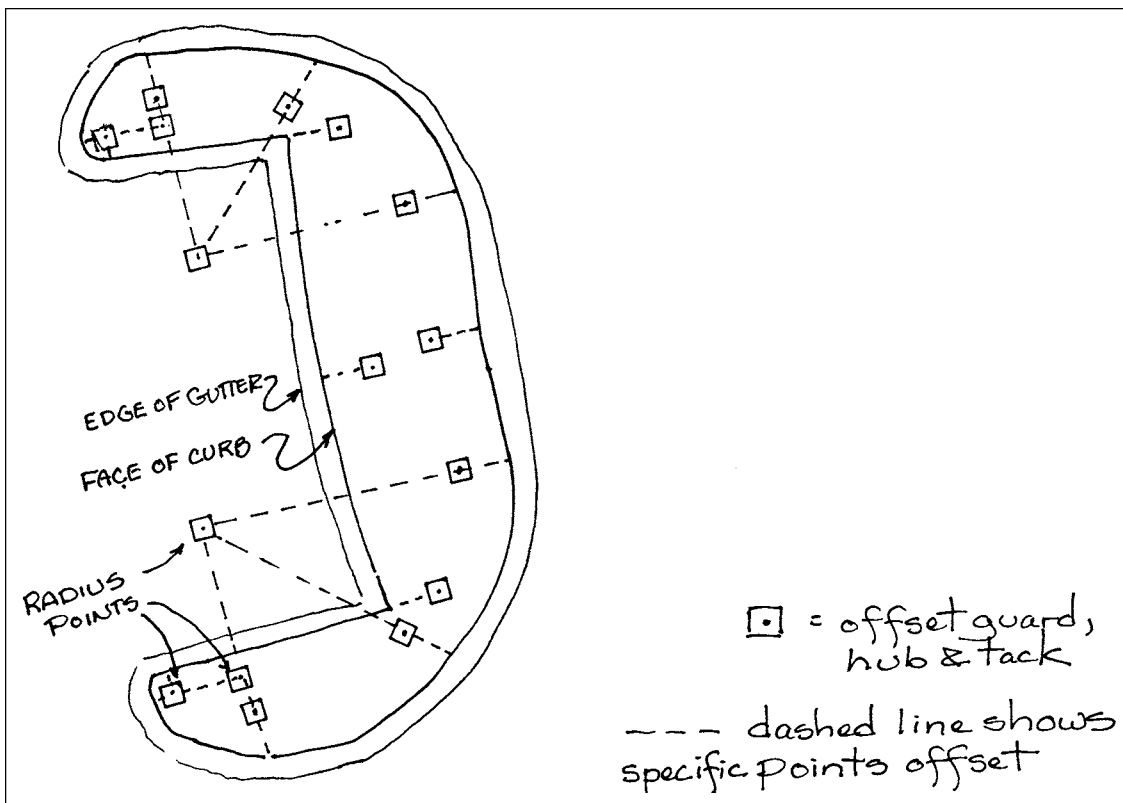
family-attached units (townhouses), and low and mid-rise retail and commercial buildings. Two important considerations in establishing the building location for the stakeout survey are adherence to zoning setback restrictions and allowable accuracy for construction. Although the building is shown conforming to setback restrictions on the plans (otherwise the plans would not have been approved), the stake-

out computations and the subsequent field procedures must insure that the field location is consistent with the plan location. Additionally, the dimensions of the building and the corner angles must be set to within the tolerable limits required for construction purposes. Building stakeout can be as straightforward as setting offset stakes for four corners or as complex as setting column-line locations for a high-rise structure.

Several construction phases of a building require separate stakeouts: grading, building layout, footing, and brick point stakeouts. In most cases, all of these are used for residential construction. However, commercial buildings and other types with atypical foundations or special design characteristics may require unique stakeout methods different from those mentioned.

**Grading Stakeout for Buildings**

Preliminary preparation of a site involves the removal of topsoil and the manipulation of the earth to bring it to subgrade level. To accomplish this, the contractor requires a grading stakeout. The most expedient method extends the outermost lines of the building to create a simple rectangle, whose corners are staked with offsets (see Figure 35.30). Usually, a 10-ft offset is sufficient to enable earth-moving equipment to move about freely and yet retain the stakes for horizontal and vertical referral. Since this stakeout is merely intended to guide rough grading, the hubs are not



**FIGURE 35.29** Parking lot curb island stakeout examples.

tacked, but they are guarded with tall stakes for easy visibility.

The offsets markers are plotted on the grading plan and then set by computing or protracting the angles and distances from available horizontal controls such as lot corners, centerline street points or traverse stations. Once the grading stakes are set and checked, a level run determines the existing ground elevation of each hub. The elevations are marked on the back of each guard. The field notes are checked by office personnel and a cutsheet based on finished basement-floor grades is issued for the contractor's use.

### Construction Stakeout for Buildings

Once the rough-grading operation is complete, the building is staked for construction. To insure compliance with zoning setback requirements, computing the building location should be based on its relationship to the property lines.

Coordinates are set for each corner of the building. Corresponding point numbers are assigned and recorded on

the building stakeout sheet. Once all the building coordinates are set, the actual setbacks and side-yard distances can be computed. Check to see that all setback distances to property lines comply with local code and restrictions. Because of expensive land and rising development costs, planners strive to maximize the number of houses on a site, using the minimum restriction lines. For this reason, there is little room for movement, or error, in the stakeout of a building. As another precaution, a computer plotting should be made and overlain on the development plan to check compatibility.

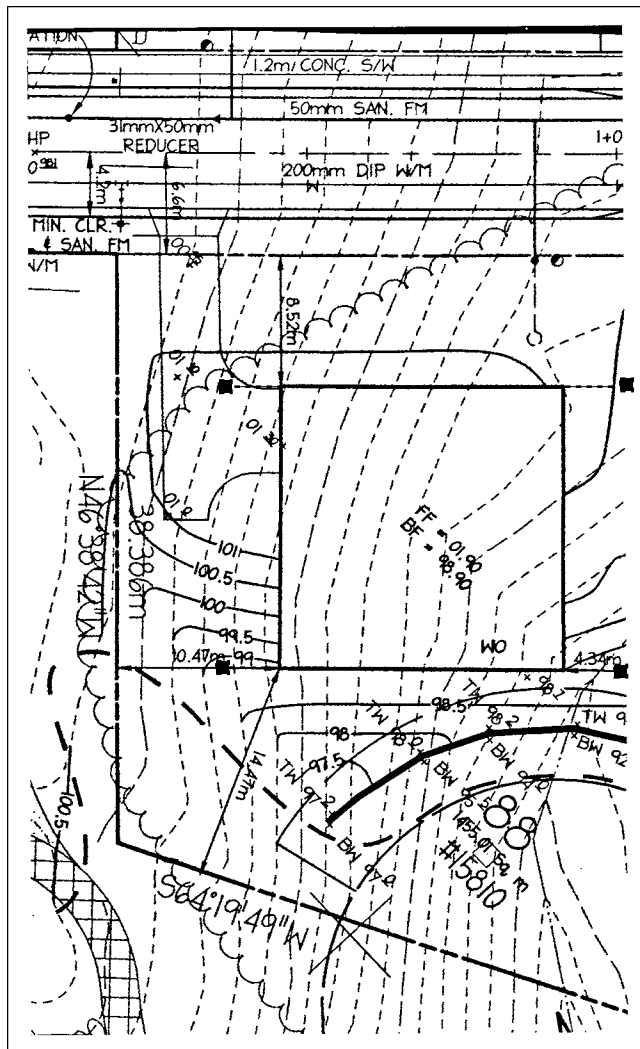
Ten-foot offsets are often used for most building stakeouts; this distance is sufficient to accommodate the over-dig of the basement, or the foundation, and is short enough for the contractor to use accurately. The proposed 10-ft offsets are drawn onto the stakeout sheet; the placement is usually from side-to-side, but front-to-back offsets can be used as well. The front side-to-side offsets will be on the face of building line, 10-ft from the edge of the squared building. The offsets are assigned point numbers and coordinates.

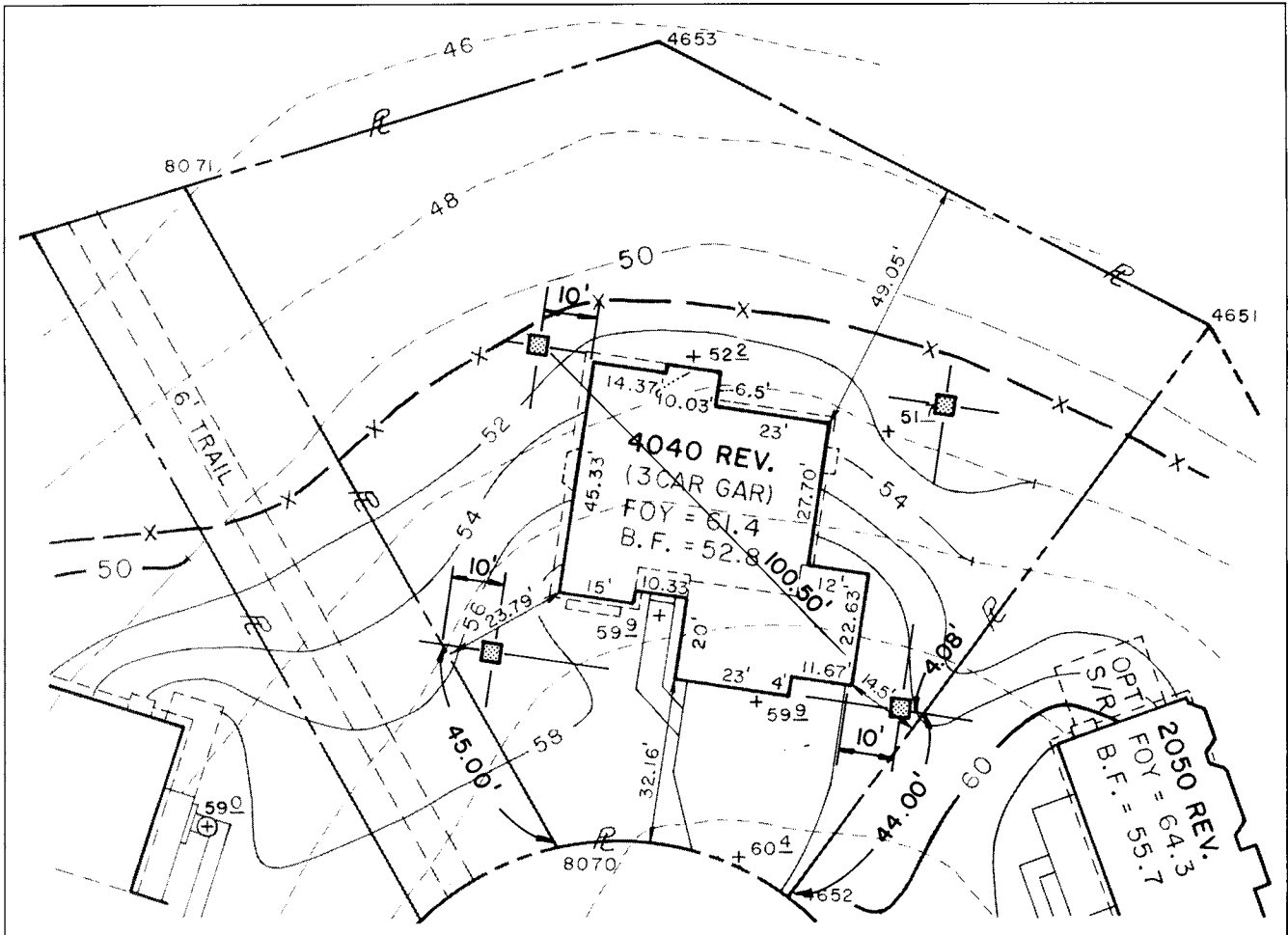
The building stakeout sheet (see Figure 35.31) will have coordinate point numbers written next to each lot corner, at intersection points on lot lines, on offset points, and at the building corners. Stakeouts tied from control points to the offset hubs of the corners of the building may also be shown. A list of the point numbers and their coordinates is attached to the stakeout sheet to facilitate any necessary field computations. In addition, the side-yard distances and the minimum distances as prescribed by zoning regulations are given. Curve information is also supplied where applicable. A copy of the building stakeout sheet is placed on file, and a copy is used by the survey crew.

After the building offset points are set, it is good practice to turn angles and measure distances to other control points that are independent of the stakeout procedure to verify the stakeout. For practical and legal purposes, many surveying companies will require a location made of the building stakeout. Some may furnish a pre-printed form along with the stakeout sheet; one side is a photocopy of the building stakeout sheet and the other is a standardized form with prompts and spaces for answers necessary for the record.

Elevations are run using at least two benchmarks, and a front offset hub is established as a temporary benchmark by using it as a turning point in the level circuit. The lowest floor elevation furnished on the development plan, whether it is Low Basement, Basement or First Floor, is used to calculate the earthwork cut or fill to finish-floor level. In undisturbed areas, it is prudent to record elevations at the building corners to check the topography. On site, the hub elevation is written on the stake along with the cut or fill. Off site, the field notes and the elevations are checked and a formal cutsheet is issued.

**Footing Stakeout.** After the area is excavated, contractors frequently request a footing or foundation stakeout. The building stakeout offset hubs are recovered or replaced,





**FIGURE 35.31** House stakeout showing 10-ft side offsets.

checked, and then used to lay out the actual building corners. The front (or rear) building offset line may be used as a baseline to transfer the corners to the foundation excavation. Another method is a radial stakeout, occupying one offset and using coordinates of each building corner to determine bearings and distances from the setup point. The latter method can be employed successfully if all distances between footing points are checked thoroughly. Diagonal measurements between footing corners are truthful checks of squareness.

The vertical portion of the footing stakeout consists of setting a grade marker, established as a temporary benchmark by using it as a turning point when the level circuit is run. This point is used as a vertical reference to finish grade.

**Brick Points.** After the footing is poured and set, the building is ready for a brick-point stakeout. This consists of setting concrete nails in the footing at each corner or jog of the building. These will be used by masons (block) or a forming crew (poured walls) to align the building walls. The stakeout should be very accurate to provide a square foundation for subsequent work.

Temporary surveyor's tacks can be set as points on a baseline used by turning  $90^\circ$  angles to rear jogs in the building. An alternative is to set the points radially from one point, but this approach dictates the need for extreme care in the setting of each point; otherwise, losses in accuracy may occur, particularly in the squaring of the building. A third method uses a compromise of the above two methods. The front baseline is set and a combination of  $90^\circ$  angles and trigonometrically calculated angles and distances are figured from each main point. The distance between each nail is checked carefully. Generally, the method that provides the most accuracy is the one that relies the least on correction of slope distances. Since the building-stakeout offsets are away from the basement hole and, therefore, much higher in elevation, large vertical angles are the correcting force for slope-taped or total station distances. A small variation in the vertical-angle read, the correction figured, or deviation from the corrected tape height will result in error. When setting brick points that will be placed in a deep foundation hole, it is often best to set the first point diligently as described, and then set the instrument in the foundation hole for subsequent points. Also, each time the



instrument is moved to another nail, a backsight and a check foresight should be made to establish the accuracy of the nail occupied, before setting any other points.

Vertically, masonry brick points often require grade pegs set to finish-floor grade. This involves setting a stake so that the top is equal in elevation to the finish-floor grade (or an even foot higher or lower, as convenient). Another method uses a stake with a nail driven into its side at grade. Poured-wall brick points often circumvent this formality, because the foundation is already in place and functions as the critical vertical surface in relation to the finish-floor grades. Some surveying companies check the elevation of the footer, as a precaution, for a poured-wall brick point stakeout. A consultation with the client on site can determine the necessity of a level run.

**Single-Family Attached Units (Townhouses).** Stakeout for townhouses proceeds through the same stages as that for detached dwellings. The main difference is the need for more offsets to delineate each unit. The overall building, which typically consists of three to nine individual units, is squared as a whole, and its relationships to side, front, and rear property lines are determined. The four corners of the townhouse block will be set and checked first, and then the front and rear lines are used to offset the middle units.

In Figure 35.32, the distance from the side lot line to the building on lot 14 has been computed at 9.25 ft. This distance is measured along the front property line, where a hub and tack is set, and its stake is marked 16 ft offset to

the left front building corner. The figure shows the dimensions along the front and rear of each unit, but for the stakeout the centerline of the wall dimensions is used. Since 23.08 ft is the distance to the outside corner along the front face, a distance of 22.75 ft from the left front corner to the centerline of the wall results if the wall thickness is 8 in (i.e.,  $23.08 - 0.33 = 22.75$ ). Therefore, a distance of 22.75 ft is measured along the property line and another hub and tack is set. This stake is marked 16 ft offset to centerline of wall lot 14 and 17 ft offset to centerline of wall lot 15, using the appropriate symbols and abbreviations. Continuing down the line in this fashion, the stakeout's accuracy is checked by measuring from the last offset—the right front corner of the building—to the lot corner. As shown in the example, the check-in is 9.67 ft.

This accomplished, the two building-corner offsets are occupied, a  $90^\circ$  angle is turned, and the rear building-corner offsets are staked on the rear property line. Moving the instrument to the rear line, the  $90^\circ$  angle and the distance are checked before staking the remainder of the rear-line offsets. As mentioned in the previous section, the surveying company may require that a location be made of the stakeout. When this is completed, a level run will establish the elevation of each front offset. The cutsheet will provide cuts (or fills) to the finish-basement grades of each unit.

The footer stakeout is accomplished just as described in the single-family-dwelling section. Brick points may be set

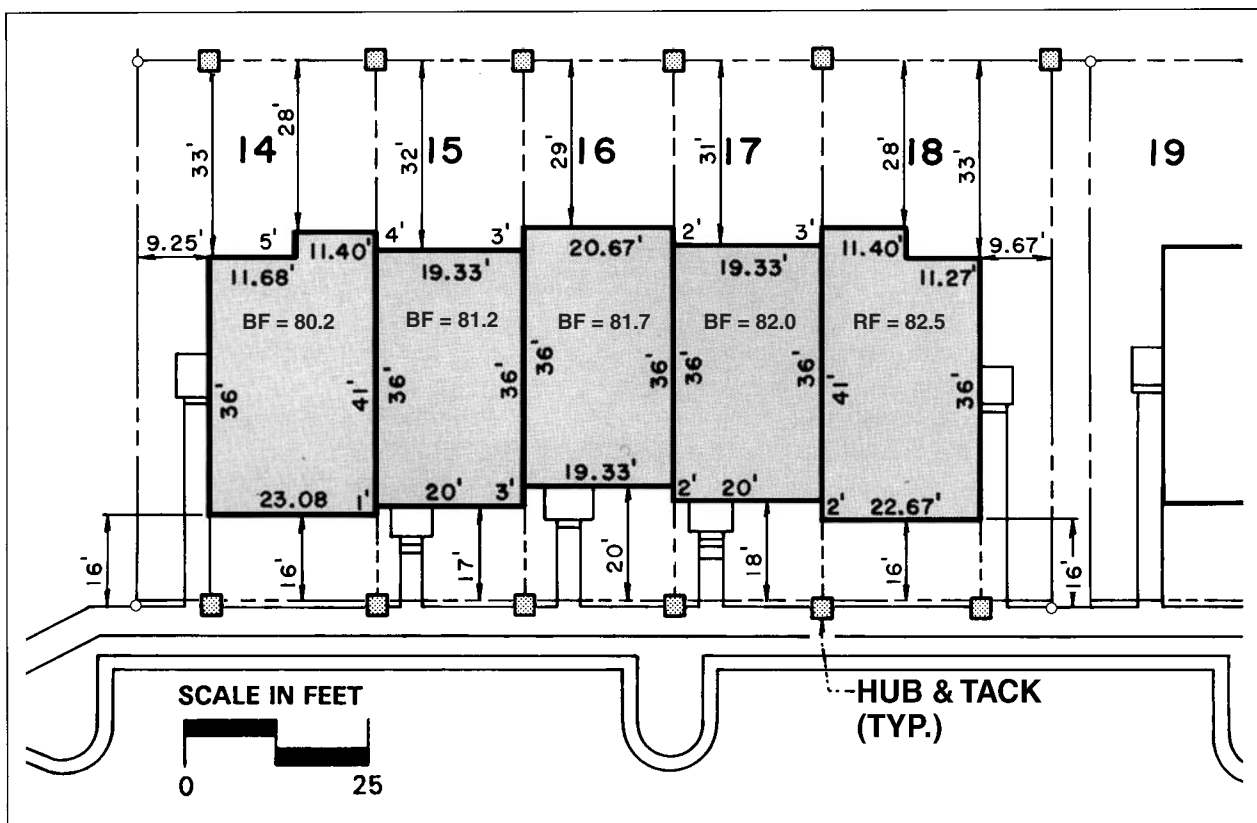


FIGURE 35.32 Typical townhouse layout for construction (BF = Basement Floor Elevation).

at the actual corners using the dimensions shown along the front and rear lines of each unit. Some contractors may request brick points on the centerline of walls. If so, the simplest method is to occupy each front offset, backsight the front offset line, turn 90°, and check to the corresponding rear offset. Then, using lot 14/15 on Figure 35.29 as an example, the distances measured from the front offset would be 16, 17, 53, and 57 ft.

**Commercial/Retail Buildings.** A large building may be controlled by column lines, baselines, or specified control points, depending upon the structure's configuration. The key to successful layout is to maintain high standards of accuracy and to reference the building-control network off site.

A great deal of preparation precedes the layout: traverse control has been set and computed, vertical control established and verified, and architectural plans scrutinized and checked. Ties from the traverse control to the building's control points are computed next, and a stakeout plan is drafted for the survey crew. The instrument has been checked and re-calibrated, if necessary, and the optical plummets on all tribrachs have been checked and adjusted.

On site, extra precautions are taken to maintain high standards of accuracy. The instrument should have good, solid set-ups. Angles should be double-centered.

Each time the instrument is fine-leveled, the backsight and the ensuing foresight should be achieved as quickly and efficiently as possible. The longer an instrument occupies a point, the more its chance of settling, thus producing error. Double-centering helps to eliminate error in the plates, and turning angles while the set-up is still fresh helps to eliminate angular drift. The backsight is taken and the foresight angle is turned. The head chainman sets a sight stake on that line and pencil-marks the top. The instrument operator then reloads the backsight angle 180 degrees different from the first sighting, and turns to the foresight line, now 180 degrees different. This may be done several times. The head chainman marks the new line on the top of the stake. There will probably be a discrepancy between the first and second (or subsequent) marks. This discrepancy is divided in half, and a tack is driven at the mean mark for a visible sight on the foresight line. The instrument operator monitors this sight as points are set.

Once the sightlines are established, the distances are measured for the building points to be set. Distances should be checked using a prism mounted on a tribrach and tripod. All the control lines should be extended and referenced off site, as any subsequent re-stake *must* match the initial stakeout. Appropriate checks for squareness should be performed—that is, turning 90° angles and measuring diagonal distances. Finally, all stakeout points and references should be located from traverse control, so that they can be checked in the office and filed for future reference.

Elevations are run using at least two benchmarks, and each stakeout hub is established as a temporary benchmark by using it as a turning point in the level circuit. On site, the hub elevations are written on the stakes. In the office,

the field notes and the elevations are verified and a formal cutsheet is issued.

**As-built Surveys.** An as-built survey is performed after construction of the project is complete and all inspections have been performed. The survey furnishes a municipality with evidence that a project has been constructed in accordance with the approved engineering plans. If the developer has posted a bond to insure compliance, the as-built plan is used as evidence to release the bond money. The format of the as-built survey depends on local requirements. Typically, the items surveyed include the public utilities, other structures and features intended to be maintained by the local municipality, and such items that show compliance of the project with zoning and subdivision ordinances.

In some instances, as-built surveys are a requirement for the contractor in his agreement with the project's owner. This verifies that the contractor built the project according to design and entitles him to payment. The as-built survey, also known as a record drawing, is prepared on the original plans. Appropriate information is shown in a specific manner to differentiate it from the design information already on the drawings. One method is to enclose the as-built survey information in boxes and to use larger and bold lettering to distinguish it from previous design information.

The sanitary sewer and the storm-sewer system typically are part of the as-built survey. The manholes and catch basins permit access for measuring invert elevations. (Other utilities, such as the waterline, where access is nearly impossible, are not included in the as-built survey.) For sanitary sewers and storm-sewer systems, the invert elevations are measured and the pipe sizes and lengths are verified. Pipe slopes are computed using this information and compared to the proposed design. When the as-built pipe slope is less than the proposed design slope, the hydraulic capacity is checked to insure hydraulic adequacy.

Sanitary sewer as-built information prepared for a municipality will generally include deed book and page references where easements are recorded, the top and invert elevations for all structures, the distance, pipe size, type of pipe, and pipe slope to all structures, and the bearing for each line. A storm sewer as-built provides the same detailed information. In addition, the as-built survey shows spot elevations around detention/retention pond areas, spillways and open drainage ditches and shows that riprap and velocity checks are in place. Occasionally, hydraulic tabulations for the retention/detention as-built information may be required.

Where public buildings are constructed, as-built surveys are used to check adherence to zoning requirements pertaining to building size and height, distance to property lines, and required parking. Site plan record drawings, in addition to sanitary and storm-sewer information, include the boundary bearings and distances, all easements with reference to where the easement is recorded, the square footage of the property and buildings, distances to the property lines, building dimensions, first-floor elevations and

building address numbers, fire-hydrant locations, number of parking spaces and dimensions, and any deviations from the original design in improvements to the site.

The as-built survey, if required by the local municipality, is then submitted for review and approval. Any discrepancies between the constructed project and original design are noted and then must be resolved by the contractor.

## SUMMARY

Construction stakeout requires substantial preparation before sending the survey crew to the field. Successful computing requires familiarity with construction techniques as well as good surveying skills. Inevitably there will be times when the survey crew will have to adjust the stakeout procedure in the field because of unforeseen jobsite conditions. Therefore, the field crew must take all of the plans, plats and hard copies and geometric coordinates with them in order to perform calculations necessary to complete the task. Contractors hate to have their workers and equipment stand idle. Delays caused by poorly prepared survey crews costs them time and money. The procedures for the various stakeout methods described above are intended as a suggestion of the ways to perform construction stakeouts. Many localities have regulations that dictate construction methods and preferences for performing construction stakeout because of local conditions and biases. In addition, many surveyors have adopted other ways to perform stakeout work because of the type of instruments (both field and office) at hand, local contractor's preferences, and field conditions, and because they have found what works best for them given all of the tangible and intangible factors. Before employing any of the foregoing methods, it is best to consult local ordinances and public agencies to avoid conflicts and to assure compliance with the prevailing regulations.

## REFERENCES

- American Congress on Surveying and Mapping, and American Society of Civil Engineers, *Definitions of Surveying and Associated Terms*, Revised 1978, American Congress on Surveying and Mapping, Bethesda, Md., 1989.
- Buckner, R.B., *Land Survey Review Manual*, 3rd ed., Ann Arbor Press, Ann Arbor, Mich., 2000.
- Cheves, Marc, "Making Data Collection Work," *P.O.B.—Point of Beginning*, 15:6 (August–September, 1990), pp. 54, 58, 60.
- Davis, Raymond, Francis Foote, and Joe Kelly, *Surveying: Theory and Practice*, 5th ed., McGraw-Hill, Inc., New York, 1968.
- Denny, Milton, "Noteworthy Field Notes," *P.O.B.—Point of Beginning*, 25:2, (November 1999), pp. 27–28.
- Dewberry & Davis, *Procedure & Checklist Manual*, In-house guidelines. Dewberry & Davis, Fairfax, Va., 1981.
- Dewberry & Davis Institute, *Party Chief Training Manual*, In-house training text. Dewberry & Davis, Fairfax, Va., 1984.
- Herubin, Charles, *Principles of Surveying*, 3rd ed., Reston Publishing Company, Inc., Reston, Va., 1982.
- Moffitt, Frances, and Harry Bouchard, *Surveying*, 7th ed., Intext Educational Publishers, New York, 1982.
- Munjy, Riadh, and others, "Total Station Survey System (TSSS) Software." *Surveying and Mapping. Journal of American Congress on Surveying and Mapping*, 49:4 (December 1989), pp. 31, 34–35.
- Pafford, F. William, *Handbook of Survey Notekeeping*, John Wiley and Son, Inc., New York, 1962.
- Root, Ray B., LLS, Personal Communication, May 1, 2001.
- Shrestha, Ramesh, "Formats and Specifications for an Electronic Field Book," *Surveying and Land Information Systems. Journal of American Congress on Surveying and Mapping*, 50:3 (September 1990), pp. 215–224.
- Virginia Association of Surveyors, Inc., *Standards of Practice for Professional Land Surveyors in the Commonwealth of Virginia*, 3rd ed., Richmond, Virginia, Virginia Association of Surveyors, Inc., 1993.

## CHAPTER 36

# BUILDING PERMITS AND CERTIFICATES OF OCCUPANCY

*Lawrence A. McDermott*

### INTRODUCTION

In order to insure that proper construction practices are followed, as well as to minimize responsibility for faulty construction, a municipality has specific processes that must be followed. Typically, these include site-plan review, building-permit application and an occupancy-application process. Site plans are reviewed to insure that the proposed project is designed in compliance with the local regulations. Municipalities require building permits in order to confirm that proposed buildings are structurally sound and conform to local and federal building codes. After construction is complete, municipalities require applications for occupancy permits before buildings can be occupied. This final stage allows the governing municipality to perform a final inspection of the structure to verify that the structure was built as required. The occupancy permit also represents one of the final stages in the construction process, after which the municipality is willing to release the pre-construction bonds.

### SITE PLANS, BUILDING PERMITS AND CERTIFICATES OF USE AND OCCUPANCY

#### Site Plans

In municipalities that have created a site-plan-review process for non-subdivisions, or as an additional step for certain types of subdivisions, the preparation and review of plans follows a procedure similar to the final subdivision plan. That is, plans of increasing detail and accuracy are prepared by the land development team and presented for review and approval to the coordinating agency. This agency distributes site plans to other agencies for comments and recommendations. The primary difference is that, in addi-

tion to external relationships and connections, the plan must show the internal relationship of buildings, the circulation system, parking, drainage, utilities and landscaping. Where elements of the project are controlled and constrained by zoning and other development regulations, the plan must demonstrate compliance with such requirements. Bonding of proposed public improvements may also be required.

In addition to the information shown on a subdivision plan submittal, the following information may be required on formal site-plan submissions:

- Interior lot lines
- Building foundation and footprint
- Lateral connections to underground utilities
- Parking, loading and circulation layout, showing calculations and construction details
- Fire lanes
- Landscaping plan and installation details
- Other statistical and graphic information needed to demonstrate compliance with ordinance requirements and public-facility manual standards

#### Building Permits

After subdivision or site plans are approved, and in situations where no such approval is required, building permits typically are required for the construction of all structures, whether residential or non-residential. The assumption made is that no new property lines are being created, and the primary focus of review is architectural and structural

design, and connection to existing streets and facilities. Building-permit review allows for examination of criteria contained in both the zoning ordinance and building codes applicable in the jurisdiction. For zoning, the primary concerns are proper setback from property lines, area of building coverage, density and floor area, parking and other required facilities that support or are accessory to the principal use of the property. Determining compliance with the building codes requires extensive analysis of construction details to insure structural integrity, as well as review of internal systems for heating, plumbing and electric service. Fire-protection systems and fire-code construction standards are also reviewed.

As with other approval procedures, the designated lead agency receives plans and distributes them to others. The review is usually conducted in a single phase, although various components of the structure may be reviewed by different departments and agencies. In addition to the payment of application fees, other fees may also be collected at the time of submittal. These include availability or connection charges for water and sewer service. Fee calculations may be based on floor area, number of fixtures, projected usage or other indicators. In some communities, impact fees also are collected at the time of permit application.

Most municipalities prepare pre-printed application forms and checklists showing the information and plans that are required for the building-permit submission. The complexity of documents grows with the complexity of both the project and type of construction. In addition, alternative procedures may be established to help coordinate and simplify project submission and reviews.

- For individual single-family-detached structures, simple lot grading, foundation and framing plans may be sufficient. Depending on the size of the structure, preparation by an architect may not be required.
- For subdivisions, the submission might include overlot grading plans showing connection to storm drain and other utility systems, master documents showing foundation and framing plans for each type of house plan to be constructed, structural connection details, soils information and soil test data showing areas requiring special treatment because of problem soils, and alternate details for nontypical situations. The municipality will normally perform inspections at the completion of each phase of construction prior to allowing work to be concealed: for example, closing in a wall before the electric and/or plumbing in the wall has been inspected/approved, in preparation for the next phase.

- For complex, non-residential structures, a complete set of construction drawings, showing all utility connections and internal systems is required. Local regulations often establish elaborate testing and inspection procedures with, in some instances, provisions for third-party inspection by independent firms.

### Occupancy Permits

An occupancy permit is the final legal authorization needed from the jurisdiction to establish a use or structure. Issued in conjunction with final inspection of construction activity, the occupancy permit signifies that the structure is complete, properly connected to utilities and in compliance with all codes, ordinances, and established policies. It also is an acknowledgement that the proposed use is allowed by and conforms to the zoning ordinance. It is at this stage that residents or businesses are allowed to occupy the premises.

Note that the public infrastructure for the development might not yet be completed, and the improvements may still be under bond. It is often local practice to delay completion of the streets until all construction equipment is taken off the site. A base pavement coat may be installed, with the final top coat installed as the last step prior to bond release. This approach minimizes the potential for damage caused by construction equipment or moving vans that the developer would otherwise be responsible for repairing. Depending on the time of year, landscaping improvements may also be delayed until the start of the growing season. Where landscaping is required and included as a bonded item, the jurisdiction may allow the execution of a substitute agreement or cash escrow deposit and release the developer's primary bond.

### REFERENCES

- David Jensen Associates, Inc., *How to Win at the Zoning Table*, National Association of Home Builders, Washington, D.C., 1984.
- Engineers and Surveyors Institute, *Improving Preparation, Review & Approval of Subdivision & Site Plans in Fairfax County*, Engineers and Surveyors Institute, Fairfax, Va., 1988.
- Institute for Participatory Management and Planning, *Citizen Participation Handbook*, Institute for Participatory Management and Planning, Laramie, Wyo., 1986.
- International City Management Association, *The Practice of Local Government Planning*, International City Management Association, Washington, D.C., 1979.
- National Association of Home Builders, *Subdivision Regulation Handbook*, National Association of Home Builders, Washington, D.C., 1978.
- Urban Land Institute, *Working with the Community: A Developer's Guide*, ULI-The Urban Land Institute, Washington, D.C., 1985.

## **PART VIII**

# TECHNICAL APPENDICES



## CHAPTER 37

# GEOSPATIAL DATA

David F. Maune, Ph.D., C.P.

“Spatial data deals with location, shapes and the relationships among features (topology). Descriptive data deals with the attributes or characteristics of the features. The combination of the two types of data is the key to creating intelligent maps.” (The Power of Maps exhibit, Smithsonian Institution)

### INTRODUCTION

*Geospatial data* is information that identifies the geographic location and characteristics of natural or constructed features and boundaries of earth. This information may be derived from, among other things, mapping, surveying, and remote sensing technologies. *Geospatial data* and *spatial data* are normally used interchangeably. However, *spatial data* could include the size, relative location, and/or characteristics of features without knowing their exact geographic location (e.g., generic building plans that are not georeferenced), whereas *geospatial data* is always georeferenced in terms of geographic coordinates (latitude and longitude) or some form of rectangular coordinates (normally State Plane or UTM coordinates).

Land development engineers and surveyors need to fully understand geospatial data, accuracy standards, modern geospatial technologies and capabilities/limitations thereof, as well as information that could affect the selection of subcontractors.

### TYPES OF GEOSPATIAL DATA

This section is important because failure to understand the different types of geospatial data could cause the wrong data to be requested, resulting in the subcontracting for data unsuitable for its intended purpose and/or additional expenses to correct misunderstandings.

### Raster Data and Vector Data

*Raster data* consists of georeferenced pixels or grid cells of uniform resolution. Digital images are raster data, as are maps or engineer drawings scanned at 500 or 1000 dots per inch (dpi), for example. A *pixel* (picture element) is the smallest indivisible element of a digital image. One pixel of a LANDSAT satellite multispectral image equates to a 30-m by 30-m square area on the ground; 1 pixel of a standard USGS Digital Orthophoto Quarter-Quad (DOQ) covers a 1-m by 1-m square area; and 1 pixel of a high-resolution digital orthophoto produced specifically for a land development project might cover a 6-in. by 6-in. area on the ground. Such images are referred to as having 30-m, 1-m, or 6-in. pixels respectively. This is also called the *image resolution*. The resolution of scanned documents is referred to in terms of dpi. Raster images can be displayed on the computer at different scales, to the point where “pixel breakdown” occurs and individual pixels are visible, but “zooming in” doesn’t improve the accuracy of such raster data.

Somewhat different from a pixel, a raster *grid cell* is one element of a matrix formed by horizontal and vertical lines at user-defined spacing (e.g., 10-m grid spacing). Raster grid cells often are used to display soil types, geology, vegetation classification, land use, land classification, and natural features that do not need to be depicted with smooth lines and curves. Raster data normally is displayed in colors representing differing image characteristics for each pixel or differing values for the attributes describing each grid cell.

*Vector data* consists of 2-D ( $x/y$ ) or 3-D ( $x/y/z$ ) coordinates defining the locations of point, line, and area features (related terms include *nodes*, *vertices*, *shape points*, *arcs*, *de-*



generate lines, line strings, line chains, edges, polygons, and other terms that each have their own definitions). Vector data may be displayed on a computer with different colors, line styles (solid, dashed, dotted, etc.), line weights (thicknesses), and symbols. Vector lines and curves are normally smooth, while grid cell raster data has a “stair step” appearance, especially when zoomed-in to view the raster data at a large scale. CADD drawings are a form of vector data, as are survey data with points having 2-D or 3-D coordinates, and lines having distances and angles.

*Merged raster/vector data* are now common with modern geographic information systems (GIS). Raster images are more understandable to humans, but attributed vector data is more intelligible to computers. The merger of raster and vector data (normally the overlay of vector data on top of raster images) allows the best of both worlds. The raster data normally serves as a base map for overlay of vector GIS data.

### Descriptive Data and Map Annotations

*Descriptive data* more fully describes the geospatial data. Attributes describe the various point, line, and area features, such as length, diameter, manufacturer, model, serial number, material composition, burial depth, or installation date of a utility feature. Often, feature-attribute codes are used to describe diverse feature types. Alpha-numeric textual data or attributes are often stored in relational databases such as Oracle, Informix, or dBase, and selected items from the database may be displayed on maps. Descriptive databases can be managed separately from geospatial data; for example, a tax assessor's database may be linked to a county's parcel maps but maintained separately and confidentially from spatial databases available to the general public.

*Annotations* are map symbols or alpha-numeric labels, such as route numbers, elevations, or names of towns, streets, rivers, or mountains, placed on maps. On digital maps, annotations are normally key-entered, then stored with coordinates for the beginning, center, or ending of the annotation and/or with other rules for placement and orientation, text fonts, and special characters and symbols of various sizes.

### Planimetric Maps and Topographic Maps

*Planimetric maps* display the horizontal positions of natural and man-made features and boundary lines, and *planimetric data* is displayed in two dimensions only. If maps do not display elevation data, with contour lines or an alternative method, they are planimetric maps. In a computer, planimetric data is treated as 2-D files having  $x,y$  coordinates. Digital orthophotos are a form of planimetric map where no elevation data is presented. Planimetric maps and planimetric data normally form the base maps for overlay of other data required for a land development project.

When someone refers to “planimetrics”, he or she normally means planimetric data that can be seen and mapped horizontally from stereo aerial photography, including hydrographic features (e.g., river and shore lines), transportation features (e.g., road/highway edge of pavement, railroad tracks, airport runways and taxiways), man-made features (e.g., building footprints, bridges, transmission lines) and other features that can be seen from aerial photography. The term “planimetrics” does not normally include boundary lines and underground utilities that cannot be mapped with stereo aerial photography, even though they can be surveyed and georeferenced on planimetric maps. Similarly, a “plan view” shows the horizontal location of features as though looking straight down from infinity.

*Topographic maps* display both the horizontal positions of natural and man-made features and boundary lines as well as elevation data (normally contour lines). In a computer, *topographic data* may be 3-D files having  $x,y,z$  coordinates for individual points or line strings, or they may be different forms of 2-D files with at least one file including contour lines with elevation attributes.

Both planimetric data and topographic data are produced by surveying, photogrammetric mapping and/or remote sensing. Topographic data is also produced by new technologies, especially Light Detection and Ranging (LIDAR). These technologies are discussed below.

Topographic data and topographic surveys normally end at the edge of water bodies, but they often include the elevation of a lake.

### Digital Elevation Models (DEMs)

DEMs have at least three different meanings to different users. For some, DEM is a generic term for digital topographic and/or bathymetric data in all its various forms. For the United States Geological Survey (USGS), a DEM is a standard form of elevation dataset at regularly spaced intervals in  $x$  and  $y$  directions (normally 10 or 30-m spacing), using  $z$ -values georeferenced with UTM coordinates. For others in the United States, a DEM has  $z$ -values at regularly-spaced intervals in  $x$  and  $y$  directions, but following alternative specifications, perhaps with narrower grid spacing and State Plane coordinates, for example. DEMs always imply elevations of the terrain (bare earth  $z$ -values) void of vegetation and man-made features, as opposed to Digital Surface Models (DSMs) that include the elevations of tree-tops, rooftops, towers, and other features elevated above the terrain. In Europe, DEMs are considered to be synonymous with Digital Terrain Models (DTMs); but in the United States, DTMs may include irregularly-spaced points, Triangulated Irregular Network (TIN) data, or DEMs supplemented with breaklines where the slope changes.

For more information on this subject, see ASPRS, 2001 published by the American Society for Photogrammetry and Remote Sensing (ASPRS); it is entitled: “Digital Elevation Model Technologies and Applications: the DEM Users Man-

ual.” Also, procedures on photogrammetric production of DEMs is included below.

### Contours

Contours are lines of equal elevation. We all know them and generally love them. They are best produced by stereo photogrammetric compilation where the operator can see the breaklines (where the slope changes) and can manually shape the contour lines so that they are aesthetically pleasing. When manually compiling photogrammetrically, the compiler also shapes contours (according to established rules) where they cross streams and roads, adjoin lakes and rivers, etc.

When contours are generated automatically from DEMs, they are not as aesthetically pleasing. A DEM has no way of knowing where a breakline exists between DEM points and, therefore, can't automatically shape the contours to depict streams, roads, retaining walls, etc. correctly. High-resolution imagery can assist when using DEMs to develop contours, but it is best to generate contours photogrammetrically in the first place. It's a much simpler process to convert from contours to DEMs than it is to convert from DEMs to contours.

### Digital Orthophotos

A digital orthophoto is a digital image from a perspective aerial photograph, corrected by photogrammetric differential rectification so that image displacements caused by camera tilt and terrain relief (changes in elevation) are removed. This converts the image from a perspective projection into an orthographic projection (as though looking down on all points from straight above). The digital orthophoto is created by scanning an aerial photograph diapositive transparency with a precision-image scanner. The scanned data file is then digitally rectified to an orthographic projection by processing each image pixel through photogrammetric space-resection equations. This process requires, as input, ground control points acquired from ground surveys, camera calibration parameters, camera exterior-orientation parameters from aerotriangulation, and a DEM. Because they are rectified differentially with DEMs, only ground features shown on digital orthophotos are accurate positionally; elevated features (e.g., rooftops, towers) are normally displaced outward from the center of the photograph from which they are produced. Digital orthophotos are very popular as community base maps because of their relative low cost, making it affordable to revise orthophotos periodically to keep base maps up-to-date. More information on digital orthophotos is included below.

### HORIZONTAL AND VERTICAL DATUMS

Survey data referenced to the wrong datum can have catastrophic results on a land development project, especially if new buildings are constructed below the base flood elevation (BFE) that should have been built higher than the

BFE, or if buildings are constructed on the wrong side of a property line. Understanding horizontal datums requires an understanding of *ellipsoids*, and understanding vertical datums requires an understanding also of the *geoid*. Many land development projects go astray annually in the U.S. because of confusion regarding these topics, discussed in this section.

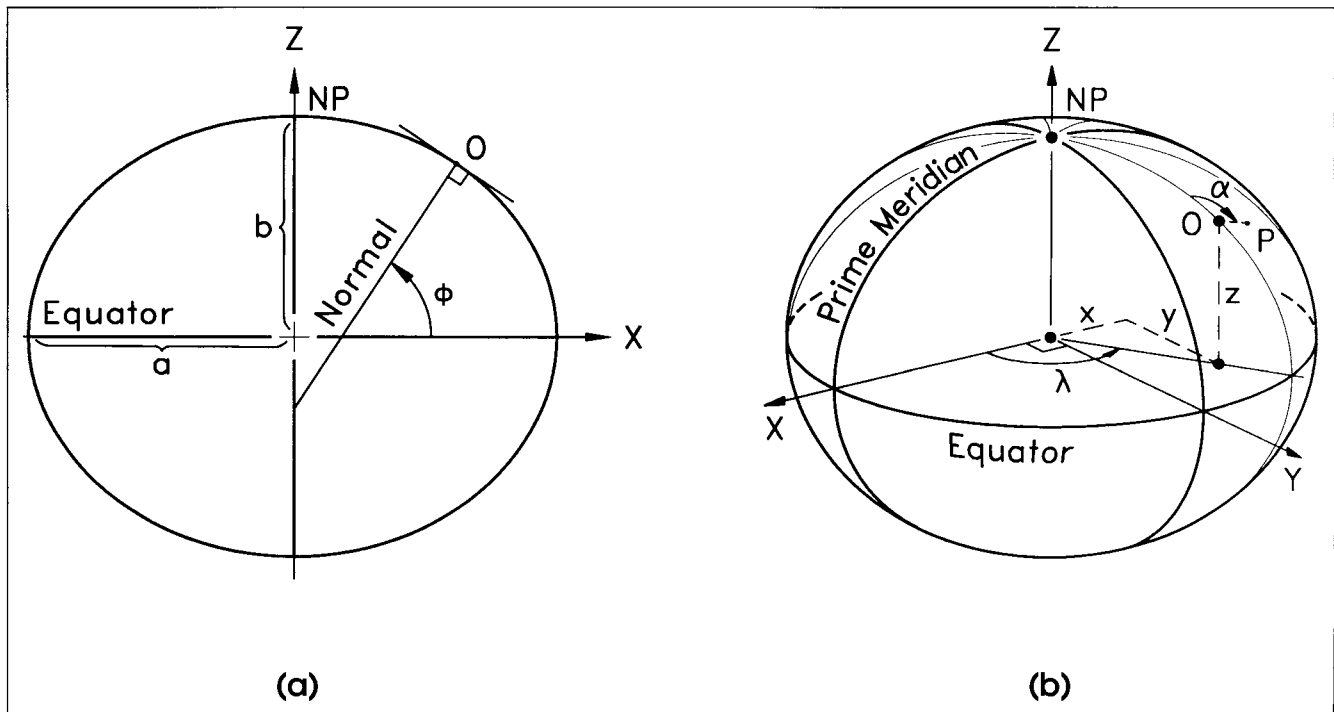
### Ellipsoids

Precise survey calculations are based on a mathematical surface called an ellipsoid—an ellipse of revolution, rotated around the (shorter) polar axis, which approximates the size and shape of the earth. As shown at Figure 37.1, these ellipsoids have a semi-major axis “*a*” (distance from the ellipsoid's center to the surface along the equatorial plane) and a semi-minor axis “*b*” (distance from the ellipsoid's center to the poles). From “*a*” and “*b*,” the flattening “*f*” =  $(a - b)/a$  can be calculated. For centuries, different ellipsoids were used around the world, chosen to best fit the terrain in different geographic areas. Until the 1970s, the U.S. used the Clarke 1866 ellipsoid that was not earth-centered (ellipsoid's center coinciding with the center of the earth). Other countries used different ellipsoids that better fit their local topography (e.g., International, Bessel, and Clarke 1880 ellipsoids used in Europe, Asia, and Africa), but none of those ellipsoids were earth-centered.

With the advent of earth-orbiting satellites in the 1960s and 1970s, the United States was able to develop earth-centered ellipsoids, optimized to fit the entire world and not just individual countries in isolation. One of the first was the World Geodetic System of 1972 (WGS 72), then the Geodetic Reference System of 1980 (GRS 80), and finally the World Geodetic System of 1984 (WGS 84), which is slightly different from GRS 80 but can be considered to be identical for all practical purposes. The defining parameters of the ellipsoids mentioned above are listed in Table 37.1. These parameters are embedded in most standard CADD and GIS software used in the United States today.

### Datums

A datum can be thought of as the origin or basis to which all survey measurements are referenced. Horizontal datums, used for horizontal coordinates, are based on ellipsoids on which mathematical survey calculations can be performed; but this doesn't just require definitions of the size of the ellipsoid, but the origin and orientation of that ellipsoid relative to the equator, poles, and the astronomic universe. Vertical datums, used for elevations, are based on some form of mean sea level (or where mean sea level would project under the land's surface coinciding with changes in gravity). The problem with datums is that there are too many of them. For example, some communities have over a dozen different vertical datums, several without known translation formulas into the United States standard vertical datum.



**FIGURE 37.1** There are two common ways to express the three-dimensional coordinates of a point on the Earth: (a) Orthogonal  $x, y$  coordinates (e.g., State Plane or UTM coordinates), and  $z$  values (elevations); and (b) geographic coordinates, i.e., latitude ( $\phi$ ), longitude ( $\lambda$ ), and elevation (not shown). Figure 37.1-a shows the semi-major axis ( $a$ ) and semi-minor axis ( $b$ ), which define an ellipsoid, as well as the geodetic latitude ( $\phi$ ) of a hypothetical point  $O$ . Figure 37.1-b shows the geodetic longitude ( $\lambda$ ) of point  $O$  and the azimuth angle ( $\alpha$ ) to a fixed point  $P$ .

Horizontally, United States mapping agencies such as the U.S. Geological Survey (USGS) and the U.S. Army Corps of Engineers previously used the Clarke 1866 ellipsoid with a datum point origin at Meades Ranch in Kansas for the North American Datum of 1927 (NAD27); but they now use the GRS 80 ellipsoid with the earth center of mass as the datum point origin for the North American Datum of 1983 (NAD 83). NAD 83 was completed by the National

Geodetic Survey (NGS) in July 1986 and was adopted as the official horizontal datum of the United States in June 1989 and Canada in May 1990. Failure to convert correctly between NAD 27 and NAD 83 coordinates can cause horizontal errors of hundreds of meters.

Vertically, there are also differences between mean sea level (MSL), the National Geodetic Vertical Datum of 1929 (NGVD 29), and the North American Vertical Datum of

**TABLE 37.1 Common Ellipsoids**

ELLIPSOID	SEMI-MAJOR AXIS (A)	SEMI-MINOR AXIS (B)	FLATTENING INVERSE (1/f)
International	6,378,388 meters	6,356,911.9462 meters	437
Bessel 1841	6,377,397.155 meters	6,356,078.9643 meters	439.15281
Clarke 1866	6,378,206.4 meters	6,356,583.8 meters	434.97869
Clarke 1880	6,378,249.145 meters	6,356,514.8696 meters	433.465
WGS 72	6,378,135 meters	6,356,750.5 meters	438.26
GRS 80	6,378,137 meters	6,356,752.3141 meters	438.25722
WGS 84	6,378,137 meters	6,356,752.3142 meters	438.25722

1988 (NAVD 88). Local mean sea level (LMSL) is the result of hourly values of still water levels observed over a tidal epoch of 19 years. The arithmetic mean of the observed levels at each station is taken as LMSL for each station included in the observations. LMSL can be used for a vertical datum of reference that is based on the observation of one or more nearby tidal gauging stations. NGVD 29 was established based on the assumption that the height of local mean sea level at 21 tidal stations in the United States and five tidal stations in Canada equaled 0.0000 foot on the NGVD. However, local mean sea level at these 26 tidal stations actually represents 26 different elevations, and not zero elevation, because topography, ocean currents, tides, winds, and other factors cause actual elevations to be different at these 26 tidal stations. Imagine if Panama used the same logic and said that mean sea level on the Pacific end of the Panama Canal represented the same (zero) elevation as mean sea level on the Caribbean end of that canal; major elevation discrepancies would undoubtedly exist throughout that country.

The United States is fortunate that this basic erroneous assumption for NGVD 29 causes elevation discrepancies of no more than 1.5 m anywhere in the United States. But the magnitude of the problem wasn't understood until satellite technology enabled surveyors to think and survey globally in order to determine the true size and shape of the earth.

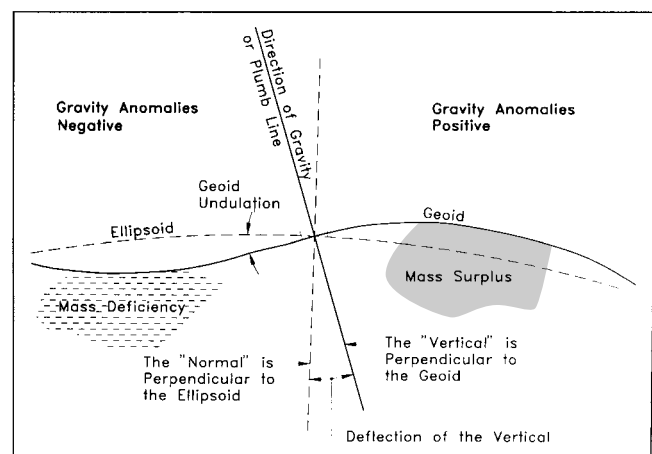
NAVD 88, officially implemented by a *Federal Register* announcement on June 21, 1993, is based on the mass or density of the earth instead of the varying values of the heights of the seas. Measurements of the acceleration of gravity are made at observation points in the network being used, and only one point is defined as the datum point. The vertical-reference surface, the *geoid*, is then defined by the surface on which the gravity potential values are equal to the datum point value; this equipotential surface is called the *geoid*. The National Geodetic Survey completed the analyses of measurements and the adjustments of the resulting data. The datum point for NAVD 88 was defined using the reference station at Pointe-au-Pere (Father Point/Rimousk) on the St. Lawrence River, Province of Quebec, Canada. Failure to convert elevations correctly between NGVD 29 and NAVD 88 could cause elevation discrepancies up to 1.5 m.

The National Geodetic Survey (NGS) provides NADCON software for conversions between NAD 27 and NAD 83 and VERTCON software for conversions between NGVD 29 and NAVD 88 (see the NGS home page at [www.ngs.noaa.gov](http://www.ngs.noaa.gov), NGS Geodetic Tool Kit, to download PC software). Similarly, the U.S. Army Corps of Engineers provides free CORPSCON software for this purpose, based on NADCON and VERTCON (see the U.S. Army Topographic Engineering Center home page at [www.tec.army.mil](http://www.tec.army.mil); then What We Do; Products and Services; Software Available; and CORPSCON. Numerous Corps of Engineers Engineer Manuals can also be downloaded from <http://>

[crunch.tec.army.mil](http://crunch.tec.army.mil); under Surveying Engineering and Mapping Center of Expertise; Publications).

## Geoid

Traditional land survey techniques utilize instruments that are "leveled" to establish a horizontal line of sight that is perpendicular to the local direction of gravity. Astronomic observations for determining latitude, longitude and azimuths, as well as traditional land surveying techniques (leveling, triangulation, trilateration, or traverse), measure angles, distances or elevations relative to plumb lines called "verticals," which normally do *not* point to the center of the earth. Gravity anomalies, resulting from variations in the mass of the earth, cause the direction of gravity to shift slightly towards the direction of mass excesses (e.g., mountains) and away from the direction of mass deficiencies (e.g., oceans). The geoid (an equipotential surface where the gravity potential is equal to that at the vertical datum origin) undulates smoothly up and down, causing a *geoid height* or *geoid undulation* above or below the ellipsoid. Elevations (orthometric heights) from conventional surveys are referenced to this geoidal surface, whereas elevations from GPS surveys are actually "ellipsoid heights" or elevations above or below the WGS 84 ellipsoid (see Figure 37.2). When merging GPS data with conventional survey data, it is imperative that the survey specialist use the latest NGS geoid model to convert ellipsoid heights (from GPS observations) into orthometric heights (equivalent to conventional surveys). Failure to do so can cause elevation discrepancies of 30 mm or more. During the past decade, NGS



**FIGURE 37.2** The Earth's mass anomalies cause variations in the direction of gravity. The geoid is a smooth but undulating surface that always has the direction of gravity perpendicular to it. The vertical, which follows the plumb line or direction of gravity for any given point, is perpendicular to the geoid at that point and usually does not pass through the Earth's center of mass. The normal is perpendicular to the ellipsoid and usually does not pass through the center of the ellipsoid. The deflection of the vertical is the angle between the normal and the vertical. The geoid undulation is the distance, taken along a perpendicular to the ellipsoid, from that ellipsoid to the geoid.

has published revised geoid models on a 3-yr cycle (i.e., Geoid 90, Geoid 93, Geoid 96, and Geoid 99).

## NATIONAL SPATIAL DATA INFRASTRUCTURE (NSDI)

### Framework Data

The National Spatial Data Infrastructure (NSDI) is defined as the technologies, policies, and people necessary to promote sharing of geospatial data throughout all levels of government, the private and non-profit sectors, and the academic community. The NSDI *framework* is a collaborative effort to create a widely-available source of basic geographic data. It provides the most common data themes geographic data users need, as well as an environment to support the development and use of these data. The framework represents “data you can trust”—the best available data for an area, certified, standardized, and described according to a common standard.

The NSDI *framework* provides a foundation on which organizations can build by adding their own detail and compiling other data sets. Its key aspects are: (1) seven themes of digital geographic data that are commonly used, including geodetic control, orthoimagery, elevation, transportation, hydrography, governmental units, and cadastral information; (2) procedures, technology, and guidelines that provide for integration, sharing, and use of these data; (3) institutional relationships and business practices that encourage the maintenance and use of the data.

### National Spatial Reference System (NSRS)

The National Spatial Reference System (NSRS) is a consistent coordinate system that defines latitude, longitude, height, scale, gravity, and orientation throughout the United States, and how these values change with time. The NSRS comprises the following:

1. The National CORS, a set of global-positioning-system, continuously-operating reference stations meeting the highest standards for defining the geodetic datum
2. A network of permanent survey monuments including the Federal Base Network (FBN), Cooperative Base Network (CBN), and User Densification Network (UDN)
3. A consistent, accurate, and up-to-date national shoreline
4. A set of accurate models describing dynamic geophysical processes that affect spatial measurements

The NSRS provides a highly accurate, precise, consistent geographic framework throughout the United States. It is the foundation for the NSDI, a critical component of the information superhighway, and a significant national resource. It includes hundreds of thousands of geodetic control monuments and benchmarks that have been surveyed

relative to our horizontal and vertical datums (i.e., NAD 83 and NAVD 88). As per the NSDI framework motto, the NSRS represents survey control “data you can trust”—the best available data for any area within the United States. It’s accurate, it’s readily available on the web, it’s simple to use, and it’s free. NSRS data should be used as the foundation for any land development survey project.

To locate the most suitable NSRS survey monuments for any project area, visit the NGS home page at [www.ngs.noaa.gov](http://www.ngs.noaa.gov); then click on Data Sheets. Under “Search the NGS DataBase and Retrieve Data Sheets,” one can select either “Radial Search” or “Rectangular Search.” Under “Radial Search,” enter the latitude and longitude of the center of the land development project area or the estimated geographic coordinates of the local-network control point to be surveyed, plus the circular search radius in miles, up to 20 miles maximum. Alternatively, under “Rectangular Search,” enter the Min-Max values for the latitude and longitude of the rectangle surrounding the project site or the local-network control point to be surveyed. NGS data sheets will be listed for all NSRS monuments within the search area, so that the best monuments can be selected for extension of subsequent control. If more than 100 NSRS control points exist within the specified area, the user will need to subdivide the rectangular search area or narrow the radial search area in order to fit below the 100 maximum data sheets that can be printed from the web site.

Last, the user will be asked to select survey control points in one of three categories:

- Any horizontal and/or vertical control point (surveyed by any method—conventional or GPS)
- Any horizontal control point only (surveyed by any method—conventional or GPS)
- GPS sites only

## FEDERAL GEOGRAPHIC DATA COMMITTEE (FGDC) GEOSPATIAL POSITIONING ACCURACY STANDARDS

### Replacement for National Map Accuracy Standard

For a half century, nearly all maps were produced in accordance with the National Map Accuracy Standards (NMAS) published in 1947. But the NMAS was replaced in 1998 by new FGDC Geospatial Positioning Accuracy Standards for digital geospatial data. It is important for land development surveyors and engineers to understand the differences between the new and old standards. This subsection will first summarize the old standards before comparing them with the new standards.

According to the NMAS (Bureau of the Budget, 1947), horizontal and vertical accuracy of maps were defined as follows:

- For maps produced at a scale of 1:20,000 and larger (which includes most land development projects), “not more than 10 percent of the points tested

shall be in error by more than  $\frac{1}{30}$  inch, measured on the publication scale . . . These limits of accuracy shall apply in all cases to positions of well defined points only. 'Well defined' points are those that are easily visible or recoverable on the ground, such as the following: monuments or markers, such as bench marks, property boundary monuments, intersections of roads, railroads, etc.; corners of large buildings or structures (or center points of small buildings), etc. In general what is 'well defined' will also be determined by what is plottable on the scale of the map within  $\frac{1}{100}$  inch." At map scale, the Circular Map Accuracy Standard (CMAS) equals  $\frac{1}{30}$ th of an inch; at ground scale, the CMAS equals  $\frac{1}{30}$ th of an inch divided by the map scale. For example, DOQs compiled at 1 in = 1,000 ft have a CMAS of  $\frac{1}{30}$ th of an inch on the map or 33.3 ft on the ground.

- "Vertical accuracy, as applied to contour maps on all publication scales, shall be such that not more than 10 percent of the elevations tested shall be in error more than one-half the contour interval. In checking elevations taken from the map, the apparent vertical error may be decreased by assuming a horizontal displacement within the permissible horizontal error for a map of that scale." The Vertical Map Accuracy Standard (VMAS) equals one-half the contour interval on the map, or 2.5 ft in ground elevation if the contour interval is 5 ft, for example.

Both the CMAS and the VMAS are standards at the 90% confidence level, but these VMAS errors do not necessarily follow a normal distribution—that is, there are no limits to the size of the errors that exceed the CMAS or VMAS.

With digital data, the geospatial-data community recognized that these standards were inappropriate since the scale and contour interval of a digital map could be changed with the click of a computer mouse, without changing the underlying accuracy of the data. Just because one can easily "zoom in" on GIS data and view the data at high resolution does not mean that the data suddenly gets more accurate.

In 1990, the American Society for Photogrammetry and Remote Sensing published the ASPRS Standards for Large Scale Maps (ASPRS, 1990). These "ASPRS 90" standards were modified subsequently by the FGDC in the FGDC Geospatial Positioning Accuracy Standards, published in 1998, which officially replaced the VMAS for digital spatial data. These new standards have three parts:

- Part 1: Reporting Methodology (FGDC, 1998a)
- Part 2: Standards for Geodetic Networks (FGDC, 1998b)
- Part 3: National Standard for Spatial Data Accuracy (NSSDA)(FGDC, 1998c)

## Part 1: Reporting Methodology

Reference FGDC-STD-007.1-1998 at [www.fgdc.gov/standards/documents/standards/chapter1.pdf](http://www.fgdc.gov/standards/documents/standards/chapter1.pdf).

Part 1 (FGDC, 1998a) provides a common methodology for reporting the accuracy of horizontal and/or vertical coordinate values for clearly-defined features where the location is represented by a single-point coordinate, either 2-D or 3-D. Examples are: survey monuments; prominent landmarks such as church spires, standpipes, radio towers, tall chimneys, and mountain peaks; and targeted photogrammetric control points. It provides the means to compare directly the accuracy of coordinate values obtained by one method (e.g., a cartographically-derived value) with that obtained by another method (e.g., a GPS geodetic network survey) for the same point. This helps the users to know not only the coordinate values but also the accuracy of those coordinate values, so they can decide which coordinate values represent the best estimate of the true value for their applications.

According to FGDC, 1998a, horizontal and vertical accuracy are to be reported in ground distances at the 95% confidence level, as follows:

- Horizontal: "The reporting standard in the horizontal component is the radius of a circle of uncertainty, such that the true or theoretical location of the point falls within that circle 95-percent of the time." Part 3 refers to this horizontal accuracy component as "Accuracy<sub>x</sub>."

- Vertical: "The reporting standard in the vertical component is a linear uncertainty value, such that the true or theoretical location of the point falls within  $\pm$  of that linear uncertainty value 95-percent of the time." Part 3 refers to this vertical accuracy component as "Accuracy<sub>z</sub>."

- Furthermore, accuracy is to be reported in terms of *local accuracy* or *network accuracy*.

- "The *local accuracy* of a control point is a value that represents the uncertainty in the coordinates of the control point relative to the coordinates of other directly connected, adjacent control points at the 95-percent confidence level. The reported local accuracy is an approximate average of the individual accuracy values between this control point and other observed control points used to establish the coordinates of the control point." (For practical purposes, *local accuracy* can be considered the equivalent of *relative accuracy*, defined in other FGDC standards as "point-to-point accuracy.")

- "The *network accuracy* of a control point is a value that represents the uncertainty in the coordinates of the control point with respect to the geodetic datum at the 95-percent confidence level. For NSRS network accuracy classification, the datum is considered to be

best expressed by the geodetic values at the Continuously Operating Reference Stations (CORS) supported by NGS. By this definition, the local and network accuracy values at CORS sites are considered to be infinitesimal, i.e., to approach zero.” (For practical purposes, *network accuracy* can be considered the equivalent of *absolute accuracy*, defined in other FGDC standards as “accuracy stated with respect to a defined datum or reference system.”)

**Part 2: Standards for Geodetic Networks**

Reference FGDC-STD-007.2-1998 at [www.fgdc.gov/standards/documents/standards/chapter2.pdf](http://www.fgdc.gov/standards/documents/standards/chapter2.pdf).

Part 2 (FGDC, 1998b) provides a common methodology for determining and reporting the accuracy of horizontal and vertical coordinate values for geodetic control points represented by survey monuments such as brass disks and rod marks. It provides a means to compare directly the accuracy of coordinate values obtained by one method (e.g., a classical line-of-sight traverse) with the accuracy of coordinate values obtained by another method (e.g., a GPS geodetic network survey) for the same point.

According to FGDC, 1998b, geodetic control surveys are usually performed to establish a basic control network (framework) from which supplemental surveying and mapping work is performed (e.g., for land development). Geodetic network surveys are distinguished by use of redundant, interconnected, permanently-monumented control points that comprise the framework for the NSRS or are often incorporated into the NSRS. Such surveys must be performed to far more rigorous accuracy and quality assurance standards than those for control surveys for general engineering or construction. However, geodetic control surveys are required for: controlling interstate transportation corridors (e.g., highways, pipelines, and railroads); long-span bridge-construction alignment; geophysical studies; and structural-deformation monitoring of dams, buildings, and similar facilities.

The NSRS supports both local accuracy and network accuracy. Local accuracy is best adapted to check relations between nearby control points; for example, a surveyor checking closure between two NSRS points is mostly interested in a local accuracy measure. On the other hand, someone constructing a GIS or LIS will often need some type of positional tolerance associated with a set of coordinates. Network accuracy measures how well coordinates approach the error-free datum. To establish the network accuracy of a control point, it is not necessary to connect directly to a CORS; however, it is necessary that the survey be connected properly to existing NSRS control points with established network accuracy values.

Table 37.2 lists current FGDC accuracy standards. Such accuracies are reported separately for horizontal coordinates, ellipsoid heights, and orthometric heights (elevations).

**TABLE 37.2 FGDC Accuracy Standards**

ACCURACY CLASSIFICATION	≤ M AT 95% CONFIDENCE LEVEL
1-Millimeter	0.001 meters
2-Millimeter	0.002 meters
5-Millimeter	0.005 meters
.....	
1-Centimeter	0.01 meters
2-Centimeter	0.02 meters
5-Centimeter	0.05 meters
.....	
1-Decimeter	0.1 meter
2-Decimeter	0.2 meter
5-Decimeter	0.5 meter
.....	
1-Meter	1.0 meter
2-Meter	2.0 meters
5-Meter	5.0 meters

**Part 3: National Standard for Spatial Data Accuracy (NSSDA)**

Reference FGDC-STD-007.3-1998 at [www.fgdc.gov/standards/documents/standards/chapter3.pdf](http://www.fgdc.gov/standards/documents/standards/chapter3.pdf).

The National Standard for Spatial Data Accuracy (NSSDA) (FGDC, 1998c) implements a statistical and testing methodology for estimating the positional accuracy of points on maps and in digital geospatial data, with respect to georeferenced ground positions of higher accuracy. The NSSDA applies to fully georeferenced maps and digital geospatial data, in either raster, point or vector format, derived from sources such as aerial photographs, satellite imagery, and ground surveys. Contrary to the National Map Accuracy Standard, which is now obsolete, the NSSDA does not define threshold accuracy values. Producing agencies and users must identify acceptable accuracies for their applications. Data and map producers must determine what accuracy exists or is achievable for their data and report it according to the NSSDA.

According to FGDC, 1998c, “Accuracy is reported in ground distances at the 95% confidence level. Accuracy reported at the 95% confidence level means that 95% of the positions in the dataset will have an error with respect to true ground position that is equal to or smaller than the reported accuracy value. The reported accuracy value reflects all uncertainties, including those introduced by geodetic control coordinates, compilation, and final computation of ground coordinate values in the product.”

The NSSDA uses root-mean-square error (RMSE) to estimate positional accuracy. RMSE is the square root of the average of the set of squared differences between dataset coordinate values and coordinate values from an independent source of higher accuracy for identical points.

Well-established procedures (e.g., field surveys or photogrammetry with fully-analytical aerial triangulation) report accuracy as follows: “Compiled to meet \_\_\_ (meters, feet) horizontal accuracy at 95% confidence level; \_\_\_ (meters, feet) vertical accuracy at 95% confidence level.”

New or questionable procedures (e.g., LIDAR or photogrammetry without fully-analytical aerial triangulation) report accuracy as follows: “Tested \_\_\_ (meters, feet) horizontal accuracy at 95% confidence level; \_\_\_ (meters, feet) vertical accuracy at 95% confidence level.”

When determined to be required, accuracy testing by an independent source of higher accuracy is the preferred test for positional accuracy. Horizontal accuracy should be tested by comparing the planimetric coordinates of well-defined points in the dataset with coordinates of the same points from an independent source of higher accuracy. A well-defined point represents a feature for which the horizontal position is known to a high degree of accuracy and position with respect to the geodetic datum; for the purpose of accuracy testing, well-defined points must be easily visible or recoverable on the ground, on the independent source of higher accuracy, and on the mapping or GIS product itself. Vertical accuracy should be tested by comparing the elevations in the dataset with elevations of the same points as determined from an independent source of higher accuracy; graphic-contour data and digital-topographic data may not contain well-defined points; therefore, the accuracy of a DEM is computed by a comparison of linearly-interpolated elevations in the DEM with corresponding known elevations of check points of higher accuracy.

Normally, a minimum of 20 check points should be tested; then, the 95% confidence level allows one point to fail the threshold given in product specifications.

For horizontal accuracy testing, the radial RMSE ( $RMSE_r$ ) is determined; then  $Accuracy_r = 1.7308 \times RMSE_r$ , where  $RMSE_r = \sqrt{RMSE_x^2 + RMSE_y^2}$ .

If horizontal errors are believed to have a normal distribution, a conversion can be made between the NMAS's Circular Map Accuracy Standard (CMAS) and NSSDA's  $RMSE_r$  and  $Accuracy_r$  as follows:

$$CMAS = 1.5175 \times RMSE_r$$

$$\text{or } Accuracy_r = 1.1406 \times CMAS$$

Table 37.3 provides examples of equivalent horizontal accuracy standards, comparing the NMAS accuracy thresholds (90% confidence level) for specified map scales with comparable accuracy reported per NSSDA terminology in terms of the  $RMSE_r$  and the  $Accuracy_r$  (95% confidence level).

For vertical accuracy testing, the vertical RMSE ( $RMSE_z$ ) is determined; then  $Accuracy_z = 1.9600 \times RMSE_z$ . (At [www.fgdc.gov/standards/documents/standards/elevation/elev\\_199.pdf](http://www.fgdc.gov/standards/documents/standards/elevation/elev_199.pdf), FGDC, 1999, the  $RMSE_z$  of DEMs is computed by a comparison of linearly-interpolated elevations in the DEM with corresponding known elevations.)

If elevation errors are believed to have a normal distribution, a conversion can be made between the NMAS's Vertical Map Accuracy Standard (VMAS) and NSSDA's  $RMSE_z$  and  $Accuracy_z$  as follows:

$$VMAS = 1.6449 \times RMSE_z$$

$$\text{or } Accuracy_z = 1.1916 \times VMAS$$

$$\text{or } \text{Contour Interval} = 3.2898 \times RMSE_z$$

Table 37.4 provides examples of equivalent vertical accuracy standards, comparing the NMAS accuracy thresholds (90% confidence level) for specified contour intervals with comparable accuracy reported per NSSDA terminology in terms of  $RMSE_z$  and  $Accuracy_z$  (95% confidence level).

It should be noted that the NMAS/NSSDA equivalencies in Tables 37.3 and 37.4 can only be made by assuming that all errors have a normal distribution. With the NSSDA, this assumption is clearly true since the 5% of outlier errors (like all others in the dataset) will also be squared in the RMSE calculation; however, with the NMAS, this assumption is not necessarily true since 10% of the check points are allowed to exceed the accuracy threshold, regardless of size. The NMAS outliers could presumably be very large errors, whereas the NSSDA outliers would presumably be relatively small, or else they'd skew the RMSE calculations. Both methods presume that all systematic errors have been identified and corrected prior to accuracy determinations.

## MAP PROJECTIONS

Whereas precise survey calculations are based on a mathematical surface called an ellipsoid, survey calculations performed for land development purposes routinely assume the earth is flat, and they utilize a Cartesian coordinate system of 2-D (x,y) or 3-D (x,y,z) coordinates. This section explains how map projections are used to map the nearly spherical earth onto a 2-dimensional map sheet, including cylinders and cones that are cut and laid flat; the next section will explain how State Plane Coordinate Systems are developed from these map projections tailored to best fit individual states within the United States.

To set the record straight from the very beginning, one cannot map a spherical surface onto a flat piece of paper without compromising one or more of the following: angles, distances, or areas. Something has to give! In projecting from a spherical or ellipsoidal surface to a flat surface, the true shape of the features must change. However, projections can be designed to preserve angles, distances, or areas, but not all three factors at once. Conformal projections retain both angles and distances within defined limits. The two most common conformal projections in the U.S. are the Lambert Conformal Conic and the Transverse Mercator.

## Scale Factors

All dimensions surveyed on the earth are changed proportionately when mapped at a reduced scale. Few points,



**TABLE 37.3 NMAS and NSSDA Horizontal Accuracy Equivalencies**

MAP SCALE OR COMPILATION SCALE	NMAS, CMAS AT 90% CONFIDENCE LEVEL	NSSDA, ACCURACY, AT 95% CONFIDENCE LEVEL	RMSE,
1" = 50' or 1:600	1/30th inch = 1.67' ground distance	1.90' ground distance	1.10' ground distance
1" = 83.3' or 1:1000	1/30th inch = 2.78' ground distance	3.17' ground distance	1.83' ground distance
1" = 100' or 1:1200	1/30th inch = 3.33' ground distance	3.80' ground distance	2.20' ground distance
1" = 200' or 1:2400	1/30th inch = 6.67' ground distance	7.60' ground distance	4.39' ground distance

however, are mapped exactly to the specified scale where the *scale factor* is exactly 1.000000000. Scale factor = (map distance)/(ground distance × scale). With UTM coordinates, the scale factor is normally between 0.99960 and 1.00040. With State Plane coordinates, the scale factor is normally between 0.99990 and 1.00010. The scale factor on any map varies throughout the map and may vary in different directions at any given point.

The overall *scale factor* (also called the *grid scale factor*) is the product of the *elevation factor* and the *zone scale factor*.

The elevation factor accounts for the elevation of the terrain being either above or below the ellipsoidal surface on which map projections are based. As shown at Figure 37.3, the elevation factor equals  $R/(R + H + N)$

where:

- R = the mean radius of the earth (assume 6,372,000 m)
- H = orthometric height or elevation above mean sea level (the geoid)
- N = the geoid height or separation from the ellipsoid (between -8 and -53 meters in the continental U.S.—i.e., always a negative number in the above equation).

As shown in Figure 37.3, where the terrain is above the

ellipsoid, the elevation factor will be slightly less than one. Alternatively, if the terrain were near to sea level (below the ellipsoid), the elevation factor would be slightly greater than one.

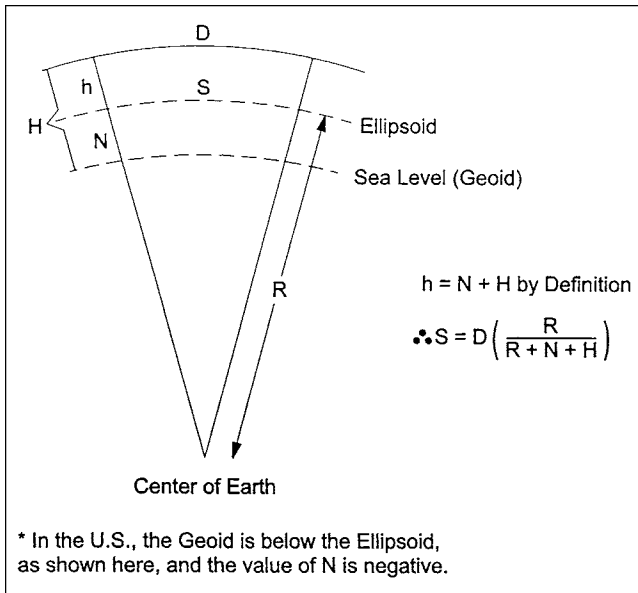
The zone scale factor accounts for the projection surface being either above or below the ellipsoid on which the map projection is based. The zone scale factor is exactly one only along a line where the projection surface is tangent to the ellipsoid. To better understand this concept, let's see how this works with the three standard projection surfaces: cylinders, cones, and planes.

**Cylindrical Projections**

Since the elevation factor has already accounted for those scale errors caused by variations in elevation, suppose the earth had a smooth ellipsoidal shape (no mountains or valleys), and the entire surface of the earth was covered with a thin glass, ellipsoidal shell, with all features painted on it, and then the earth was miraculously sucked out, leaving only the hollow, transparent shell shaped like the ellipsoid. Then, suppose there was a very bright point light source at the center of the earth, and the earth (shell) was wrapped with a huge paper cylinder that just touched the earth's ellipsoid at its equator. Then the light would cause all features to be projected through the glass to the cylinder of paper that could be unrolled as a flat 2-dimensional map.

**TABLE 37.4 NMAS and NSSDA Vertical Accuracy Equivalencies**

CONTOUR INTERVAL AT WHICH COMPILED	NMAS, VMAS AT 90% CONFIDENCE LEVEL	NSSDA, ACCURACY, AT 95% CONFIDENCE LEVEL	RMSE,
6"	3"	3.57" or 9.08 cm	1.82" or 4.63 cm
1'	6"	7.15" or 18.16 cm	3.65" or 9.27 cm
2'	12"	14.30" or 36.32 cm	7.30" or 18.53 cm
5'	30"	35.75" or 90.80 cm	18.24" or 46.33 cm



**FIGURE 37.3** Features scaled from a map (S) do not exactly match the surveyed ground distances (D) between those features, even after applying the published map scale because of variations in the elevations of mapped features. Figure 37.3 explains the elevation factor that causes scaling errors as a result of elevations above or below the ellipsoid on which the map projection is based.

Only features on the equator would be mapped to the correct scale where the zone scale factor is exactly 1, and other features would be mapped larger than they really are, with a zone scale factor larger than 1. The line along which the scale is exact is called a *standard line*. In this case, the standard line is the equator. As one proceeds towards the north or south poles, the areas are projected so much larger that the scale factors get too big and the map becomes very distorted. In fact, the north and south poles themselves would never appear on the map, even if the paper cylinder extended to infinity. This theoretical approach is the basis for the common Mercator projection used to produce world maps; on these maps, Greenland appears much larger than it really is, and nothing is normally mapped north of 80 or 85 degrees north latitude or south of 80 or 85 degrees south latitude. The important point to remember from a Mercator projection is that only those features on or near the equator are mapped to their approximate true scale where the zone scale factor is approximately 1.

Next, suppose the huge paper cylinder is turned sideways so that it only touches a single meridian that passes through the north and south poles along a Great Circle. If the cylinder touches the meridian that passes through Washington, D.C., for example, then only those locations with the same longitude as Washington, D.C. would be mapped to their true scale with a zone scale factor of 1. This standard line would include all points along a Great Circle that includes the north and south poles, all points along that meridian in Canada, the United States, Cuba, Colombia, and Peru in the Western Hemisphere, as well as other points along the opposite meridian in Russia, China,

Vietnam, Laos, Cambodia, and Indonesia in the Eastern Hemisphere.

Next, suppose the diameter of this cylinder is somewhat smaller than the diameter of the earth. Now the cylinder cuts through the glass ellipsoid along two circular standard lines that are smaller than a Great Circle. For the Washington D.C. example, one circle is parallel to but east of the Great Circle that passes through Washington, D.C., and the second circle is parallel to but west of the Great Circle that passes through Washington, D.C. (See Figure 37.4). Along these two circles, the zone scale factors are exactly 1. Between these two circles is the central meridian where features would be mapped smaller than they really are and the zone scale factors are less than 1. Outside these two circles, features would be mapped larger than they really are and the zone scale factors would all be larger than 1. Now, within a narrow north-south band, all locations are mapped at their approximately correct scale. This theoretical approach is the basis for the Transverse Mercator projection used throughout much of the world for areas longer in the north-south direction.

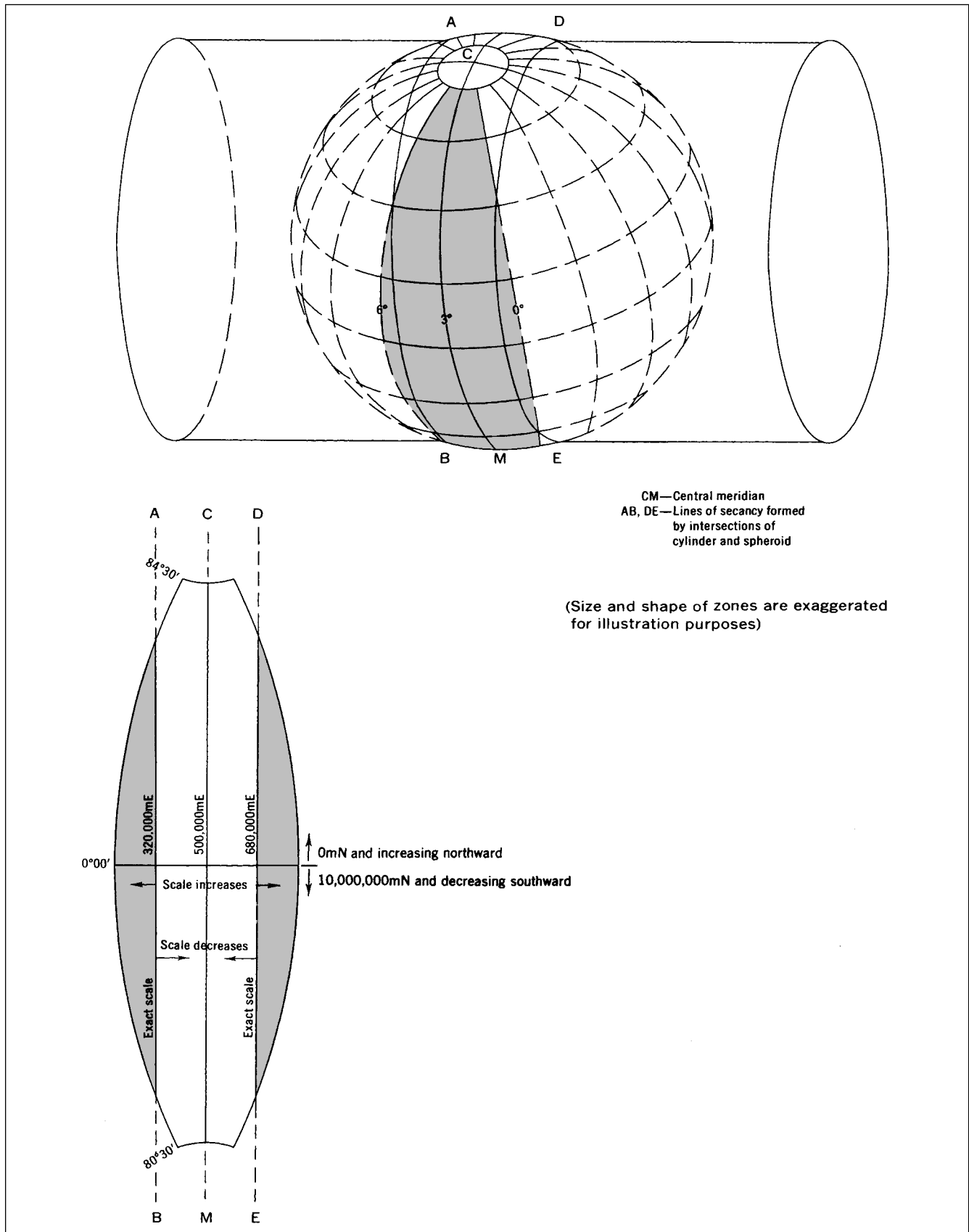
The important thing to remember about the Transverse Mercator projection is that the theoretical cylinder could be rotated to any longitude so as to change the narrow north-south band for which maps would have the approximate correct scale. The Transverse Mercator projection is best for states or small countries that are longer in the north-south direction (e.g., Idaho, Illinois, and the Florida peninsula).

### Conic Projections

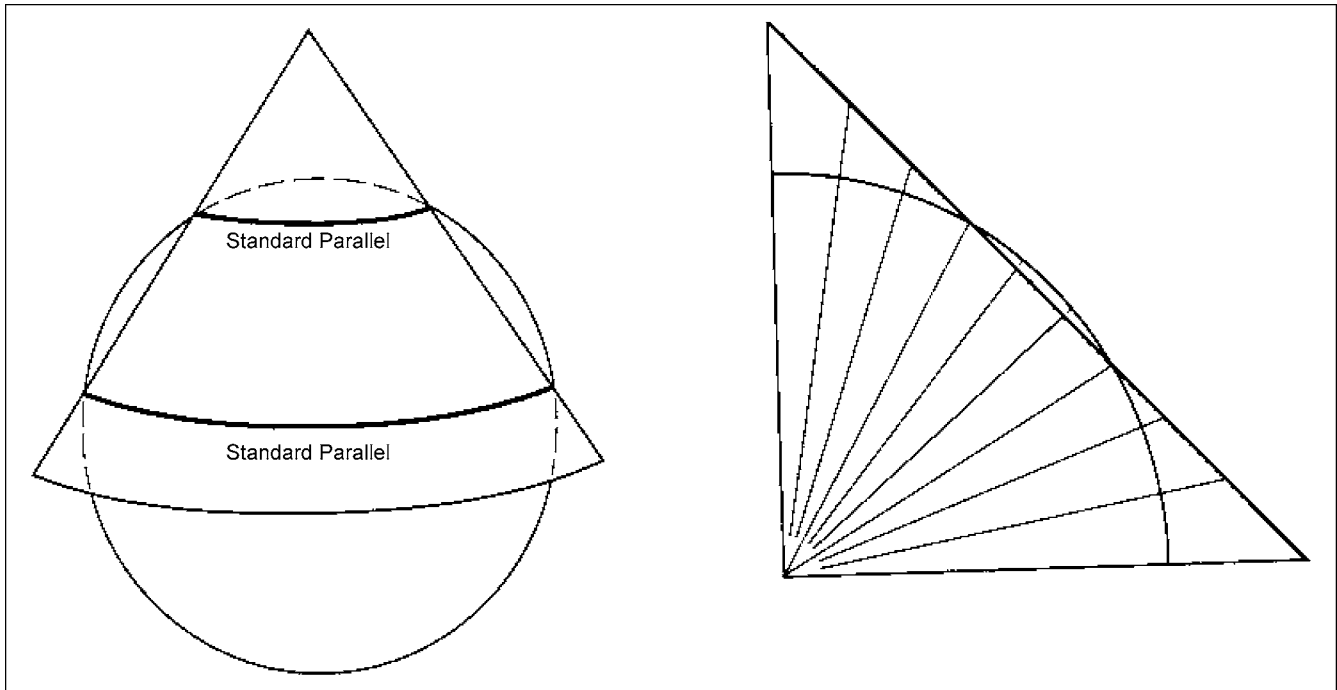
Now, suppose we had the same glass ellipsoid, but instead of having it wrapped in a paper cylinder, it is instead covered with a huge paper cone with the apex in space above the north pole. Depending on the shape of the cone, it could be tangent to the earth along a southern parallel (e.g., along the latitude of Miami), or along a northern parallel (e.g., along the latitude of Seattle), or anywhere else in the Northern Hemisphere. (A similar cone can be constructed as well for the Southern Hemisphere.) Wherever the cone is tangent to the ellipsoid, all locations with the same latitude would be mapped to scale with a zone scale factor of exactly 1, and all other locations would be mapped larger than they really are (i.e., with a zone scale factor larger than 1).

Then, suppose that the same cone is pushed down to cut through the ellipsoid along two different parallels (see Figure 37.5). Now the map scale would be correct along both of those parallels. Between those two parallels the mapped features would be too small, with a zone scale factor less than 1; and outside those two parallels, the mapped features would be too large, with a zone scale factor larger than 1. This theoretical approach is the basis for the Lambert Conformal Conic projection used throughout much of the world for areas longer in the east-west direction.

The important thing to remember about the Lambert Conformal Conic projection is that the theoretical cone can be reshaped to be steeper or shallower so as to change the



**FIGURE 37.4** This figure depicts the Transverse Mercator projection where the cylinder cuts through the Earth along two circles where the zone scale factors are exactly 1. Between these two circles (standard lines), the zone scale factors are slightly less than 1; outside these two standard lines the zone scale factors are slightly greater than 1.



**FIGURE 37.5** This figure depicts the Lambert Conformal Conic projection where the cone cuts through the Earth along two parallels where the zone scale factors are exactly 1. Between these two standard parallels (standard lines) the zone scale factors are slightly less than 1; outside these two standard parallels the zone scale factors are slightly greater than 1.

narrow east-west band for which maps would have the approximate correct scale. The Lambert Conformal Conic projection is best for states or small countries that are longer in the east-west direction (e.g., Tennessee, Pennsylvania, and the Florida “panhandle”).

### Plane Projections

Now, suppose that we have the same glass ellipsoid, but instead of having it wrapped with a cylinder or covered with a cone, it is simply touched by a plane surface. If the plane is tangent to the ellipsoid at only a single point, then all other points will be mapped larger than they really are (zone scale factor greater than 1) when the central light source projects through the glass to the plane paper.

Next, suppose that the plane doesn’t just touch the ellipsoid but cuts beneath the surface of the ellipsoid a bit. Now, where the plane cuts into the ellipsoid, the *standard line* will be a circle (or ellipse) where the scale is correct. Inside the circle, features are mapped smaller than they really are, and outside the circle, features are mapped larger than they really are. This theoretical projection is the basis for the Polar Stereographic projection used to map the north and south polar regions. This can be done elsewhere too, and Dewberry & Davis developed a special plane projection upon request for Washington Dulles International Airport so that the average scale factor was 1.0000000 (to seven decimal places), but State Plane coordinates, dis-

cussed next, are more generally understood and are best for land development projects.

### STATE PLANE VERSUS UTM COORDINATES

#### State Plane Coordinates

To simplify calculations for surveyors, each state in the United States has a State Plane Coordinate System (SPCS), initially adopted in the 1930s at the recommendation of what is now the National Geodetic Survey (NGS), whereby the scale factors are kept within defined limits, normally between 0.99990 and 1.00010.

Because they are broader in the east-west direction, most states utilize the Lambert Conformal Conic projection to satisfy their needs best. These states have adopted one or more unique SPCS zones, with two standard parallels passing through each zone. These parallels are standard lines where the zone scale factor is 1.00000. Tennessee, for example, has one such zone; Pennsylvania has two; Colorado has three; and Texas has five different Lambert Conformal Conic zones.

Most other states, longer in the north-south direction, tailored unique Transverse Mercator projections to meet the needs of their states. Here, each SPCS zone has two parallel lines (not meridians) where the zone scale factor is 1.00000. Delaware, for example, has one such zone; Illinois has two; Idaho has three; Wyoming has four; and Alaska has eight different Transverse Mercator zones.

Not all states follow standard north-south or east-west rules for selecting SPCS zones. California has six Lambert Conformal Conic zones, although it is longer in the north-south direction. Florida has one Lambert Conformal Conic SPCS zone in the (east-west) “panhandle” and two Transverse Mercator SPCS zones in the (north-south) peninsula. Only southeastern Alaska uses a projection other than Lambert Conformal Conic or Transverse Mercator; it uses a unique Oblique Mercator projection that will not be discussed here.

Each SPCS is based on a rectangular grid of X and Y coordinates. Both the Lambert Conformal Conic and Transverse Mercator projections have a *central meridian* that constitutes the Y-axis of a SPCS; the central meridian is assigned a large X-coordinate to avoid negative coordinates anywhere within the SPCS zone. Similarly, the X-axis is well to the south of the zone and is assigned a Y-coordinate of zero, so that all Y-coordinates within the zone are positive numbers.

For each SPCS zone, *grid north* differs from *geographic north*, except along the central meridian. In the Northern Hemisphere, the true meridians converge toward the central meridian as they go north.

Reprinted with permission from POB magazine, The GPS Observer, October, 1999, Figure 37.6 shows State Plane Coordinate zones in use in the United States. The zone boundaries are often not straight lines because it is common for entire counties (that include theoretical zone boundaries) to de-

cide on either one or the other SPCS zone to avoid having two different zones within a single county.

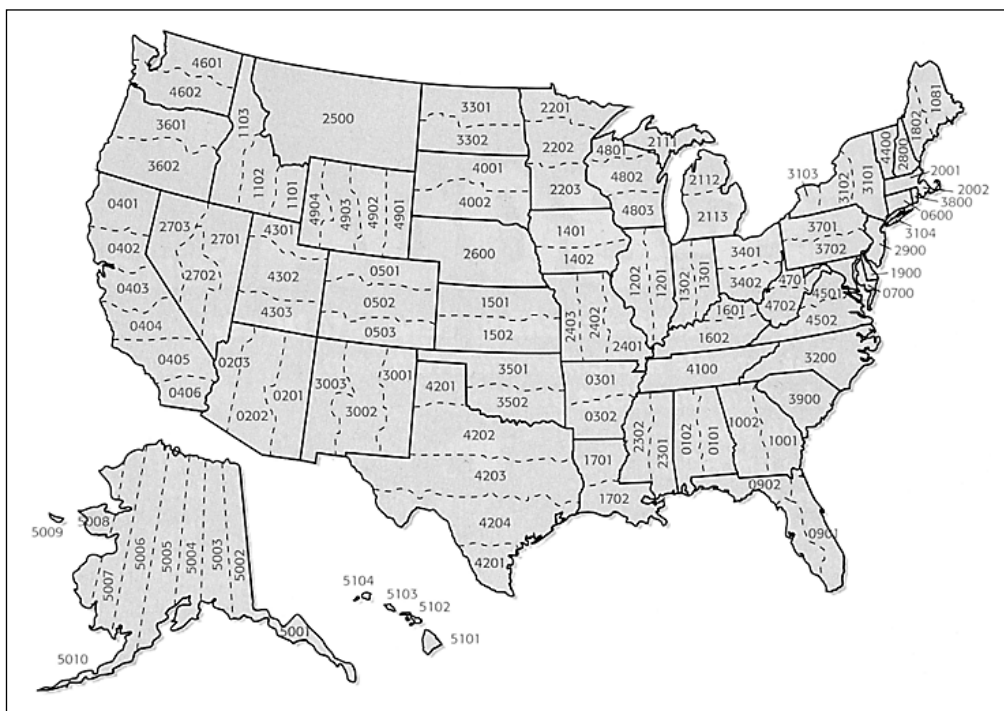
**Universal Transverse Mercator (UTM) Coordinates**

UTM coordinates were originally devised for use by the military so that soldiers could be trained to read and utilize a standard coordinate system, on military topographic maps, that makes the most sense globally. Each UTM zone covers 6° of longitude, so there are 60 UTM zones globally covering all 360° of longitude. In addition to topographic maps produced by the National Imagery and Mapping Agency (NIMA), maps with UTM coordinates became popular also for other federal agencies with nationwide mapping responsibilities, to include the U.S. Geological Survey (USGS), National Oceanic and Atmospheric Administration (NOAA), and the Federal Emergency Management Agency (FEMA).

Because UTM zone scale factors vary between 0.99960 and 1.00040, UTM coordinates should normally *not* be used for land development projects. State plane coordinates are preferred because their zone scale factors normally vary only between 0.99990 and 1.00010 with few exceptions (e.g., Montana and Nebraska, where zone widths are wider than allowed to keep zone scale errors within 1 in 10,000 limits).

**PHOTOGRAMMETRY**

Photogrammetry uses 2-D measurements on stereo photography to map either 2-D or 3-D coordinates of photo-



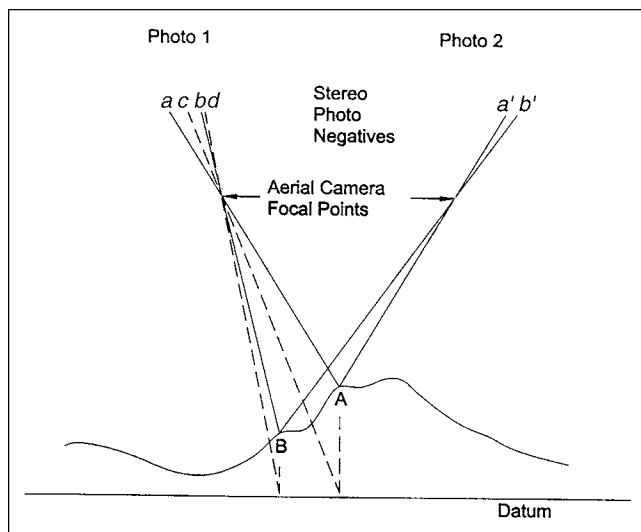
**FIGURE 37.6** Most states have adopted a State Plane Coordinate system so as to keep zone scale factors between 0.99990 and 1.00010, i.e., so that scaling errors will be 1:10,000 or less. Generally speaking, states longer in the north-south direction utilize one or more Transverse Mercator zones, whereas states longer in the east-west direction utilize one or more Lambert Conformal Conic zones. Zone boundaries normally curve to coincide with county boundaries in order to keep entire counties within a single zone.

identifiable terrain features. Planimetric maps do not include elevation data; they normally are defined by their map scale. Topographic maps include elevation data; they are normally defined by their contour interval. Photogrammetry has traditionally been used to map the terrain prior to land development projects, and again after construction to map the as-built development. Photogrammetry is also used to produce digital orthophotos and digital elevation models (DEMs). This section will cover basic photogrammetric information for land development applications. The basic reference for photogrammetric theory is ASPRS, 1980.

### Photogrammetric Equipment

Aerial photographs record perspective views of the terrain; features that are taller or closer to the camera appear larger than features that are farther away; thus, aerial photographs inherently have “relief displacement” that causes the images of elevated features to be displaced outward from the center of the photographs. Furthermore, aerial photographs rarely are truly vertical (i.e., looking straight down at the ground); there is normally some degree of roll, pitch, and yaw of the aircraft that causes photographs to be slightly oblique. Thus, aerial photographs also inherently have “tilt displacement” that causes some features to be tilted outward while other features are tilted inward relative to the center of an aerial photograph.

The objective of photogrammetric mapping is to remove all relief displacement and tilt displacement in order to map the terrain with an orthographic projection as though every feature on the ground is viewed looking straight down from infinity (see Figure 37.7).



**FIGURE 37.7** Photogrammetry converts images from a perspective projection into an orthographic projection, as though looking straight down from infinity. As shown here, points *a* and *b* in the left photo should be at *c* and *d* in order to be orthographically correct. Photogrammetric corrections are also applied, pixel by pixel, to digital orthophotos, but only for features at ground level.

There are three distinct generations of photogrammetric equipment that might be used for planimetric and/or topographic mapping for land development projects:

- Analog plotters are optical/mechanical photogrammetric instruments, used with hard-copy aerial photos, that physically replicate (at reduced scale) the spatial geometry that existed when the aerial photographs were taken. Analog plotters use different techniques to enable the photogrammetric map compiler to see the stereo images in 3-D, determine the elevation of points on the ground, and plot contour lines. Although often considered to be obsolete, analog plotters are still used by some photogrammetric companies for small mapping projects such as those for land development projects. Because of limited capabilities, analog plotters are the least accurate and the least efficient.
- Analytical plotters are computerized photogrammetric instruments, used with hard-copy aerial photos, that mathematically replicate the spatial geometry that existed when the aerial photographs were taken. Special optics enable the photogrammetric map compiler to see simultaneously the stereo images in 3-D and place a *floating dot* on the ground so as to determine the elevation of any point seen in stereo. When set for a specific elevation, the floating dot can be moved along the ground to trace a contour line of uniform elevation. Analytical plotters are very accurate and excellent for land development mapping projects.
- Soft-copy photogrammetric workstations are special purpose computers, used with digital aerial photos, that mathematically replicate the spatial geometry that existed when the aerial photographs were taken. Polarized images or other sophisticated techniques enable the photogrammetric map compiler to see simultaneously the stereo images in 3-D on a computer screen and determine the elevation of any point seen in stereo. Looking at the computer screen with polarized glasses (or alternative procedures), the compiler keeps a floating dot on the ground and traces contour lines. Many other steps are automated. Hard-copy photos are scanned at resolutions between 7.5 and 25 micrometers, typically, to convert them into high-resolution digital images. Soft-copy workstations yield high-accuracy planimetric and topographic maps, as well as other photogrammetric products such as digital orthophotos and DEMs. The basic reference for softcopy photogrammetry is ASPRS, 1996.

### Aerial Triangulation

Aerial triangulation (AT) has traditionally been required to control a block of aerial photographs, to force mapped coordinates to fit surveyed ground control, and also to control other mapped coordinates for which surveyed ground co-

ordinates are unknown. This traditionally has required a large number of photo-identifiable, surveyed ground control points. Photogrammetrists often use painted symbols or survey panels containing a  $Y$ ,  $T$  or  $X$ , pre-positioned and surveyed prior to acquisition of aerial photography, to insure that there are sufficient photo-identifiable points with known coordinates. When there are already ample photo-identifiable features, photogrammetrists may instead rely on photo points that naturally appear at desirable locations on the overlapping photography, and survey those points after the photography has been flown and the film developed. The objective of AT is to determine six exterior orientation parameters for each of the aerial photographs to be used; these six parameters include  $x$ ,  $y$ , and  $z$  coordinates (in airspace) of the focal point of the lens when each photograph was taken, as well as the roll, pitch, and yaw of the aircraft around the  $x$ -axis (direction of flight),  $y$ -axis (cross direction), and  $z$ -axis (vertical axis).

When these photo control points are surveyed from multiple control monuments surrounding the area, the AT solution is frequently weakened by the fact that multiple control monuments are often inconsistent relative to each other; then, AT residual errors are high. It is preferred that GPS procedures be used, with a single differential GPS base station at only one known control monument (the best in the area) in order to obtain consistent results, a strong AT solution, and maps that will be more accurate as a result; when this is done, then the AT residual errors are small. Experienced photogrammetrists know how to examine these residuals to determine if the survey control is strong or weak. Fully analytical aerial triangulation (FAAT) is generally considered to yield the highest accuracy and lowest residuals.

Since the mid 1990s, some aerial survey firms have regularly used airborne GPS and inertial measurement unit (IMU) technology to simplify the AT process. Airborne GPS is capable of measuring the  $x$ ,  $y$ , and  $z$  coordinates directly in airspace; and IMUs are capable of measuring the roll, pitch, and yaw directly as each photograph is exposed. Some ground control points are still used for the AT process, but the numbers are reduced significantly when airborne GPS and IMU technology is properly used. However, this advanced technology is more relevant to large mapping projects than to small projects for land development applications.

### Photo Scale

Photo scale is a critical factor in planning aerial photography. Photo scale is a function of the flying height above the terrain, and the focal length of the mapping camera. With a mapping camera having a given focal length, the required photo scale normally establishes the flying height to be used. Subsequent scale variation in a photograph or be-

tween successive photographs is caused by variation in the terrain elevation, by variation in flying heights, or both.

Figure 37.8 depicts two photographs taken over terrain having an average elevation of 400 ft above the datum and a range in elevation from 175 ft to 600 ft. In each case, the average photo scale is 1 in. = 200 ft.

With a 6-in. focal-length camera, the flying height would be 1600 ft above mean terrain (amt). At an elevation of 175 ft, the scale would be 6 in. (0.5 ft) divided by (1600 - 175) ft, or 1 in. = 238 ft. However, at an elevation of 600 ft, the scale would be 0.5 ft divided by (1600 - 600) ft, or 1 in. = 167 ft. This is a relatively large variation in scale.

With a 12-in. focal-length camera, the flying height would be 2800 ft amt. At an elevation of 175 ft, the scale would be 1 ft divided by (2800 - 175), or 1 in. = 219 ft. However, at an elevation of 600 ft, the scale would be 1 ft divided by (2800 - 600) ft, or 1 in. = 183 ft. This is a relatively small variation in scale.

Normally, long focal-length/narrow-angle lenses are better for planimetric mapping, allowing higher flying heights to be used. Lower flying heights with shorter focal-length/wide-angle lenses are better for topographic mapping where elevations are critical. The standard 6-in. lens is a compromise, and subsequent tables assume that 6-in. lenses are used.

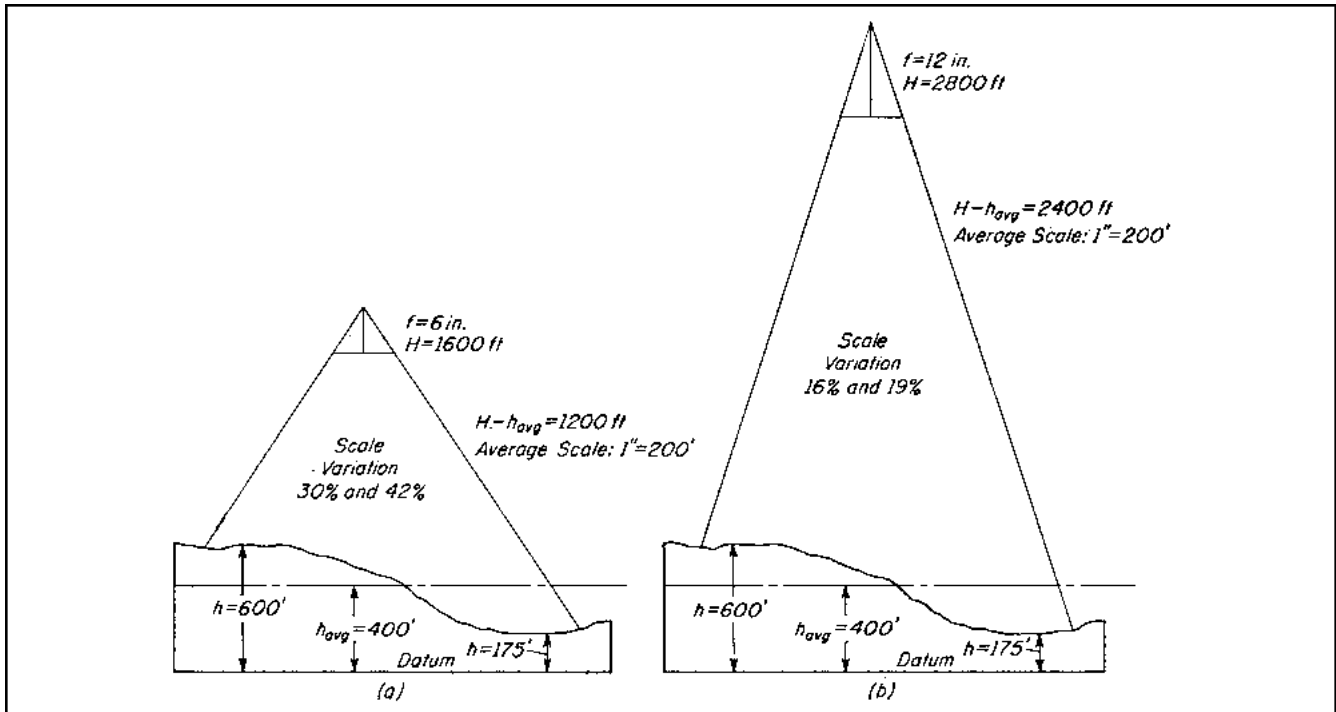
### Map Scale

Table 37.5 lists the photo scale and the flying height necessary for planimetric mapping at specified map scales, assuming a standard mapping camera with a 6-in. focal length. Note that analog plotters require aerial photography to be flown from a lower altitude in order to achieve accuracy comparable to that of analytical plotters or soft-copy workstations using higher altitude photography. Mapping photography normally is flown with a forward overlap of 60% and sidelap of 15–20%.

### Contour Interval

Table 37.6 lists the photo scale and the flying height necessary for topographic mapping at specified contour intervals, assuming a standard mapping camera with a 6-in focal length.

The most demanding requirement from these two tables controls the flying height to be used. For example, if maps with a horizontal scale of 1 in. = 50 ft (see Table 37.5) are desired with a 1-ft contour interval (see Table 37.6), then the photography should be flown at 2000 ft above mean terrain (instead of 2100 ft) if an analytical plotter is to be used. Lower-altitude photography yields more accurate maps, both horizontally and vertically, but lower-altitude photography will cost more than higher-altitude photography because high-altitude photography covers a larger area per photograph, requires fewer photographs, and simpler aerotriangulation.



**FIGURE 37.8** The average scale of a typical 9-in. × 9-in. aerial photograph depends upon the focal length of the camera, and the variation in terrain elevations relative to the altitude of the aircraft. Figure 37.7 shows two ways to acquire photographs with the same average scale, but acquired with cameras having 6-in. and 12-in. focal lengths flown at two different altitudes.

**Digital Orthophotos**

Digital orthophotos are digitized aerial photographs that have every pixel photogrammetrically corrected and repositioned so that relief displacement and tilt displacement are removed and the image has an orthographic projection as though looking straight down from infinity. There are sev-

eral different technical procedures for producing digital orthophotos, but the two points most critical for land development engineers and surveyors to understand are the following:

- Digital orthophotos are relatively inexpensive and are widely considered to be ideal as the base map for

**TABLE 37.5 Photo Scale and Flying Height for Desired Map Scale**

DESIRED MAP SCALE	ANALOG PLOTTERS		ANALYTICAL PLOTTERS & SOFTCOPY W/STA	
	PHOTO SCALE	FLYING HEIGHT	PHOTO SCALE	FLYING HEIGHT
1" = 20'	N/A	N/A	1:1680	840 ft amt
1" = 30'	N/A	N/A	1:2520	1260 ft amt
1" = 40'	1:2880	1440 ft amt	1:3360	1680 ft amt
1" = 50'	1:3600	1800 ft amt	1:4200	2100 ft amt
1" = 60'	1:4320	2160 ft amt	1:5040	2520 ft amt
1" = 100'	1:7200	3600 ft amt	1:8400	4200 ft amt
1" = 200'	1:14,400	7200 ft amt	1:16,800	8400 ft amt



**TABLE 37.6 Photo Scale and Flying Height for Desired Contour Interval**

CONTOUR INTERVAL	ANALOG PLOTTERS		ANALYTICAL PLOTTERS & SOFTCOPY W/STA	
	PHOTO SCALE	FLYING HEIGHT	PHOTO SCALE	FLYING HEIGHT
0.5 ft	1:1600	800 ft amt	1:2000	1000 ft amt
1 ft	1:3200	1600 ft amt	1:4000	2000 ft amt
2 ft	1:6400	3200 ft amt	1:8000	4000 ft amt
4 ft	1:12,800	6400 ft amt	1:16,000	8000 ft amt
5 ft	1:16,000	8000 ft amt	1:20,000	10,000 ft amt

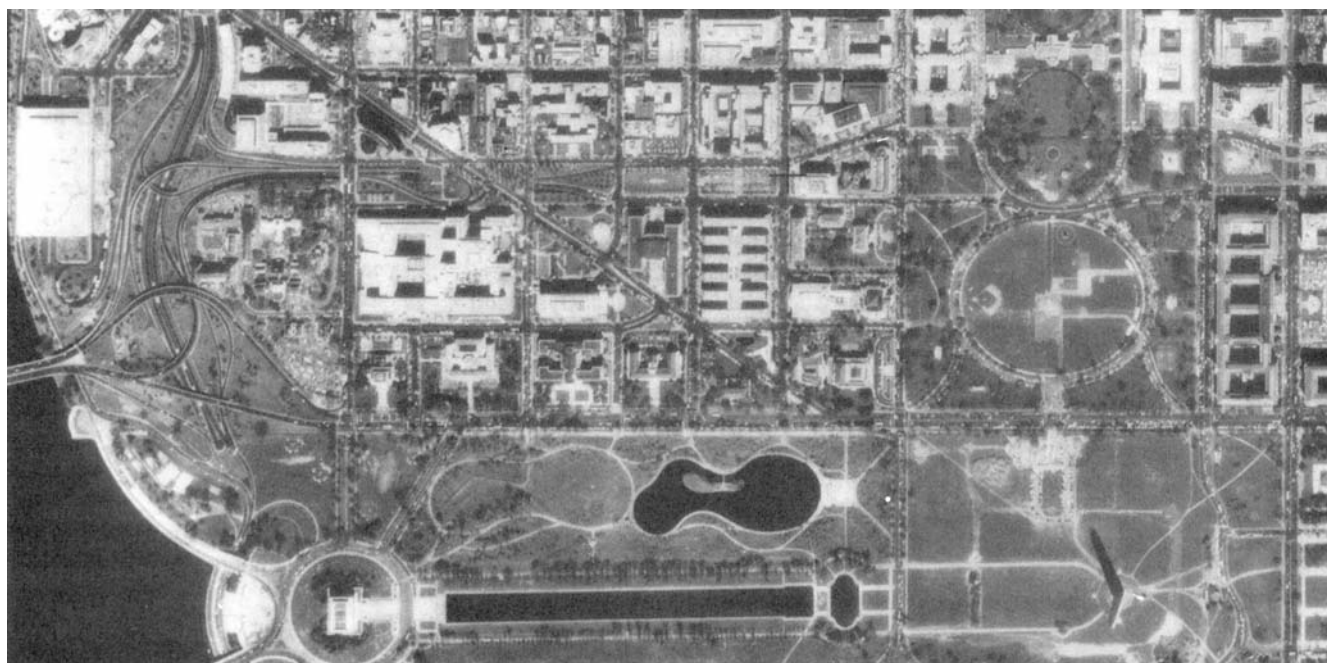
overlay of CADD and/or GIS vectors, points, and polygons developed by land development engineers and surveyors.

- Digital orthophotos contain no elevation data. They are orthographically correct only for features at ground level, whereas elevated features (tops of towers, utility poles, rooftops) are displaced outward from the centers of the digital orthophotos. Figure 37.9 shows a digital orthophoto of the National Mall in Washington, D.C. Note that the shadow of the Washington Monument correctly projects to the northwest because of the sun angle from the southeast; but the top of the Washing-

ton Monument is displaced to the southwest because the digital orthophoto is only correct at the base of this monument.

**Digital Elevation Models (DEMs)**

DEMs have at least two different definitions. Some refer to DEMs in a generic sense as covering all forms of digital elevation data that have x, y, and z coordinates of the terrain. Others refer to DEMs as only those datasets representing terrain elevations where the x and y coordinates have a uniform grid spacing (e.g., having “elevation posts” every 5 ft in x and y directions). In this example, when



**FIGURE 37.9** This is part of the southwest quadrant of a larger digital orthophoto that corrects every image pixel so that all surface points are projected orthographically, but elevated features (e.g., towers, rooftops) are displaced outward from the center of the photograph used to create the image. In this orthophoto of the National Mall in Washington, D.C., near the lower-right corner of the image, the Washington Monument is shown with its dark shadow pointing to the northwest. The top of the monument is displaced to the southwest; the monument base and ground features are correctly georeferenced.

using State Plane coordinates (in feet), all Northings and Eastings would be evenly divisible by 5. DEMs clearly refer to the bare-earth terrain, whereas Digital Surface Models (DSMs) refer to the upper surfaces (e.g., rooftops and tree canopies). There are several other terms that might be confused with DEMs, as explained in Chapter 6 of ASPRS 1996:

- Digital Terrain Models (DTMs) are often used synonymously with DEMs, especially in Europe. In the United States, however, DTMs are frequently defined as bare-earth terrain datasets that have a uniform grid spacing but supplemented with additional irregularly-spaced elevation data to reflect significant topographic features and breaklines to better characterize the shape of the terrain.
- Breaklines are linear features that describe a change in the smoothness or continuity of a surface. Hard breaklines define interruptions in surface smoothness; they are used to define streams, shorelines, dams, ridges, building footprints, and other locations with abrupt surface changes. Soft breaklines insure that known  $z$  values along a linear feature are maintained, and they insure that linear features and polygon edges are maintained in a TIN surface model, as described below, by enforcing the breakline as TIN edges; but they do not define interruptions in surface smoothness.  $Z$ -values along a breakline can be constant, or can vary throughout the breakline's length. These breakline data normally augment other data that may be uniformly or randomly-spaced. The net result is that the distinctive terrain features are clearly defined, and contours generated from DTM more closely approximate the real shape of the terrain.
- A triangulated irregular network (TIN) is a set of adjacent, non-overlapping triangles computed from irregularly-spaced points with  $x$ ,  $y$  coordinates and  $z$  values. The TIN data structure is based on irregularly-spaced point, line, and polygon data interpreted as mass points and breaklines. The TIN model stores the topological relationship between triangles and their adjacent neighbors (i.e., which points define each triangle and which triangles are adjacent to each other). This data structure allows for the efficient generation of surface models for the analysis and display of terrain and other types of surfaces. TINs usually require fewer data points than DEMs or DTMs, while capturing critical points that define terrain discontinuities, and are topologically encoded so that adjacency and proximity analyses can be performed. TINs have several other advantages over DEMs and DTMs; but they are probably best known for their superiority in surface modeling (e.g., calculation of slope, aspect, surface area and length; volumetric and cut-fill analysis; generation of contours; interpolation of surface  $z$  values; generation

of profiles over multiple surfaces; intervisibility analysis; and 3-D visualization and simulation).

DEMs traditionally have been produced from analytical plotters and soft-copy workstations. The highest accuracy is achieved when the photogrammetrist places individual "mass points" where he/she can see the ground. DEMs can also be produced by automated image correlation, but such DEMs are far less accurate and are likely to include DSM data of tree canopies and rooftops as well as DEM data of the bare earth.

There are two new technologies that are now competing with photogrammetry for production of DEMs: Light Detection and Ranging (LIDAR) and Interferometric Synthetic Aperture Radar (IFSAR). IFSAR is designed for major mapping projects and, therefore, is unsuitable normally for land development applications. LIDAR is relevant to land development and will be discussed in the next section.

### LIGHT DETECTION AND RANGING (LIDAR)

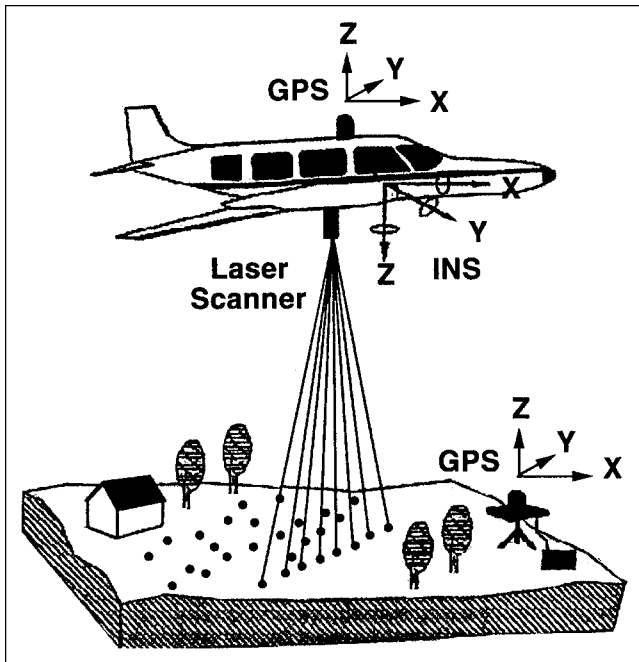
Light Detection and Ranging (LIDAR) is now highly competitive in generation of digital elevation data. Over a dozen topographic LIDAR vendors are in operation in the U.S., and one vendor also provides bathymetric LIDAR of relatively shallow coastal waters and near-shore terrain. The low-altitude topographic LIDAR systems are used for land development purposes, whereas the bathymetric LIDAR system normally is used for coastal mapping where the water is nearly clear. The best overall reference for LIDAR technology is ASPRS, 2001.

### Data Acquisition

Airborne LIDAR sensors emit between 5000 and 50,000 laser pulses per second, in some form of scanning array as shown at Figure 37.10. The most common scanning array goes back and forth sideways, relative to the direction of flight, providing a zig-zag pattern of points measured on the ground. The scan angle and flying height determine the nominal point spacing in the cross-flight direction, whereas the flying height and the airspeed determine the nominal point spacing in the in-flight direction. Each laser pulse has a pulse width (typically about 1 m in diameter) and a pulse length (equivalent to the short time lapse between the time the laser pulse was turned on and off again); therefore, each laser pulse actually is like a cylinder of light with diameter and length.

Several technologies must operate correctly in order to survey high-accuracy data points on the ground:

- Airborne GPS is needed to determine the  $x$ ,  $y$ ,  $z$  coordinates of the moving sensor in the air, surveyed relative to one or more differential GPS base stations. This establishes the origin of each of the 5000 to 50,000 laser pulses emitted each second.
- The inertial navigation system (INS) inertial measuring unit (IMU) directly measures the roll, pitch and



**FIGURE 37.10** LIDAR sensors survey 5,000 to 50,000 points per second on the ground by using airborne GPS to continuously survey the  $x$ ,  $y$  and  $z$  coordinate of the LIDAR sensor in the air; inertial navigation system (INS) inertial measuring unit (IMU) to measure continuously the roll, pitch, and yaw of the aircraft around the  $x$ ,  $y$ , and  $z$  axes; and the LIDAR sensor itself that measures the laser scan angles and the times it takes each laser pulse to travel to the ground and reflect back to the sensor. Post-processing is then required to remove elevated features and generate bare-earth digital elevation models (DEMs).

heading of the aircraft, establishing the angular orientation of the sensor about the  $x$ ,  $y$  and  $z$  axes in flight.

- The LIDAR sensor itself measures the scan angle of the laser pulses. Combined with IMU data, this establishes the angular orientation of each of the 5,000 to 50,000 laser pulses emitted each second.
- Finally, the LIDAR sensor measures the time necessary for each emitted pulse to reflect off the ground (or features thereon) and return to the sensor. Time translates into distance measured between the aircraft and the point being surveyed.

Some LIDAR sensors are capable of receiving multiple returns, up to five returns per pulse. This means that the sensor must be capable of recording up to 150,000 returns per second while in flight for a 30-KHz sensor. The “first return” is the first thing hit by a laser pulse; this could be a treetop, rooftop, ground point, or a bird in flight. When a laser pulse hits a soft target (e.g., tree or weed field), the first return represents the top or canopy of that feature. However, a portion of the laser light beam continues downward below the canopy and hits a tree branch, for example; this would provide a second return. Theoretically, the last return represents the bare earth terrain, but this is often not

the case. Some vegetation is so thick that no portion of the laser pulse penetrates to the ground. This is surely the case with sawgrass, mangrove, and dense forests where the canopy is so thick that a person on the ground can't see the sky.

A limitation of topographic LIDAR is that it normally does not penetrate water. In most cases, LIDAR pulses are absorbed by the water so there are no returns; however, sometimes there are returns from water, but those elevations cannot be trusted. Therefore, it is common to utilize digital orthophoto images to “cookie cut” out the LIDAR returns over water areas so these unreliable data can be discarded.

### Post-Processing

The major cost in most LIDAR projects involves the post-processing of the data as needed to generate a bare-earth digital terrain dataset. LIDAR data is far less expensive if the client needs or can accept digital surface models (DSMs) including treetops, rooftops, etc. where post-processing is minimal. The initial LIDAR dataset is irregularly spaced. For generation of bare-earth elevation data, automated and/or manual post-processing is used to eliminate laser points that impinged on elevated features; this creates data voids, so that the dataset becomes more irregular with regard to point spacing on the ground. Automated post-processing includes computerized procedures to detect elevation changes that appear to be unnatural; rooftops are identified with relative ease because there are abrupt elevation changes between the elevation of the yard and the rooftop. However, vegetation provides difficult challenges for automated procedures. Manual post-processing is more accurate (and more costly) and normally includes the overlay of data points on digital imagery so the analyst can see where the laser points are hitting something on the ground. This can be problematical if the imagery was not flown concurrently with the LIDAR data acquisition and some features moved (vehicles) or otherwise changed between the LIDAR and image acquisitions. Furthermore, LIDAR is often acquired at night, whereas imagery normally is flown during limited daylight hours when the sun angle is correct. Finally, after these post-processing steps are completed, regularly-spaced DEMs may be generated by interpolation between the irregularly-spaced data that remains after post-processing.

### LIDAR/Photogrammetry Comparison

LIDAR has an advantage over photogrammetry in generating accurate elevation datasets in dense vegetation. Photogrammetry requires two stereo photographs to see the bare-earth terrain from two different perspectives. The trees themselves block the view, so that the gaps between the trees must be fairly large in order to see the bare ground from two opposite directions. LIDAR only needs a single, near-vertical pulse to penetrate through a tree or between adjoining trees; these gaps can be small. When LIDAR pulses hit hard targets, such as rooftops, streets, dirt, sand,

rock, and short grass, the vertical RMSE is normally 15 cm or less when flown from altitudes of 3000 ft or less above mean terrain with a narrow scan angle (field of view). To achieve such accuracy from photogrammetry is possible, but it's more expensive because the photogrammetrist needs to select carefully points where he/she can see the terrain in stereo, and there will most likely be larger areas of obscured terrain where the photogrammetrist is unable to see the terrain in stereo.

Although a relatively new technology, LIDAR is used regularly and successfully to generate highly accurate, digital elevation datasets for land development projects and has become a technology of choice.

## REMOTE SENSING

Remote sensing measures characteristics of natural and man-made features from a distance, normally without walking on the area being sensed, and often recording information invisible to the human eye. Remote sensing includes sensors on aircraft, spacecraft, watercraft, and ground vehicles.

Photogrammetry is the most common form of airborne remote sensing, but hard-copy film and digital imagery is not restricted to the visible portion of the electromagnetic spectrum. For land development decisions, color infra-red (CIR) photography is preferred for mapping and analyzing vegetation and wetlands, for example. Digital multi-spectral imagery is used similarly to analyze vegetation, wetlands, geology, and other features that are difficult to classify or delineate using only the visible portion of the spectrum. Airborne thermal sensors are sometimes used by communities to determine where heat loss is occurring from buildings and utility systems. Magnetic sensors are used to help determine the best sites for potential oil-well drilling. Airborne radar sensors are best for mapping the terrain in areas that have perpetual cloud cover; and airborne LIDAR sensors are used to map the 3-D surface topography. Other sensors determine surface moisture content and other characteristics of the terrain. Depending on the flying height used, airborne imaging sensors may record ground resolutions of 6 in. or less, but linear features much narrower than the ground resolution are visible routinely. For example, highway paint stripes only 4 to 6 in. wide are often visible on digital orthophotos with single pixels covering a 1-m square on the ground. When the highest-resolution data is required, helicopter-borne sensors may be preferred (e.g., for highest-accuracy topographic mapping of a highway interchange where ground surveys are too dangerous because of passing traffic). Airborne remote-sensing systems normally integrate the specialized sensor with airborne GPS (to determine the location of the sensor in airspace) and IMUs (to determine the angular orientation of the sensor in airspace); these sensors enable the sensed information to be georeferenced in a geographic information system (GIS).

Numerous satellites also provide the types of sensors listed above, but the ground resolution is normally 1 m and

larger. Space Imaging's Ikonos satellite, successfully launched in 1999, was the first commercial satellite to acquire imagery with 1-m resolution; it also acquires multi-spectral imagery with a ground resolution of 4 m. These resolutions are suitable for land development planning purposes. Satellite imagery already acquired and available for sale to the public will be considerably less expensive than contracting for specialized imagery to be acquired upon demand. Hyper-spectral sensors with hundreds of narrower bands of the electromagnetic spectrum, and ultra-spectral sensors with thousands of still narrower bands, are becoming commercially available, but their utility for land development applications is yet to be determined.

Sensors on watercraft are most commonly used to map the subsurface terrain. Single-beam, multi-beam, and side-scan sonar systems, integrated with GPS receivers, are used for hydrographic mapping of larger-scale projects. Inexpensive, expedient techniques normally are used for small projects (e.g., a boat with differential GPS receiver to record 3-D coordinates to a point in the boat, and a weighted string to measure the distance below that point to the bottom of a lake or pond).

Sensors on ground vehicles are used to detect features that are not discernable from aerial photos. The most common are vans equipped with differential GPS, IMU, and cameras pointed forward, sideways and/or downward. When driving on a road at traffic speed and recording images every few feet, terrestrial photogrammetric procedures can be used to inventory and/or map traffic signs, fire hydrants, manholes, storm drains, parking meters, park benches, planters, statues/monuments, and other features that are either too small to be mapped accurately from above, or because features are obscured by trees, cars, or surrounding tall buildings.

Whereas airborne remote sensors normally are most relevant for land development purposes and constitute the default technology of choice, satellite, watercraft, and vehicular sensors are relevant to the degree that airborne sensors are unable to capture the required information.

## GEOGRAPHIC INFORMATION SYSTEMS (GIS)

### GIS Terminology

Some form of GIS is normally critical for land development projects. A GIS is defined herein as a computer hardware/software system that captures, stores, manages, manipulates, analyzes, displays, and/or prints spatially-referenced data for planning, managing, and decision making. For the purists, however, a GIS is not a true GIS unless it contains a database with descriptive data, supports spatial analysis and modeling, and exploits geographic data with a topological data structure that enables computer software to perform logical analyses of adjacency, proximity, and connectivity (e.g., as might be used to select the optimum site for a land development project). Thus, CADD systems that exploit digital data depicting map features, text, and

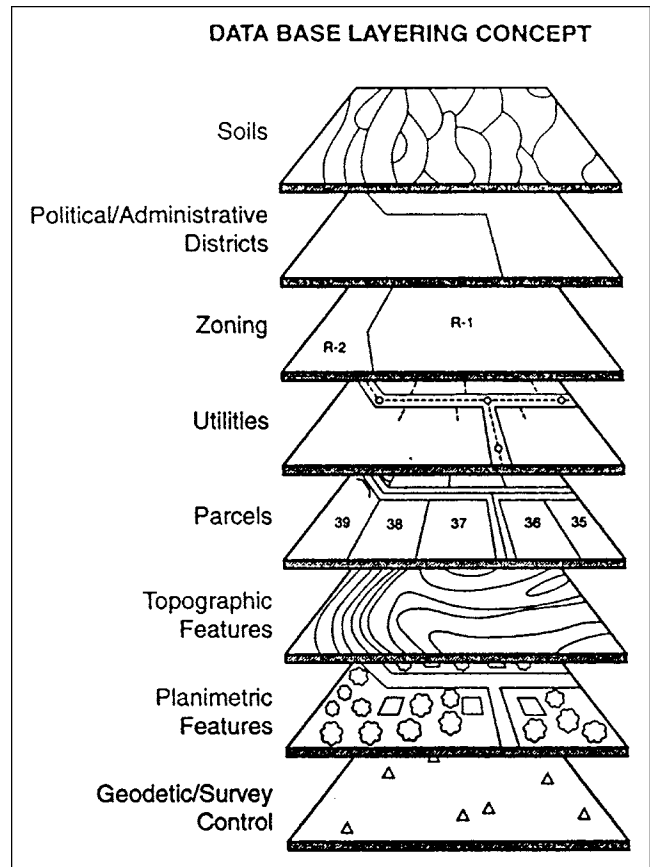
symbology, but lacking topological data structure or a supporting database, are not GISs in themselves but possible components of a GIS.

Databases are large collections of data in a computer, organized so that they can be expanded, updated, and retrieved rapidly for various uses. A GIS database is composed of sets of spatial (geographic) and descriptive (non-geographic) data managed by the computer software. Geospatial data is digital representation of geographic features on, above, or beneath the Earth's surface that can be used to generate a map or to display a map-like image on a computer screen. Geospatial data must include georeferenced coordinates (e.g., latitude/longitude, State Plane, or UTM coordinates) in order to display relative locations. Descriptive data describes attributes, characteristics, qualities, or relationships of geographic features and may include scanned images, such as digital pictures of houses, bridges, or other features. The geospatial and descriptive data can be linked or related, in GIS software, through linkage numbers or other common identifiers.

As an example, for telephone emergency response services, an E-911 GIS database may consist of two parts, geospatial and descriptive. The geospatial data can be used on the computer to display the location of incoming calls and roads leading to those locations from emergency service centers (rescue squads, ambulances, fire or police stations); this computer display could be optional in an E-911 system. The E-911 descriptive data normally would link telephone numbers with addresses and perhaps names, ages, and unusual medical needs of occupants, location of nearest fire hydrant or water supply, and designations of nearest emergency service centers. An E-911 system could operate without GIS software by using only the descriptive data with relational database management systems (RDBMS) such as dBase, Oracle, or Informix. However, if it is necessary to identify the telephone numbers, names, and addresses of all personnel living within a specified distance from a chemical spill, or if the E-911 is to select the optimum route for an emergency vehicle to access any given address, then a GIS would be required to link geospatial with descriptive data.

*Layering* enables raster and vector files of different thematic data to be registered geographically to one another, yet separated and/or combined as needed to simplify and optimize GIS analysis and display. Figure 37.11 graphically describes the layering concept. The multiple layers share a common coordinate system, so that disparate data can be registered accurately and overlain. A reference grid, planimetric base map, or digital orthophoto base map normally serves as the foundation for the overlay of vector data from separate layers.

*Geospatial analysis* includes analysis of adjacency (what adjoins a specified feature), proximity (what is within a selected distance from a specified feature), and connectivity (what is connected to a specified feature), as well as Boolean analyses of data on different layers. For example, in select-



**FIGURE 37.11** Both GIS and CADD systems rely upon a data base layering concept, with all layers georeferenced to each other and turned on or off as required.

ing an optimum land development site, we may ask the GIS to specify what areas within Monroe County have slopes less than 10%, are zoned for Z4 or Z5, but do not include soil types A, H or Y. The GIS would perform a Boolean analysis of data on four different layers—county boundary, slopes, zoning, and soil classification.

*Geospatial modeling* is used to support and extend GIS spatial analyses. For example, hydrologic models describe the relationship of slopes, soil types, vegetation cover, and land use characteristics of watersheds to predict the concentration of water at designated locations resulting from a specified rainfall. Similarly, hydraulic modeling is performed to compute surface-water velocities and elevations predicted from hydrologic-model discharges and stream-channel and floodplain cross-section characteristics. Environmental models predict the impact of natural or man-made events, including natural hazards (earthquakes, hurricanes, floods, fires) on the environment and man's use of that environment. Other GIS models are used for computer-assisted site selection and hundreds of other applications limited only by the user's imagination.

*Topological, topologic, and topology* all pertain to the building of "topo logic" into GIS data. Topology is the relationship between different features based on their geographic

location. Topology defines the way in which points, lines, and areas are formed and connected digitally so that logical relationships (e.g., adjacency, proximity, and connectivity) can be determined automatically by GIS software. With topological data structure, nodes are established at end points and intersections of lines; lines, which normally include intermediate shape points (vertices), are connected to nodes; the beginning, end, and direction of each line is defined, along with areas to the right and left of each line; and polygons (enclosed areas) are defined specifically by the bounding lines. Topological data structure allows for efficient data storage, since a feature's coordinates only need to be listed once, not many times, as long as that feature can be later defined by its relationships to other features. It also allows for efficient network analyses, E-911 vehicle routing, utility modeling, and buffer-zone analysis, to name but a few of its applications.

### GIS Processes

Data on a map or similar product can be captured by digitizing each feature individually or by using an electronic scanner to capture an entire sheet of features. Digitizing converts geospatial features on a map into digital format by converting point, line, and area features of the map into  $x/y$  or  $x/y/z$  coordinates. A single set of coordinates represents a point. A string of coordinates represents a line. One or more lines that outline an area, with an area label point inside, represents an area or polygon.

Manual digitizing is used commonly to generate vector data. The operator mounts the paper map to be digitized onto a digitizing table, registers the map, points the cursor's cross hair, and moves the cross hair along lines to be digitized while pushing the appropriate cursor buttons that record  $x$  and  $y$  coordinates by several methods. Intersecting lines are snapped to each other, symbols are placed, and annotations are entered by keystroke to include placement and rotation angle. This process is called *heads-down digitizing* because the operator is looking down at the digitizing table and using the cursor to trace the map features.

For complex digitization projects, it is often more cost-effective to use an electronic scanner to convert hard-copy maps into raster format and then to utilize commercial raster-to-vector software to trace the raster images interactively and convert them into vector format. This data-capture process is sometimes called *heads-up digitizing* because the operator is looking up at the computer screen to trace the map features. Other heads-up digitizing might involve the tracing of features from digital orthophotos that are displayed on the computer screen.

Utilizing specialized coordinate geometry (COGO) software, a GIS operator can key precise dimensions, angles, or geometric descriptions of features into a computer and utilize the COGO's mathematical algorithms to compute 2-D or 3-D coordinates of points and lines used on computer displays and printouts. COGO techniques especially are relevant for land surveyors who utilize total-station sur-

vey equipment to record automatically distances and angles, which can then be entered directly into COGO software routines for generating precise maps of surveyed areas.

In constructing vector GIS data sets, it is essential that nodes, lines, and polygons be digitized and "cleaned" in such a way that lines are connected properly with no gaps or overshoots, and that node-to-line and line-to-polygon relationships are established correctly. Without rigorous identification of such spatial relationships, automated "topologic" cannot be performed by the GIS. During the "cleaning" process, tolerances must be set carefully in order to prevent inadvertent data damage. Procedures for building topology are beyond the scope of this text and vary depending upon the GIS software being utilized.

### Land Information System (LIS)

LISs are GISs that specifically describe land records pertaining to legal boundaries, ownership of properties, land values and tax assessments, and inventory of natural resources on defined properties. They are used primarily by local governments and the real estate industry. The cadastral layer of an LIS contains legal descriptions of boundaries and markers and other descriptions defining ownership and rights to the land, such as easements, rights-of-way, and water/mineral rights. Parcel boundaries, which define ownership, are often described on a deed as metes and bounds or distances and bearings and may include extensive narrative descriptions of a property. Thus, LISs contain geospatial data linked to sizable descriptive databases.

In 1999, the Federal Geographic Data Committee (FGDC) published the "Cadastral Data Content Standard for the National Spatial Data Infrastructure." This standard can be downloaded from the FGDC web site: [www.fgdc.gov](http://www.fgdc.gov), under Standards, Status of FGDC Standards, and FGDC Endorsed Standards.

### Automated Mapping/Facilities Management (AM/FM)

Although not a true GIS, AM/FM is a GIS technology focused on the specific segment of the market concerned with management of geographically-distributed facilities and utility systems. Understanding of AM/FM is especially relevant for land development projects for federal agencies where conformance to strict standards are paramount. In 2000, the FGDC published FGDC-STD-010-2000, Utilities Data Content Standard, for the following utilities:

- Compressed air system
- Control and monitoring system
- Electrical exterior lighting
- Electrical system
- Fuel system
- General utility features
- Heating and cooling system

- Industrial system
- Natural gas system
- Saltwater system
- Storm system
- Wastewater system
- Water system

This standard can be downloaded from the FGDC web site: [www.fgdc.gov](http://www.fgdc.gov), under Standards, Status of FGDC Standards, and FGDC Endorsed Standards.

### Computer Aided Design and Drafting (CADD)

CADD software is used primarily by civil, electrical, and mechanical engineers and architects for interactive 2-D and 3-D design of infrastructure and facilities such as subdivisions, shopping malls, individual buildings, roads, bridges, water and sewer systems and other utilities. CADD software allows for efficient data entry and editing of engineering and architectural drawings using geometric shapes and functions (e.g., angle/distance dimensions, circles and ellipses, squares and rectangles, parallel lines, tangent lines, spline curves, parabolas, bisecting lines, and fillets), coordinate geometry (COGO) digitizing of surveyed distances and bearings, and computer display and printing with variable colors, line weights, and line styles.

Architect and engineer (A&E) technical drawings have been generated digitally by CADD technology for several decades, and existing technical drawings are converted routinely to digital form with relative ease by manual digitization or by electronic scanning and raster-to-vector conversion. Like the word processor, CADD technology makes it easy to make changes and to keep documents up-to-date. CADD technology enables selected items to be placed on separate layers or levels and displayed in different colors, line weights, and line styles; it also allows selected items to be turned on as needed to overlay relevant information or to be turned off as needed to reduce clutter. Technical drawings can be maintained/updated more easily and printed upon demand. Rather than using a camera to reduce or enlarge a paper map image photographically, computers and CADD software are used to zoom out or zoom in for ease of viewing and to change scales with ease. CADD technology enables automated calculation of distance, perimeter, and area measurements. A significant advantage of CADD technology is that features can be displayed in three dimensions to detect design deficiencies and to view designed items in perspective as opposed to traditional top, front, or side views. Perhaps the most significant advantage is that CADD drawings can be “intelligent graphics” with computer queries of linked non-graphic data, while traditional technical drawings are “non-intelligent graphics” requiring all interpretations and analyses to be performed manually.

## GLOBAL POSITIONING SYSTEM (GPS)

### Background

The NAVigation Satellite Timing and Ranging (NAVSTAR) Global Positioning System (GPS) is a satellite-based, navigation system operated and maintained by the Department of Defense (DoD). It was intended originally to provide passive global positioning and navigation for land, air, and sea-based strategic and tactical forces with horizontal accuracy on the order of 10 m; the (poorer) vertical accuracy was inconsequential essentially. During the 1990s, differential GPS (DGPS) procedures were developed for civil applications that yield 3-D positions with centimeter-level accuracy. These DGPS procedures are now used routinely to transfer elevations (as well as horizontal coordinates) over long distances efficiently with accuracies comparable to or better than conventional leveling and traverses.

This section will focus on those aspects of DGPS surveys relevant to high-accuracy requirements of the land development community. The last subsection will discuss GPS elevation surveys. Since accurate elevations are more difficult to achieve from GPS surveys than are accurate horizontal coordinates, steps taken to achieve specified vertical accuracies will yield horizontal coordinates of even higher accuracy.

Today, GPS is the preferred method for all control surveys performed to establish “network accuracy” relative to the National Spatial Reference System (NSRS) in accordance with Geospatial Positioning Accuracy Standards of the Federal Geographic Data Committee (FGDC). Network control surveys provide horizontal and vertical positions of points to which supplementary local surveys are performed with conventional survey procedures. GPS surveys provide the preferred control for photogrammetric mapping and for the extension of other forms of land surveys critical for planning, engineering and land development.

### GPS Satellites

The full GPS constellation consists of 24 NAVSTAR satellites, launched four each into six orbital planes inclined at 55° from the equator. Each satellite circles the globe twice per day at an altitude of about 10,900 nautical miles above the Earth. This insures that six to ten GPS satellites are in view at all times, anywhere on Earth. Each satellite has direct line-of-sight to 42% of the Earth, with a 28° cone that matches the dimensions of the Earth as seen from 10,900 nautical miles in space. Each satellite contains atomic clocks that must be accurate to a few billionths of a second since each billionth of a second timing error translates into a 1-ft positioning error on the ground.

Every GPS satellite in the constellation continuously transmits high-frequency carrier waves at the same two L-band frequencies. L<sub>1</sub> has a carrier frequency of 1575.42 megahertz and a wavelength of about 19 cm. L<sub>2</sub> has a carrier frequency of 1227.60 megahertz and a wavelength of about 24 cm.

The  $L_1$  signal is modulated with a 10-million bits-per-second Precise Code (P-code) and a 1-million bits-per-second Coarse Acquisition Code (C/A-code). The  $L_2$  signal is modulated with only the P-code. The P-code and C/A-code are unique to each satellite. A 50-bit-per-second binary data stream (0s and 1s) is superimposed on the C/A-code and the P-code signals, including update information on the satellite clocks, ephemeris data, satellite health status and other data. The ephemeris data consists of 16 constants that define a satellite's location and orbit when it transmitted its timing pulses. The precisely-timed binary pulses transmitted from each satellite take less than  $\frac{1}{10}$ th of a second to arrive at the Earth. Altitude and velocity-control subsystems orient the satellite so that its helical antennas point correctly to the Earth. Unmanned tracking stations continuously monitor the track of each GPS satellite to update its ephemeris and to correct for any timing errors in on-board atomic clocks.

For many years, a controversial disadvantage of GPS pertained to Selective Availability (S/A)—the deliberate degrading of the signal by DoD to prevent potential adversaries (lacking DoD encryption key devices) from achieving full GPS functionality. S/A produced a Standard Positioning Service (SPS) with accuracies at the 100-m level. The alternative to S/A was DoD's Precise Positioning Service (PPS). PPS used encrypted codes from which real-time 3-D accuracies at the 10 to 15-m level could be achieved. In 2000, S/A was turned off, so that single civilian GPS receivers can now achieve 3-D accuracies at the 10 to 15-m level. However, S/A has never had a bearing on the most accurate GPS surveys performed by differential GPS procedures using a minimum of two GPS receivers.

### GPS Receivers

In the simplest of terms, a GPS receiver is a range measurement device; distances affected by clock errors are measured between the receiver point and a minimum of four satellites, and the position is determined from the intersections of the four range vectors. These distances are determined by a GPS receiver that accurately measures the time it takes a signal to travel from the satellite to the station. According to Kleusberg, 1992:

A GPS receiver measures carrier phases and pseudoranges. Systematic error effects contaminate both types of data. If we disregard these errors for a moment, we can see both the carrier phase and pseudorange as ranges from the satellite to the receiver, but with two main differences. First, carrier-phase measurements have millimeter-level precision, compared with meter-level precision for pseudoranges. Second, carrier-phase measurements are ambiguous by an unknown integer number of carrier cycles, the so-called *carrier-phase ambiguity*. This carrier-phase ambiguity remains constant over time as long as the receiver is phase-locked to the incoming signal. No similar ambiguity exists for the pseudoranges.

In simple terms, we can look at the pseudorange as a low-precision unambiguous range measurement and the carrier phase as a high-precision ambiguous range measurement. If we correctly identify the carrier-phase ambiguity number, we can convert the ambiguous carrier-phase measurements into very precise unambiguous range measurements. It is this type of modified GPS measurement that gives the very high differential positioning and surveying accuracies.

Researchers have developed various operational procedures over the past years to identify the correct ambiguity number, also called *ambiguity resolution*. . . . Of critical importance for all ambiguity resolution methods is the fact that the ambiguity number is an integer by definition.

Hundreds of different kinds of GPS receivers are on the market, produced by about 40 different manufacturers. Some GPS receivers are hand-held, about the size of a pack of cigarettes; others are more the size of a shoe box. Some are mounted on cars, trucks, boats and aircraft to provide real-time navigation aids. Some receivers mounted on vehicles, hydrographic survey ships and dredges, for example, have graphical interfaces that map the route traveled while recording attribute data such as street addresses, road intersections, depth soundings, etc.

A single-frequency receiver tracks the  $L_1$  frequency signal only; it can be used effectively to develop relative positions that are accurate over baselines of less than 50 km. A dual-frequency receiver tracks both the  $L_1$  and  $L_2$  frequency signals; it is more effective on longer baselines. (Baselines are vectors between a pair of GPS receivers from which simultaneous GPS data has been collected and processed.)

Five-channel GPS receivers are popular because they track the minimum of four satellites needed for positioning, plus a spare for redundancy or as a spare in the event "lock-on" is lost to one of the satellites. Loss of lock occurs when the receiver (blocked by a building, overpass, etc.) is unable to record satellite signals continuously or the transmitted satellite signal is disrupted. Twelve-channel receivers are good for differential GPS (DGPS) base stations in order to track all potential satellites above the horizon and to support multiple mobile receivers that may be tracking any four or five of the visible satellites.

Some GPS receivers are mounted in aircraft, boats, trucks, ambulances and other vehicles for real-time navigation purposes. Others are interfaced with mapping systems that enable routes traveled to be digitally mapped while driving over the terrain.

### GPS Survey Procedures

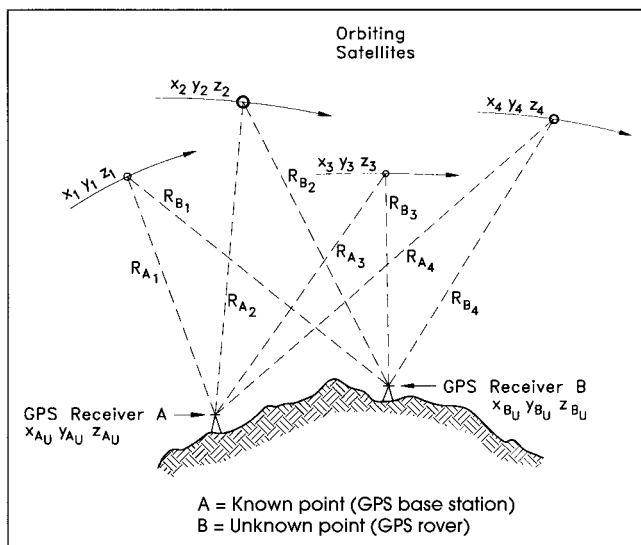
*Autonomous point positioning* is the most common application of GPS when high accuracy is not required. When operating in this passive, real-time navigation mode, pseudoranges to NAVSTAR satellites are observed by a single receiver positioned on a point for which a position is desired. This receiver may be positioned to be stationary over a point (i.e., static) or in motion (kinematic), such as on a



backpack, vehicle, aircraft, or boat. Since Selective Availability has been turned off, the civil user is able to achieve autonomous point positioning with horizontal accuracy of approximately 10 mm. Figure 37.12 would reflect both points A and B as being surveyed autonomously if the coordinates of neither point are known.

*Differential GPS (DGPS) positioning* is preferred when higher levels of accuracy are needed for land development and other engineering applications. DGPS uses two or more GPS receivers, with a base station receiver on a position of known location. GPS coordinates for the base station are compared with the known coordinates for the base station, and the differences (errors) are inserted into the calculations for the other GPS receivers locked onto the same four satellites as used by the base station for its GPS position. Since most error sources are the same for all GPS receivers in the same general area locked on to the same GPS satellites at any given instant, these errors can be measured and subtracted from all such GPS solutions. Figure 37.12 would represent DGPS procedures if A, for example, had known coordinates, and B was surveyed more accurately by knowing the corrections at A for the same four satellites observed at the same time.

The use of the carrier frequencies ( $L_1$  and  $L_2$ ) is the basis for use of GPS for precise geodetic and engineering surveying. Carrier-phase tracking takes advantage of the short 19-cm wavelength cycle. Accuracies achievable are generally accepted to be 1% of the wavelength (i.e., 1.9 mm). Basically, carrier-frequency tracking measures the phase differences between the satellite and receiver frequencies. The



**FIGURE 37.12** In this diagram, if GPS receivers A and B are both unknown points, then their locations are independently determined using absolute (autonomous) positioning procedures. However, if A is known and B is unknown, then differential positioning techniques, described above, can be used to determine the location of B with far greater accuracy; in this example, A would serve as the DGPS base station, and B would serve as the DGPS rover.

phase differences are changing continuously because of the moving satellite(s) Doppler effects; however, such effects are resolved in the receiver and subsequent data post-processing.

There are basically three different DGPS techniques in use today relevant to land development: (1) static, (2) rapid-static, and (3) real-time kinematic (RTK):

- *Static positioning* is the most accurate and most common form of DGPS positioning, and it is also used for surveying long baselines. At least two GPS receivers are used at different locations to collect carrier-phase data in a static (non-moving) mode. Normally, one GPS receiver (the DGPS base station) is positioned over a known point, and the second receiver (the rover) is positioned over an unknown point. Both GPS receivers receive signals from the same four (or more) satellites, recording data for 30 minutes to 2 hours. All data is post-processed simultaneously to determine the baseline between the receivers ( $\Delta$  latitude,  $\Delta$  longitude, and  $\Delta$  height). For some applications, Continuously Operating Reference Stations (CORS) operated by the National Geodetic Survey (NGS) can serve as the DGPS base station. Data from CORS stations can be downloaded and post-processed to perform DGPS corrections for one or more DGPS rover units; see the NGS home page ([www.ngs.noaa.org](http://www.ngs.noaa.org)) for details. If all known and unknown points are in a local area, as is common for land development projects, then the methods described below will yield comparable results but with significant reductions in observation times.

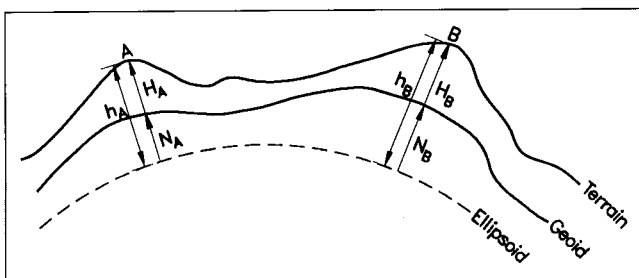
- *Rapid static positioning* is similar to static positioning, described above, but with a much shorter static period for observations. According to Kleusberg, 1992, the reduction in observation time results from faster ambiguity resolution using redundant carrier-phase measurements from multi-channel receivers that track all visible satellites. By simultaneously processing all redundant carrier-phase measurements in conjunction with advanced statistical analysis procedures, the resolution of ambiguity can be accelerated significantly.

- *Real-time kinematic (RTK) positioning* allows the operator to determine instantly the positioning of the GPS rover unit, enables the operator to navigate precisely to points, and delivers the answers while the operator is in the field and can fix mistakes there instead of discovering them back at the office. RTK procedures can be used for construction stake out and for real-time 3-D positioning of scraper and dozer blades, for example, on larger mining and construction projects. With RTK positioning, the base-station configuration uses one GPS receiver and radio to transmit the satellite observations to DGPS rover units on the project site. The rover unit configuration also has one receiver and radio, as well as a hand-held data-collector inter-

face. The rover sub-system makes its satellite measurements and also receives the base receiver's satellite data via the radio. Once the rover has all of this data, it computes its precise position relative to the base station. This process requires a short initialization procedure and continuous four-satellite tracking. If satellite lock is lost, the receiver is reinitialized at the last known point prior to loss of lock. A loss of radio contact, however, does not mean that the survey must be reinitialized. During an RTK survey, it is possible to lose lock with the reference (base station) radio and regain carrier-phase differential positioning when radio contact is re-established. So long as both receivers maintain constant, four-satellite tracking, centimeter-level positioning is possible. This method is much faster, using 5 to 30 seconds compared to other DGPS techniques that take several minutes to several hours. It should also be noted that the U.S. Coast Guard and U.S. Army Corps of Engineers operate hundreds of sites that comprise the Nationwide Differential GPS (NDGPS) system. Although originally devised to radio DGPS RTK corrections for real-time positioning of ships, barges and dredges in controlled waters, these radio signals are available for use by hundreds of other RTK positioning applications.

### GPS Elevation Surveys

DGPS procedures can meet Third-Order standards for vertical control surveys as would be obtained using conventional spirit levels. Geodetic heights determined by GPS point positioning techniques are based on WGS 84 and are heights above the WGS 84 ellipsoid. Traditional elevations from spirit leveling are orthometric heights above mean sea level (MSL), approximately equal to the geoid. As indicated at Figure 37.13, corrections for the undulation of the geoid ( $N$ ) must be made in order to convert GPS-derived geodetic heights ( $h$ ) into orthometric heights ( $H$ ) comparable to spirit leveling. This correction,  $N$ , is the separation between



**FIGURE 37.13** The elevation  $H$  is the orthometric height of the surface relative to the geoid or mean sea level (MSL) as measured by traditional spirit level surveys. The geodetic height,  $h$ , is the height of the surface relative to the ellipsoid as measured by GPS surveys. The geoid height or undulation,  $N$ , is the height of the geoid relative to the ellipsoid;  $N$  is always a negative number in the continental U.S. for use in the equation:  $h = H + N$ .

the ellipsoid and the geoid. The conversion formula is as follows:  $h = N + H$ ,

where:

- $h$  = ellipsoid height, the height of the surface relative to the ellipsoid
- $H$  = orthometric height, the height of the surface relative to the geoid (commonly called the "elevation")
- $N$  = geoid height, the undulation or separation between the ellipsoid and the geoid (always a negative number in the continental U.S.)

The National Geodetic Survey (NGS) has published a national geoid model and PC-based computer software, called GEOID99, which converts from ellipsoid heights to orthometric heights nationwide with a standard deviation of 1 cm so that the equivalent of Third-Order leveling can be accomplished nationwide with GPS. NGS has also published another PC-based software program, called VERTCON, which converts NGVD 29 values to NAVD 88. See the NGS home page for details ([www.ngs.noaa.gov](http://www.ngs.noaa.gov)).

The "bible" for GPS elevation surveys is NOAA, 1997, Technical Memorandum NOS NGS-58, "Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm)," and/or follow-on NGS guidelines for GPS-derived orthometric heights. NGS-58 provides guidelines for seven different types of GPS surveys: (1) Control, 2 cm and 5 cm; (2) Primary Base, 2 cm; (3) Primary Base, 5 cm, (4) Secondary Base, 2 cm, (5) Secondary Base, 5 cm, (6) Local Network, 2 cm, and (7) Local Network, 5 cm. As the very minimum, at least one 2-cm or 5-cm local network station should be established for new land development projects in order to prevent mismatches of other relevant data compiled relative to the NSRS (e.g., Flood Insurance Rate Maps that impact the owners' ability to purchase flood insurance).

The following is a summary of NGS-58 guidelines for local network stations:

- For both accuracy standards, use dual-frequency GPS receivers and geodetic quality antennae with ground planes. Fixed height poles are mandatory for 2-cm accuracy, optional for 5-cm accuracy.
- For 2-cm accuracy, the minimum occupation time per session is 30 min; for 5-cm accuracy, there is no minimum occupation time. For both accuracy standards, all survey sessions must be repeated on two different days with significantly different satellite geometry.
- For 2-cm accuracy, the maximum distance between same or higher-order stations is 10 km. For 5-cm accuracy, the maximum distance between stations is 20 km.
- All local network stations must be traceable to two or more primary base stations so as to relate the local

network to the National Spatial Reference System (NSRS). The simplest way to do this is to GPS survey a local network station, using two existing and suitable NSRS monuments as GPS primary base stations.

Earlier in this chapter, instructions were given for accessing data sheets from the NSRS. Under "Radial Search," the circular-search radius can be set at 6.2 mi (10 km) for a 2-cm survey or 12.4 mi (20 km) for a 5-cm survey; then, select "GPS sites only." NGS data sheets will be listed for all NSRS monuments within the search area, so that the best can be selected for use as GPS base stations for surveying the local network station. If a suitable NSRS monument already exists in the project area, it is not necessary to establish a new local network station. If there are no suitable NSRS monuments within the search area, it may be necessary to follow other NGS-58 guidelines to establish primary base station(s) at the 2-cm or 5-cm level.

### NATIONAL COUNCIL OF EXAMINERS FOR ENGINEERING AND SURVEYING (NCEES) MODEL LAW

The National Council of Examiners for Engineering and Surveying (NCEES) regulates the practice of engineering and land surveying, provides for the licensure of qualified professionals, and defines terminology to include the definition of the "practice of surveying or land surveying."

The following is quoted from Section 2 (b)(5) of the NCEES Model Law:

(5) "Practice of Surveying or Land Surveying—The term "Practice of Surveying or Land Surveying," within the intent of this Act shall mean providing professional services such as consultation, investigation, testimony evaluation, expert technical testimony, planning, mapping, assembling, and interpreting reliable scientific measurements and information relative to the location, size, shape, or physical features of the Earth, and utilization and development of these facts and interpretation into an orderly survey map, plan, report, description, or project. The practice of surveying or land surveying includes, but is not limited to, any one or more of the following:

(a) Determining the configuration or contour of the earth's surface or the position of fixed objects thereon by measuring lines and angles and applying the principles of mathematics or photogrammetry.

(b) Performing geodetic surveying which includes surveying for determination of the size and shape of the earth utilizing angular and linear measurements through spatially oriented spherical geometry.

(c) Determining, by the use of principles of surveying, the position for any survey control (non-boundary) monument or reference point; or setting, resetting, or replacing any such monument or reference point.

(d) Creating, preparing, or modifying electronic or computerized data, including land information systems, and geographic information systems, relative to the performance

of the activities in the above described items (a) through (c).

(e) Locating, relocating, establishing, reestablishing, laying out, or retracing any property line or boundary of any tract of land or any road, right of way, easement, alignment, or elevation of any of the fixed works embraced within the practice of engineering.

(f) "Making any survey for the subdivision of any tract of land.

(g) Determining, by the use of principles of land surveying, the position for any survey monument or reference point; or setting, resetting, or replacing any such monument or reference point.

(h) Creating, preparing, or modifying electronic or computerized data, including land information systems, and geographic information systems, relative to the performance of the activities in the above described items (e) through (g).

Any person shall be construed to practice or offer to practice surveying or land surveying, within the meaning and intent of this Act, who engages in surveying or land surveying or who by verbal claim, sign, advertisement, letterhead, card, or in any other way represents themselves to be a professional surveyor or land surveyor or, through the use of some other title implies that they are able to perform, or who does perform any surveying or land surveying service or work or any other service designated by the practitioner which is recognized as surveying or land surveying.

The inclusion of photogrammetry, LIS and GIS within this definition is controversial. Some states have enacted the NCEES Model Law while others have not. At least one state has made it illegal for firms to submit bids where cost is used to select the winning firms, for risk of losing their professional licenses to operate within the state. It is extremely important for engineering, surveying and related firms to understand the laws for states where they plan to operate to insure that professional registration requirements are recognized and understood.

### REFERENCES

- American Society for Photogrammetry and Remote Sensing, *Manual of Photogrammetry*, 4th ed., American Society for Photogrammetry and Remote Sensing, Bethesda, Md., 1980.
- American Society for Photogrammetry and Remote Sensing, "ASPRS Accuracy Standards for Large-Scale Maps," *Photogrammetric Engineering and Remote Sensing*, Vol. 56, No. 7, pp. 1068–1070, American Society for Photogrammetry and Remote Sensing, Bethesda, Md., 1990.
- American Society for Photogrammetry and Remote Sensing, *Digital Photogrammetry: An Addendum to the Manual of Photogrammetry*, American Society for Photogrammetry and Remote Sensing, Bethesda, Md., 1996.
- American Society for Photogrammetry and Remote Sensing, 2001, *Digital Elevation Model Technologies and Applications: The DEM Users Manual*, American Society for Photogrammetry and Remote Sensing, Bethesda, Md., 2001.

Bureau of the Budget, *National Map Accuracy Standards*, Office of Management and Budget, Washington, D.C., 1947.

Federal Geographic Data Committee, *Geospatial Positioning Accuracy Standards, Part 1: Reporting Methodology*, Federal Geographic Data Committee, c/o U.S. Geological Survey, Reston, Va., 1998.

Federal Geographic Data Committee, *Geospatial Positioning Accuracy Standards, Part 2: Standards for Geodetic Networks*, Federal Geographic Data Committee, c/o U.S. Geological Survey, Reston, Va., 1998.

Federal Geographic Data Committee, *Geospatial Positioning Accuracy Standards, Part 3: National Standard for Spatial Data Accu-*

*racy*, Federal Geographic Data Committee, c/o U.S. Geological Survey, Reston, Va., 1998.

Federal Geographic Data Committee, *Content Standards for Framework Land Elevation Data*, Federal Geographic Data Committee, c/o U.S. Geological Survey, Reston, Va., 1999.

Kleusberg, Alfred, July 1992. "Precise Differential Positioning and Surveying," *GPS World*, pp 50.

National Oceanic and Atmospheric Administration, NOAA Technical Memorandum, NOS NGS-58, *Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm)*, Version 4.3, National Oceanic and Atmospheric Administration, Silver Spring, Md., 1997.



## CHAPTER 38

# TYPES OF DRAWINGS USED IN LAND DEVELOPMENT

*John Matusik, P.E.*

### INTRODUCTION

There are numerous types of maps and drawings, each with its own characteristics with regard to scale, orientation and purpose. With practice and exposure, one can recognize many of these characteristics by inspection and be able to read and create drawings effectively. For example, to the uninitiated a site plan may seem messy, unorganized and difficult to read. However, as one gains experience, reading the site plan becomes as simple as reading a newspaper.

Project design is an evolution of ideas presented in the form of maps and drawings. Design begins with worksheet drawings—a compilation of existing information amassed from other maps and drawings. The project evolves from generalized ideas scribbled on the worksheets into the high-detail, final-design drawings when project ideas are formalized. More specifically maps, drawings and specifications are the tools that:

- Help insure a viable design
- Show where and how a project will be built
- Show where and how a project was built
- Are used for estimating costs and materials
- Show whether the project conforms to ordinances and codes
- Become legal documents for the project

Maps and drawings can be as simple or as complex as necessary; it depends on what information needs to be conveyed and who is reading them. A well-developed drawing is neat and readable, but more importantly it is accurate

and clear. Some inaccuracies are caused by the transfer of data, distortion, and paper stretch. This need not be magnified by carelessness or laziness during design. Often during design, a scaled measurement is used to further the design.<sup>1</sup> Any inaccuracy in the drawing is then carried over to other design areas and can become critical during construction. In addition, the adequacy, accuracy, and clarity of the drawings avert arguments and possibly legal conflicts during the course of the project.

Maps and drawings use symbols, abbreviations and jargon specific to the industry. People associated with land development projects must be familiar with reading civil site drawings, structural drawings, architectural drawings and survey maps. Each of these has numerous variations of plan sheets and detail drawings. By being familiar with the types of drawings, the user can identify quickly the type of drawing that has the needed information. A construction drawing set for civil site work could have 20 to 30 sheets. The architect's set of drawings for the same project will likewise have as many sheets. Add this to the numerous mechanical and structural drawings associated with the project and the finding of specific information or cross checking of dimensions could become an ordeal to the uninitiated.

### MAPS

According to Manual No. 34 by the American Society of Civil Engineers, a map is "a representation on a plane sur-

<sup>1</sup> As a rule, scaling measurements from drawings is not recommended. If a scaled measurement is necessary, it should be used only if the inherent inaccuracy is within allowable limits for the measurement's intended use.

face, at an established scale, of the physical features (natural, artificial or both) or a part or the whole of the earth's surface by the use of signs and symbols, and with the method of orientation indicated." Maps depict the real world of three dimensions on two-dimensional media. Features shown on a map originate from data collected and compiled in various ways. This data has been simplified, manipulated, enhanced and even distorted so that the map is able to present the features in a usable manner. The accuracy of a map is determined by comparing the actual position of an object from field measurements to its position as shown on the map. This relative accuracy presumes there is no error in the field measurement. Several agencies and organizations have set standards for map accuracy: the Office of Management and Budget, The Reference Guide Outline, and American Congress for Surveying and Mapping (ACSM), to cite a few.

Maps are of two basic varieties: planimetric and topographic. A planimetric map shows horizontal positions of various features at the surface such as roads, buildings, trees, lakes, and rivers. Planimetric maps show only the features that are essential to the purpose of the map. Familiar examples of planimetric maps are street maps, political boundary maps and zoning maps. Maps with contour lines, referred to as topographic maps, show spatial variations as well as essential surface features. Vertical spatial variations appear as contour lines or as relief. The types of topographic maps used in the design process include the topographic maps from local public agencies or USGS quadrangle maps, as well as those created by surveys of the project site itself. Figure 38.1 compares a planimetric with a topographic map of the same vicinity.

### Creating a Map

The making of a map begins with smoothing out the shape of the Earth, and approximating the true irregular shape of the Earth with a mathematical model does this. Most often, this approximation is a sphere, spheroid, or ellipsoid. The approximated shape is reduced in scale and then transformed onto a flat surface. Three common methods for making this transformation, as shown in Figure 38.2, are by projecting the features of the globe onto a cylinder, cone, or surfaces. These surfaces are then unrolled, or developed, to make the flat, two-dimensional map.

The transformation results in the distortion of the features, such as landforms and continents. The transformation requires that either shape or size of the feature be maintained. Maps where the shape of the feature on the developable surface have the same area as the shape on the globe are known as equal-area maps. In maintaining equal areas, the shapes of the features are changed. For example, a circle on the globe might appear as an ellipse on the developable surface. Those maps where shape is maintained at the expense of size are conformal maps. A circle on the globe appears as a circle on the developable surface; how-

ever, there is no resemblance between the areas of the two shapes. For example, the area of Greenland is about  $\frac{1}{6}$ th the area of South America. However, on many conformal maps the two appear nearly equal in size.

### Lambert and Transverse Mercator Projections

Two types of map projections specifically related to land development projects are those used to develop state plane coordinate systems. These two projections are the Lambert conformal conic projection and the transverse Mercator projection.

Most state plane coordinate systems divide the state into zones—north-south zones or east-west zones depending on whether the state is elongated in the north-south or east-west direction. Those states elongated in the east-west direction use the Lambert conformal conic projection. This type of map projection, shown in Figure 38.3, wraps a conic section around the Earth's surface, which intersects the sphere at two specific locations. The circles within the sphere created at the intersection locations represent parallels of latitude, referred to as standard parallels. Along these parallels the scale is exact. Between the two standard parallels the scale is less than true. Outside the standard parallels the scale is greater than true scale.

The transverse Mercator projection of Figure 38.4 wraps a cylinder around the Earth such that the cylinder's axis lies in the plane of the equator. Along the two locations where the cylinder cuts the sphere, the scale is exact. Distortion between these locations causes the scale to be less than true scale. Outside the locations the scale is greater than true.

Map projections are created by mathematically projecting points from the sphere along radial lines from the Earth's center onto the projection surface of the imaginary cone or cylinder. The distortions in scale between or outside the parallel circles are evident from Figure 38.5. Outside the parallel circles lines  $ab$  and  $ef$  are longer than the actual surface represented by  $a'b$  and  $e'f'$  illustrating why the scale is greater than true in these sections. Similarly, line  $cd$  on the projection surface is shorter than line  $c'd'$  on the sphere of the Earth. Distortion can be kept within tolerable limits by proper selection of the location of the intersection of the sphere and projection surface.

### LEGENDS AND SCALE

Maps and drawings use combinations of symbols and variations in lines to represent specific features and data. The meaning of the symbols and line variations is provided on the map in the legend. It is helpful if the legend is located on the map itself, usually near a corner of the sheet. However, in a map or drawing set containing more than one sheet, the legend is usually shown on the first sheet.

### Bar Scales

The relationship between distances on the map and corresponding distance at the Earth's surface is the scale. Map

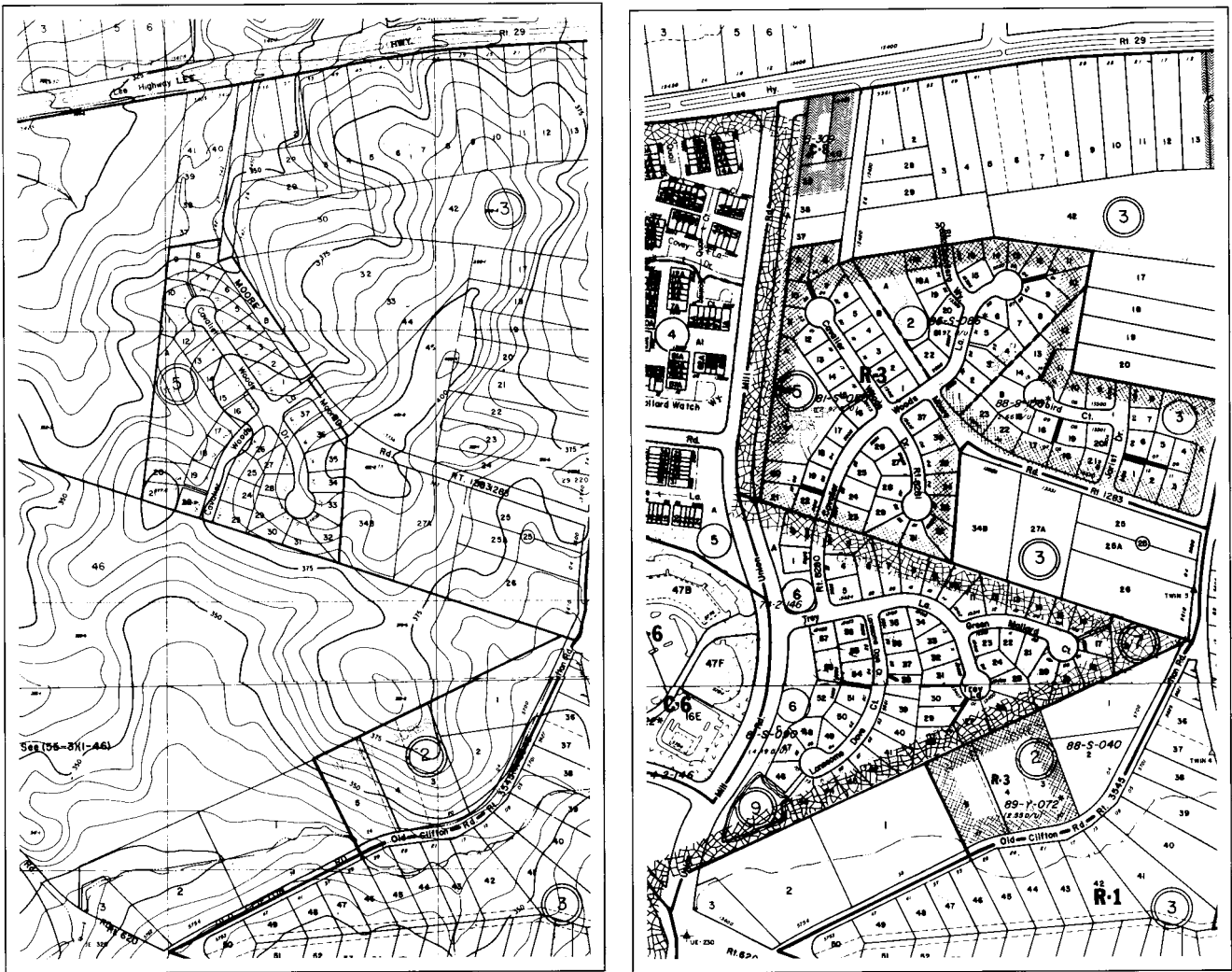


FIGURE 38.1 (a) Planimetric (zoning) map (b) topographic map.

scales are either graphic or numerical. The most common graphic method is the bar scale—a line with calibrated marks at intervals that represent distances on the map. For example, Figure 38.6 shows bar scales that might appear on a map for miles, feet, and kilometers. Although a bar scale is needed for each system of units, the advantage is evident when the map is reduced or enlarged. Although the map scale changes during reproduction, the bar scale shrinks or enlarges proportionally to the drawing.

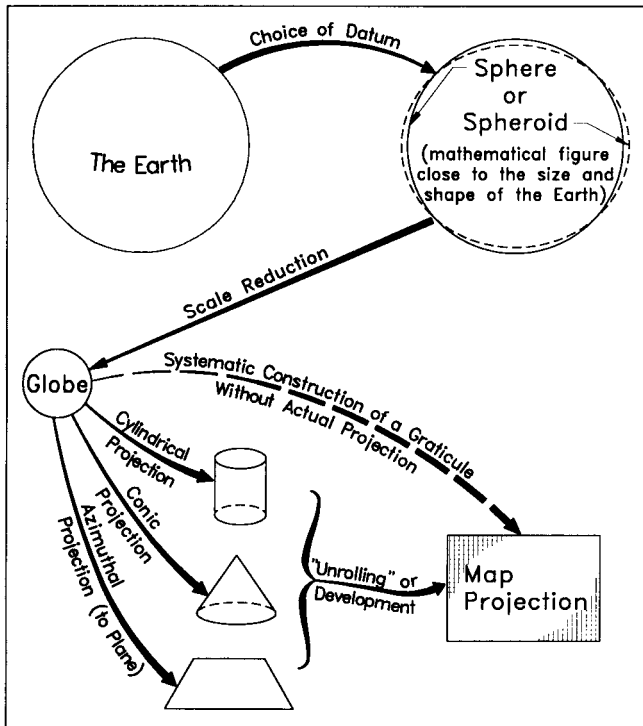
The numerical representation of a map scale is given as a ratio or representative fraction. The ratio represents a unit distance on the map to the corresponding number of equivalent units on the Earth's surface. For example, a ratio given as 1:30,000 means that one map unit represents 30,000 units on the Earth's surface. Therefore, a distance that measures 2 in. on the map is actually 60,000 in. (5000 ft) on the Earth's surface. Since the scale ratio is dimensionless, it applies to any system of units. This method is somewhat

cumbersome in that a conversion is necessary to change the larger dimension into something usable.

**Engineering and Architectural Scales**

The conversion is avoided by the use of another method for depicting scale. This method, perhaps the most popular for design drawings, associates a unit measurement on the drawing to a usable measurement at the Earth's surface. An example for this method shows scales as 1 in. = 50 ft or 1/4 in. = 1 ft. The first example is used typically on engineering drawings and is convenient for the type of tool used to measure distances. The 1 in. = 50 ft means that a measured distance of 1 in. on the plan represents 50 ft as measured on the Earth's surface. The latter, 1/4 in. = 1 ft example is typical of architectural drawings because of the way the architect's measuring tool is calibrated. Here a measured 1/4 in on the drawing represents 1 ft on the surface of the Earth.





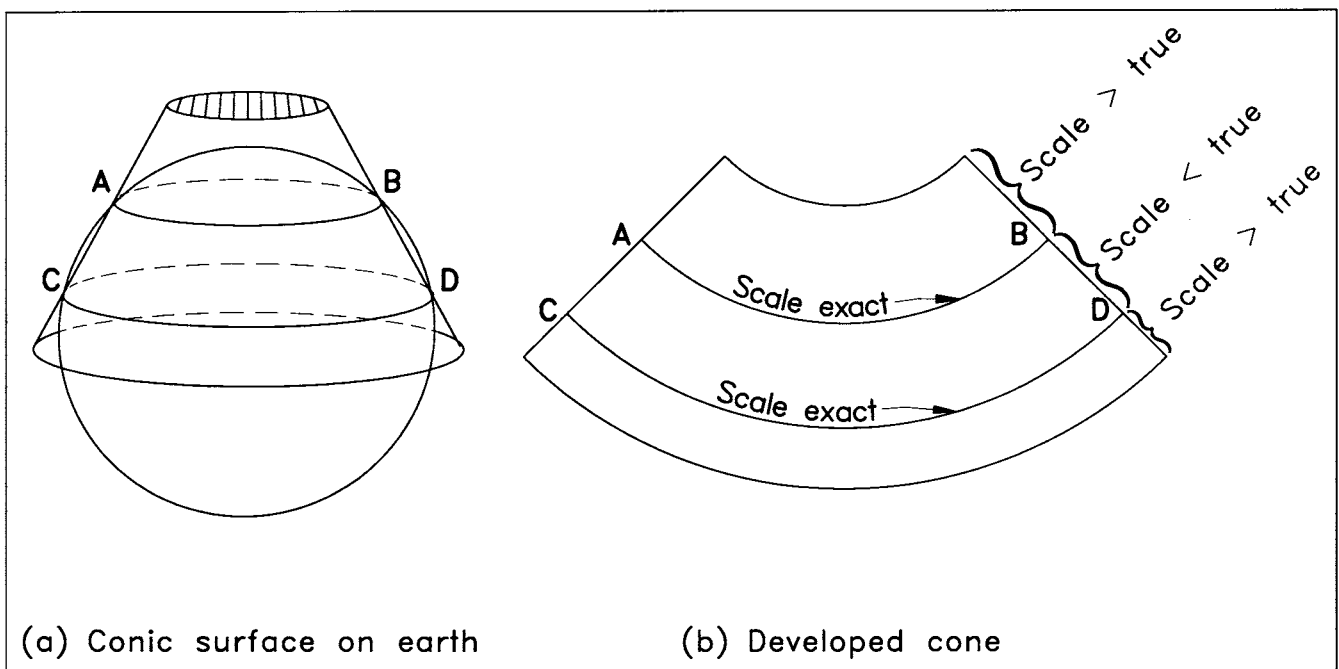
**FIGURE 38.2** Evolution of map projections. Reprinted from McDonnell, Jr., P.W. 1979, *Introduction to Map Projections*, by courtesy of Marcel Dekker, Inc., New York.

Another definition of scale is the tool that measures map distances, analogous to the 12 in. rule. The engineer's scale divides an actual inch into fractional parts representing feet and decimals of a foot. For example, the 30 scale divides an inch into 30 uniform divisions—each division repre-

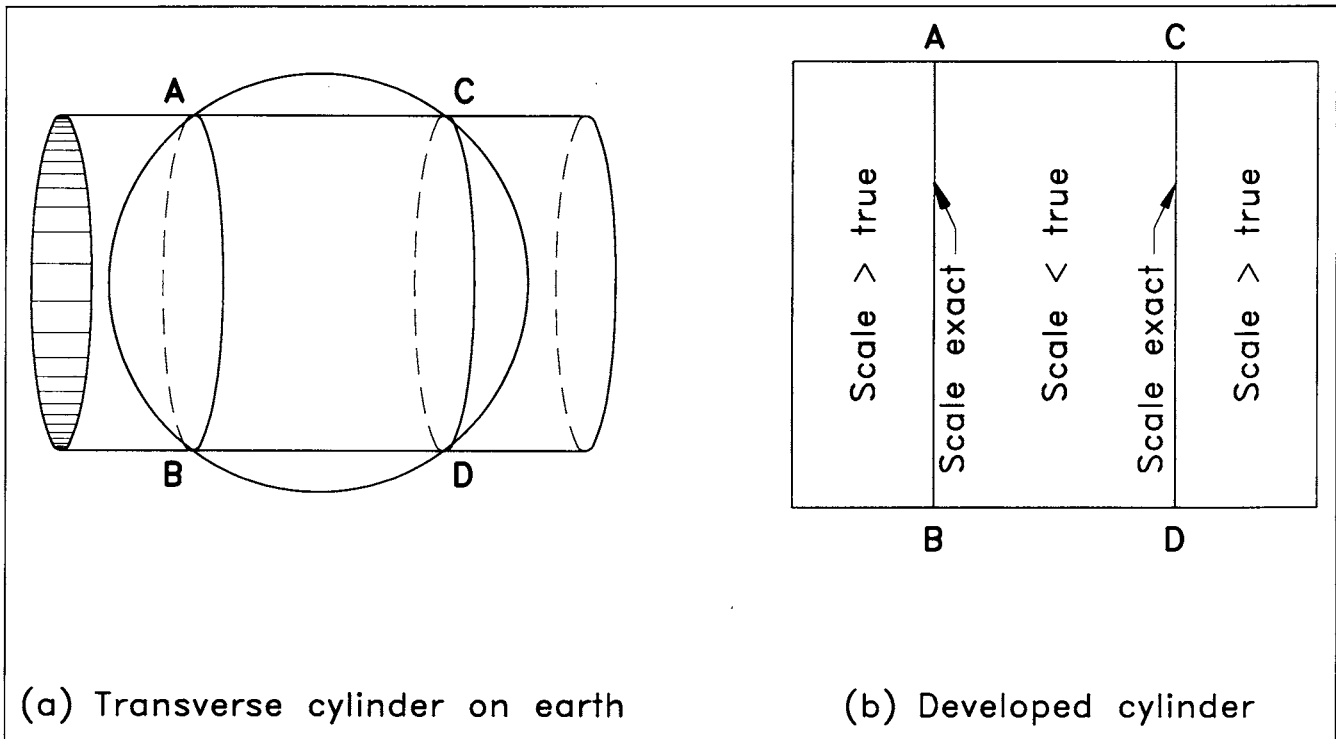
senting  $\frac{1}{30}$  of an inch. This translates to each division representing one foot when distances are measured on a scale drawing of 1 in. = 30 ft. The 30 scale also conveniently measures distances on drawings with scales of 1 in. = 3 ft, 1 in. = 300 ft, 1 in. = 3000 ft . . . etc. In which case each  $\frac{1}{30}$  division represents 0.1, 10, 100 ft, . . . etc., respectively. Engineers' scales commonly available are 10, 20, 25, 30, 40, 50 and 60. Each of these scales are appropriately subdivided to measure dimensions in decimals.

The architect's scale is based on customary units of inches and feet to be consistent with the system used by the building materials industry. A major portion of the scale is divided into one-foot increments with the ends of the scale outside the zero mark further graduated into inches and fractions of an inch. Each face of the scale has two different scales. One scale is read left to right and the other is read right to left. A number at each end identifies the appropriate scale. The number of subdivisions outside the zero mark depend on the space available for legible markings. The space available depends on the scale; therefore, the subdivisions for each scale represent different fractions of an inch. For example, the  $\frac{1}{8}$  scale has 6 markings each mark representing 2 in. The  $\frac{3}{4}$  scale has 24 markings, each mark representing  $\frac{1}{2}$  in. Typical architectural scales are available at  $\frac{3}{32}$ ,  $\frac{1}{8}$ ,  $\frac{3}{16}$ ,  $\frac{1}{4}$ ,  $\frac{3}{8}$ ,  $\frac{1}{2}$ ,  $\frac{3}{4}$ , 1, and  $1\frac{1}{2}$  in. A comparison of the two scales showing 4.5 ft is given in Figure 38.7.

The scale of a drawing is inversely proportional to its size. For the same coverage of area, increasing the scale decreases the size. For instance, a square 50-acre tract of land at a scale of 1 in. = 100 ft requires a approximately a 15 × 15-in. area. A larger scale drawing of this tract of



**FIGURE 38.3** Conic surface used in Lambert conformal conic projections.



**FIGURE 38.4** Transverse cylinder used for transverse Mercator projections.

land, say a scale of 1 in. = 30 ft, requires a minimum sheet size of 50 in. square. Decreasing the scale number enlarges the scale, and consequently the size of the drawing increases. An increase of the scale is actually a decrease in the size of the drawing but an increase of the scale number.

It is important in design that the features be drawn precisely to the scale selected for the drawing. Design drawings are used to estimate material quantities and earthwork. During some aspects of design, scaled distances from a plan are used as parameters for other aspects of design. Therefore, it is unequivocally imperative that precision in scale on a drawing be maintained.

**Selection of Scale**

The scale selected for a drawing depends on the type of drawing, the size of the drawing, the size of the sheet of paper and the desired level of detail. Many public agencies require minimum sheet sizes and scales for particular drawings. For design and analysis purposes, however, worksheet drawings and scales are selected for convenience of the task. Eventually, worksheet drawings are transformed into final drawings. Therefore the scale selected for the worksheets should be the same scale as that of the final drawings. This reduces design time and cost and lessens the chance of error in transferring information from the worksheets to the final drawings. This is the general practice for manual drafting. However, this practice is of little consequence if drawings are produced by digitized drafting methods.

The scale is selected such that the size of the drawing occupies nearly the full sheet, to provide as much clarity as

possible. For example, a 5-acre parcel at 100 scale is roughly 5 in. square. This scale is too small for a 24 × 36-in sheet of paper. A more appropriate scale might be 1 in. = 30 ft. On the other hand, a drawing should not be unreasonably enlarged just for the purpose of occupying a whole sheet. Several drawings can be shown on one sheet for those times when the scale is sufficient for the drawing and there is additional room on the sheet. For example, an adequate scale of 1/2 in. = 1 ft for a detail drawing of a foundation footing may require an area of 10 in. square. The additional area on a 24 × 36-in. sheet could be used for notes, other detail drawings or calculation charts. It is imperative to label clearly the drawing and the scale of each drawing in such circumstances. Some drawings (e.g. profile drawings—discussed later), use different scales for the horizontal and vertical directions. It is advisable to maintain consistent scales throughout a set of drawings to the extent possible. This can eliminate mistakes and confusion, especially when the drawings are taken to the jobsite where the luxuries of desk/layout space and drafting equipment may be scarce.

**DRAWINGS**

Project acceptance by the community and approval by public agencies can depend on an accurate graphical and pictorial presentation of the project. Surrounding residents of a project frequently perceive development as an intrusion and disruption to the status quo. The numerous questions by the concerned community generated by new development projects and the resulting anxiety can be addressed

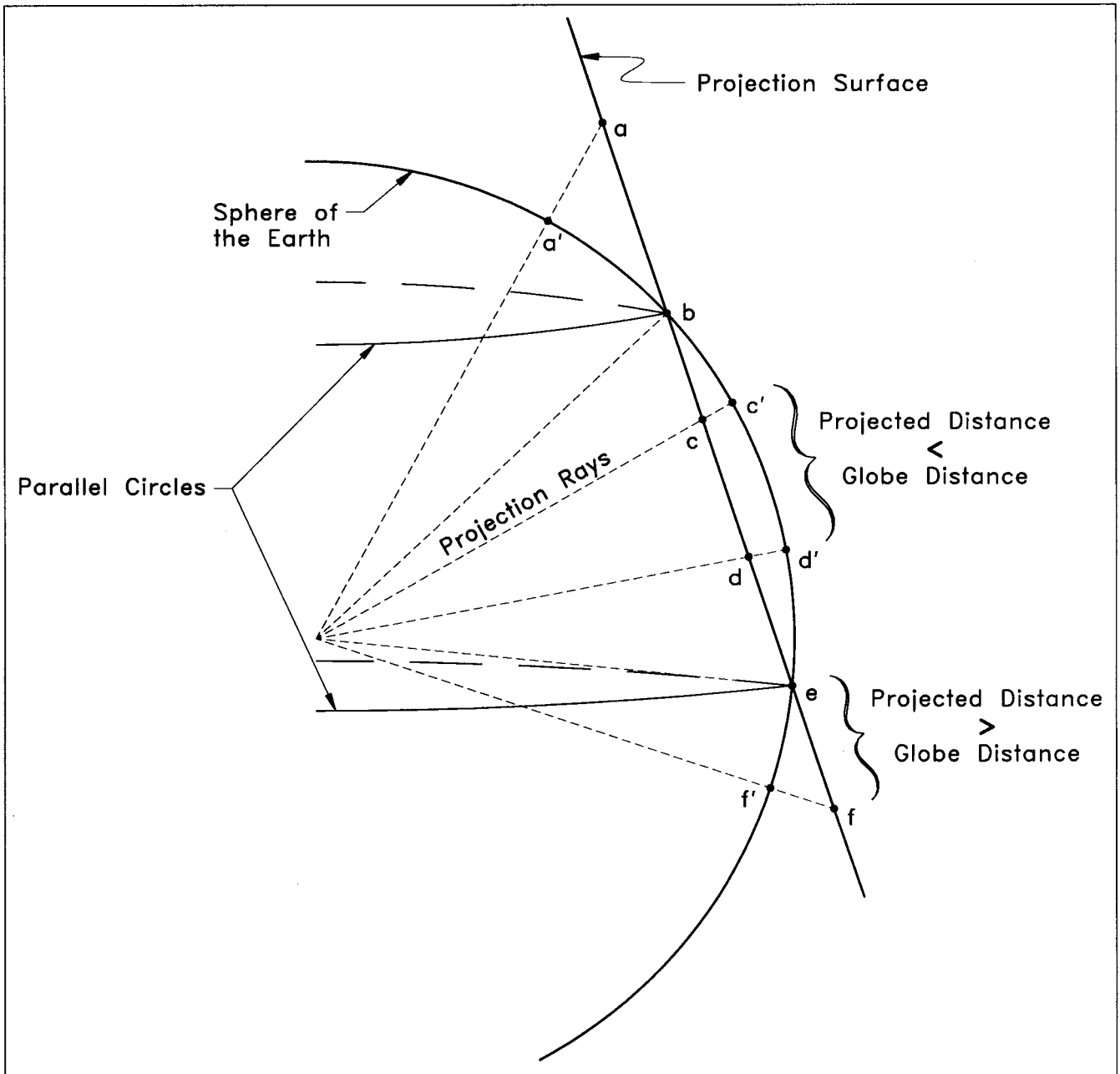


FIGURE 38.5 Distortions on projection surface.

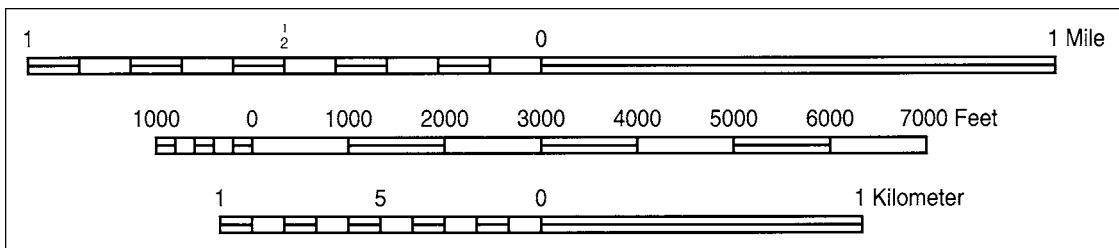
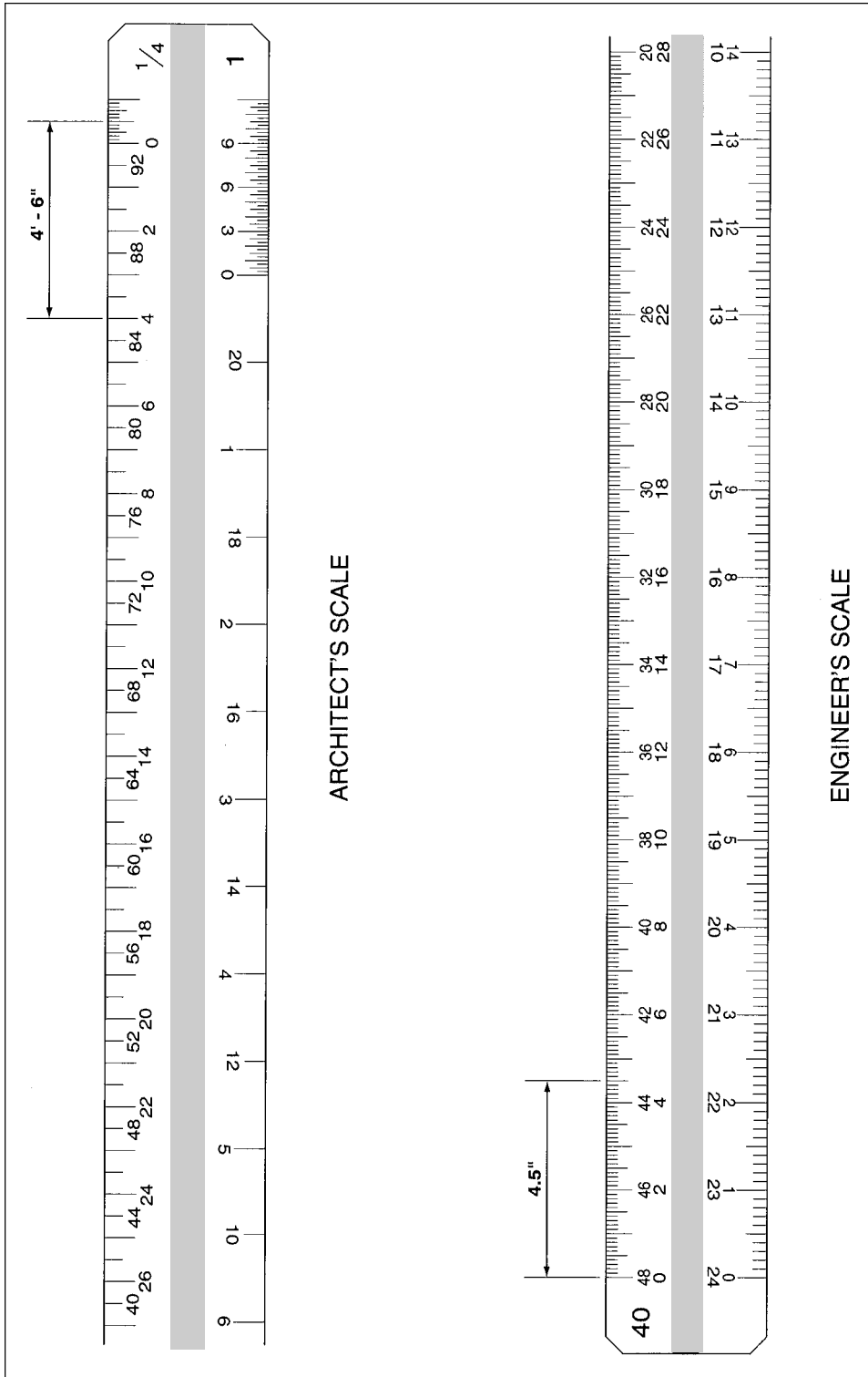


FIGURE 38.6 Bar scales.



**FIGURE 38.7** Engineers Scale and Architects Scale.

by using various types of drawings to show the effects of the project. Additionally, sufficiently detailed and accurate drawings (and specifications) helps insure that construction proceeds without delays, cost overruns, or legal actions.

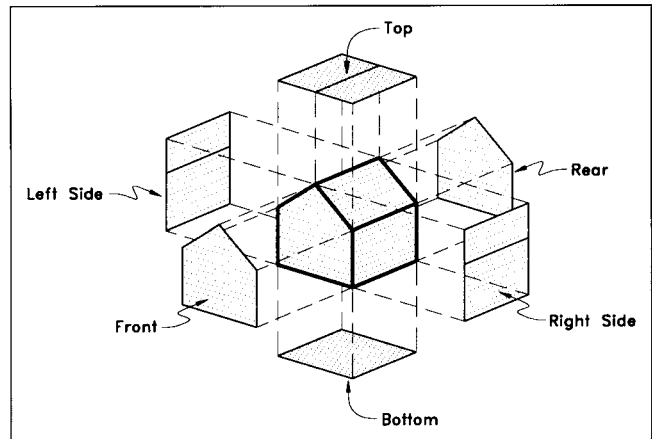
The numerous types of drawings used throughout the design process of a land development project include free-hand sketches, three-dimensional perspectives, and various types of projection drawings. During the various phases of design, these various types of drawings assist and record the progression of the project and illustrate the end product. The typical design process includes conceptual drawings, preliminary drawings, construction drawings and others.

Most of the drawings generated throughout the design are parallel-projection type drawings. Of the four types of projection drawings—multi-view, axonometric, oblique, and perspective—multi-view projection drawings are used most frequently for the civil site and some architectural drawings. Multi-view projection drawings, sometimes referred to as orthographic projection, do not have foreshortening or distortion of linear or angular measurement. The other three types of drawings, more frequently used by architects, landscape architects, and occasionally planners, distort the image in order to provide a more realistic picture.

**Orthographic Drawings (Multi-view Projection)**

An orthographic projection shows the details of an object on a plane passed through or in front of one of the faces of the object. The details project onto the plane by extending lines perpendicular to plane from the object to the plane. Orientation of the object to the plane is completely arbitrary and is determined by what or how the specific features should be shown. Any orthographic projection is only two dimensional. The simplicity of these types of drawings is the limiting of detail of only one face; the problem is that many people have difficulty conceptualizing the three-dimensional object from collectively observing the separate views.

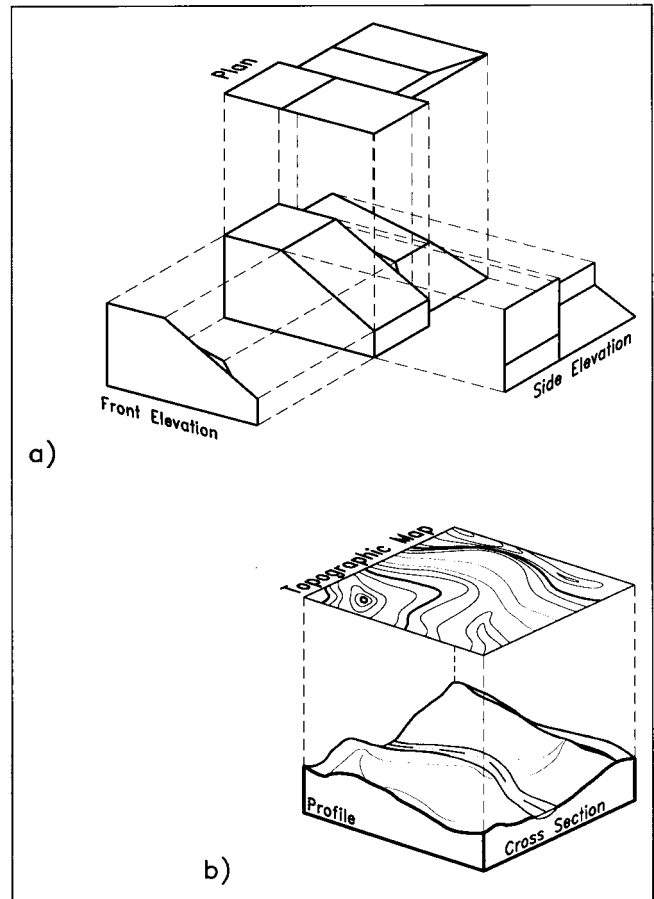
Specifically there are six views (in a rectangular coordinate system) of an object (see Figure 38.8). Each view represents the face of a cube. The six views are labeled: front, rear, top, bottom, left side, and right side. There is a correlation between these fundamental views and the common views of construction drawings. A plan view is analogous to the top view, a profile view compares to a front (or rear) view and the cross-section view compares to the left or right-side views. A plan view identifies spatial relationships in the horizontal plane by presenting the object as viewed from overhead. Profile views show the object longitudinally. They detail features of items where the length is greater than the width (e.g., streets, water mains, sanitary lines, and storm-sewer lines). Cross-section views, on the other hand, are oriented at right angles to the longitudinal direction. Street sections, channel/ditch sections, stream valleys and foundations are items frequently shown in section views.



**FIGURE 38.8** Six views of an orthographic scale.

These three views—plan, profile, and cross-section—are those used for most civil site drawings. The counterparts to these in the architectural profession are plan and elevation drawings. Figure 38.9 illustrates this comparison.

The profile and cross-section views are the result from passing a vertical plane through an object or feature. In



**FIGURE 38.9** Orthographic views for engineering and architecture.

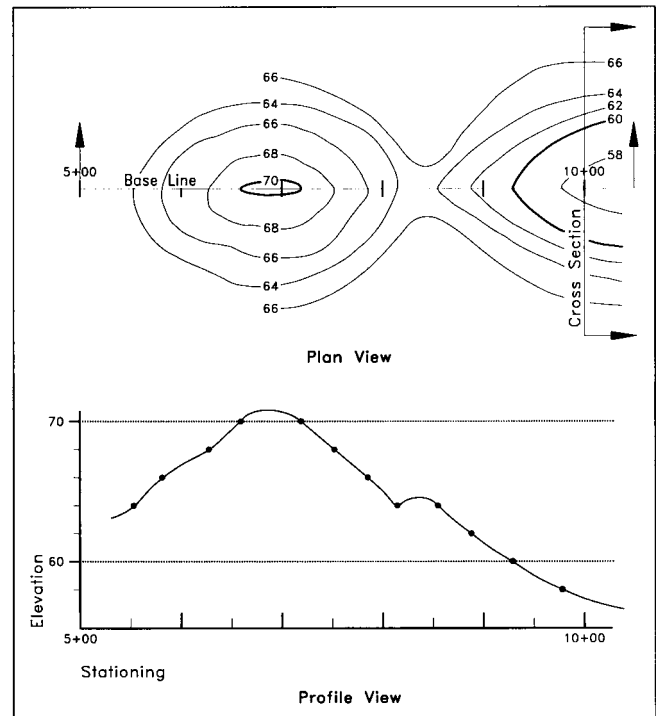
practice the vertical planes do not have to be straight. They can be angular or curvilinear if the situation warrants. The objects or features do not have to be cut by the vertical plane in order for them to appear in the view. Often the features lie behind the vertical plane and are projected onto the view. The location of the feature, whether it lies on or behind the vertical plane in one view, can be determined from observing that feature in other views.

**Plan Views.** These show the pertinent data as it would appear if viewed from directly overhead. As with any type of drawing, line weights and styles (e.g., dashed) help categorize similar elements and emphasize the important elements. For example, all proposed buildings might be shown with a thick heavy line. Tangible objects such as curb and gutter and buildings are shown, as well as intangible elements such as property lines, easement lines and, when appropriate, contour lines. The horizontal arrangement of the elements is shown to the correct scale.

Typically, a design phase of the project begins by drawing the plan view of base information. These initial sheets are referred to as base sheets. The information includes any existing features and elements that need to be shown and are pertinent to the design (e.g., property lines, existing roads and buildings, existing easements and utilities, and streams). Since this base information is what the design evolves from, it is highly imperative that this information be drawn accurately from reliable field information.

**Profile Views.** To establish the profile view, a base line or reference line is established in the plan view. The reference line might be the centerline of a road or the centerline of pipe for a water main, sanitary line, or storm-sewer line. In other cases the reference line might be edge of pavement, top of curb, or any other established reference line where the vertical relationships need to be examined. A reference system (e.g. stationing) along the reference line is shown in both plan and profile views. This helps in correlating a position on the reference line in the plan view with the profile view. Note that horizontal scale is different from the vertical scale in most profile drawings.

The horizontal axis of the profile view represents the selected-reference line. Although the reference line may curve and meander in the plan view, it appears straight in the profile view. The vertical axis of the profile view represents elevations. For example, existing and proposed ground elevations are plotted in the profile by locating the reference-line station in the plan view where contour lines cross the reference line. This elevation is then plotted at the appropriate station in the profile view. After the ground-elevation points are plotted in the profile view, they are connected by dashed lines to depict existing grade and solid lines for proposed grade. Additionally, underground utilities and other existing features that cross the reference line, or come very close to it, should be plotted and identified in the profile view. Such items will affect the design. Figure 38.10 shows how the profile is developed from the plan view. However, the two views are not necessarily aligned



**FIGURE 38.10** Plan and profile view.

such that the viewer can project elements directly from one view to the other as is the case in Figure 38.10.

**Cross-Section Views.** Often in the design of roads, large conveyance channels, and floodplain studies, cross sections become a necessary part of the design drawing set, either by ordinance requirement for approval or to facilitate the engineering design itself. Similar to profiles, cross sections also show vertical relationships of existing and design features except that they are oriented perpendicular (or nearly perpendicular) to the reference line. Their purpose is to show the vertical relationships of objects and design details that cannot be shown effectively in other views.

Cross-section views (see Figure 38.11) are plotted in a manner similar to profile views. The location of each section is determined in the plan view. Cross-section lines typically intersect the reference line at right angles. However, the situation does arise in which features can be better illustrated if the cross-section line is skewed with the reference line. At appropriate scales, the sections are plotted with the horizontal axis representing the perpendicular distance relative to the reference line and the vertical axis as elevations. The reference-line location is identified on each section by the stationing. From the plan view, the distances from the reference line to points of known elevations on the section line are determined. This same distance is measured from the reference line in the section, and the elevation is plotted. After all elevations are plotted, the points are connected and the lines appropriately labeled as existing or proposed when required for clarity.

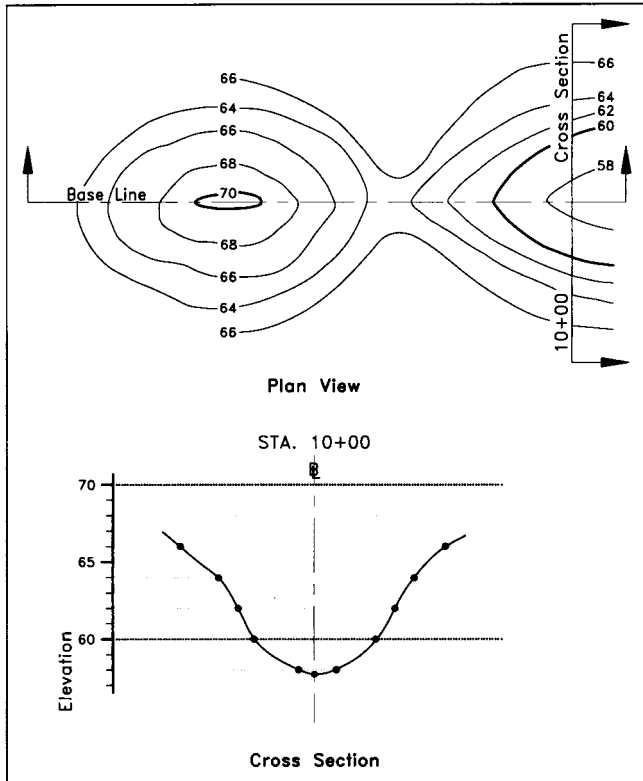


FIGURE 38.11 Cross-section view.

A common pitfall when plotting or reading sections is the location on the section relative to the reference line. Positions left or right of the reference line depend on which direction of the reference line one is facing. If the reference line runs north-south, for instance, and the stationing increases going north, then the left side of the reference line is west. This is typical for road and highway sections. The left side is the side as determined from facing the direction of increasing stationing. Although the selection left or right vis-a-vis ahead or behind stationing is purely arbitrary, the cross sections must be consistent, and indication must be given on the plans to alert the reader to the orientation.

When several cross sections appear on the same sheet, the reference line of each section should be aligned with the preceding section. As shown in Figure 38.12, this enhances the appearance of the design sheet and helps in visualizing the continuity of the sections.

Spacing of cross sections depends on their purpose. Typically in road design, cross sections are spaced every 50 or 100 ft. Channel and stream-valley cross sections are located where there is significant change in the topography to warrant a cross section, although there is usually a prescribed maximum distance set by local agencies.

The purpose of profile and section views is to show the vertical relationship of objects. Because the longitudinal distance for the object is so long, the vertical relationship cannot be shown effectively if the same scale is used in the vertical and longitudinal direction. Very often, profile views

and cross section-views use a larger scale in the vertical direction to exaggerate and accentuate these vertical relationships. As a result, take-off quantities may be better served from cross-section views rather than profile views. A comparison of an unexaggerated profile view with the exaggerated view is shown in Figure 38.13.

**Pictorial Projection Drawings**

A pictorial drawing shows the object in three-dimensional form. This type of drawing conveys design concepts in a non-technical manner. Three types of pictorial projection drawings are: perspective, axonometric and oblique. Each type differs on how the object is projected onto the projection plane. Figure 38.14 shows the projection plane for the three types of projection drawings and the multi-view drawing.

**Perspective.** Of the three types of pictorial drawings, the perspective drawing is the most realistic in that it shows the object as perceived by the human eye and eliminates optical distortion produced by other pictorial-projection drawings. Parallel lines converge to vanishing points (V.P.) in the drawing as they recede from the observer. This gives the illusion of depth. Perspective drawings may have one or several vanishing point (see Figure 38.15).

**Axonometric.** The axonometric drawing is a projection drawing where the projection lines are parallel to each other and perpendicular to the projection plane. The axonometric drawing is a view of the object such that it is turned and slightly inclined to the projection plane. The drawing is transferred to the projection plane with parallel projection lines perpendicular to the projection plane. The three edges of the object that meet at the corner nearest the observer are the axonometric axes. Axonometric drawings are classified as: isometric if the angles of the axonometric axes are equal; diametric if two angles are equal; and trimetric if the three angles are unequal. Figure 38.16 illustrates these different types of axonometric drawings. Scale can be determined along each of the three axonometric axes. All lines parallel to each axis will be to the scale of that axis.

**Oblique.** The reference axes for an oblique drawing are horizontal, vertical and depth, similar to the axonometric drawing. For elevation obliques, the object is oriented with one of the faces parallel to the projection plane. This makes the angle between the horizontal and vertical reference axes 90°. The other two angles are arbitrarily selected at the convenience of the drawing. Typically, the angle between the horizontal axes and the depth axes is 30°, 45° or 60°, the standard angles of drafting triangles. Plan obliques are depicted with a high angle of view, with either a horizontal or vertical plane receiving more emphasis than other planes.

The projection lines from the object are all parallel but make an oblique angle with the projection plane. Oblique drawings that use full size height, width and depth are known as cavalier oblique drawings. These drawings portray the object as having an exaggerated depth. To give the object a visually-correct appearance, the cabinet oblique

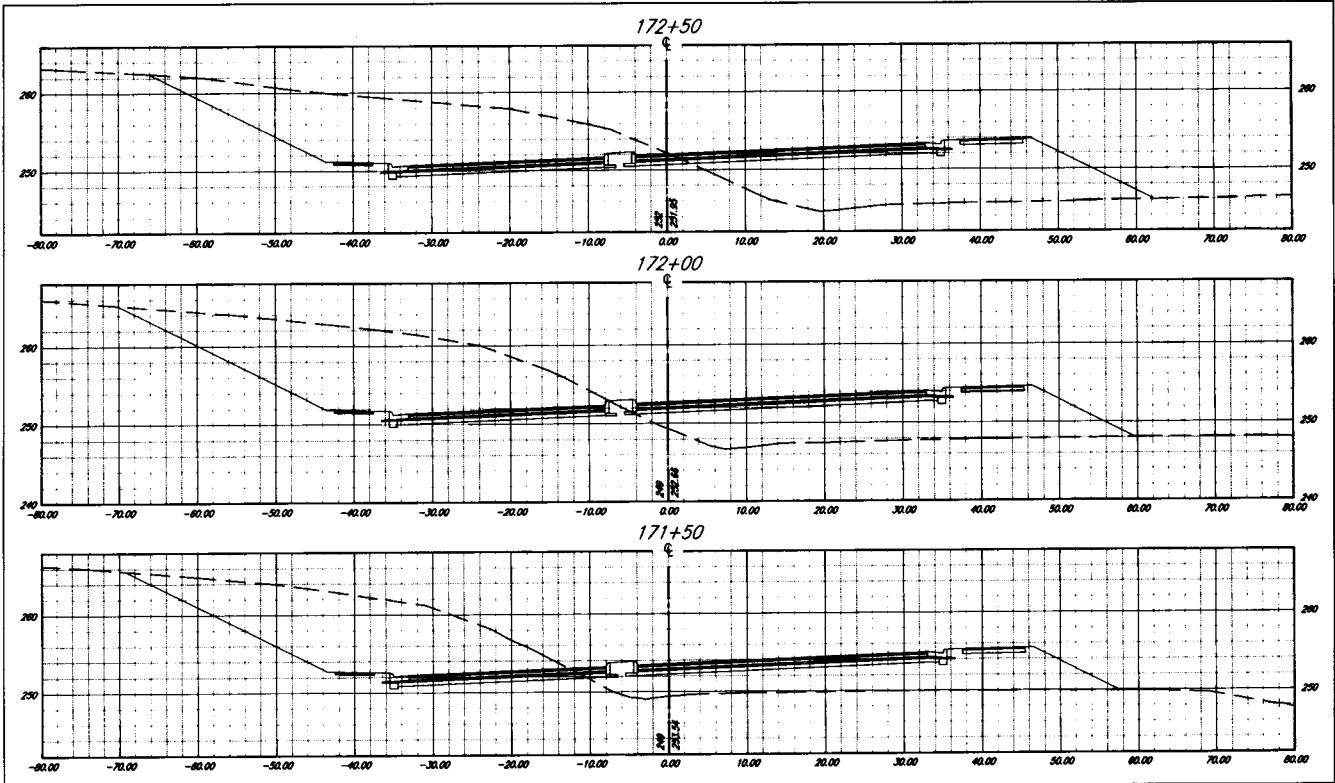


FIGURE 38.12 Cross sections with reference line aligned.

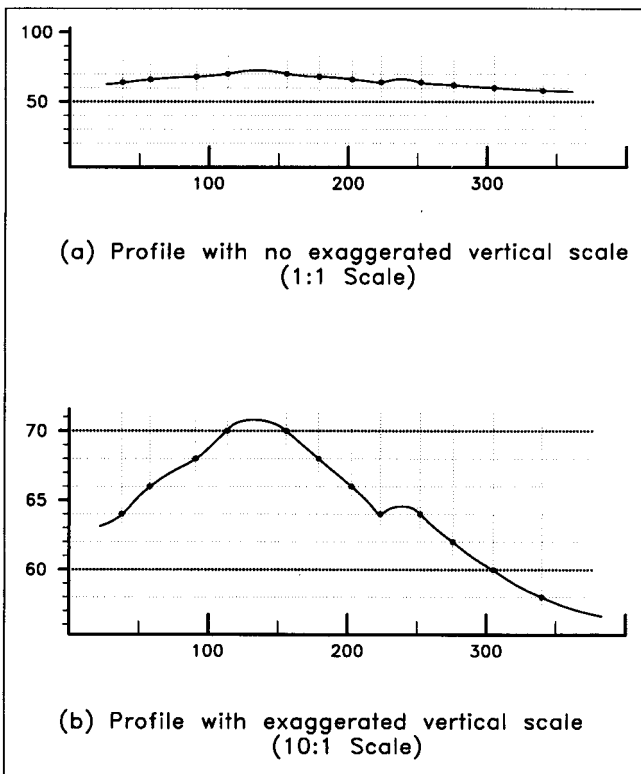


FIGURE 38.13 Profile view: (a) 1:1 scale (b) exaggerated scale.

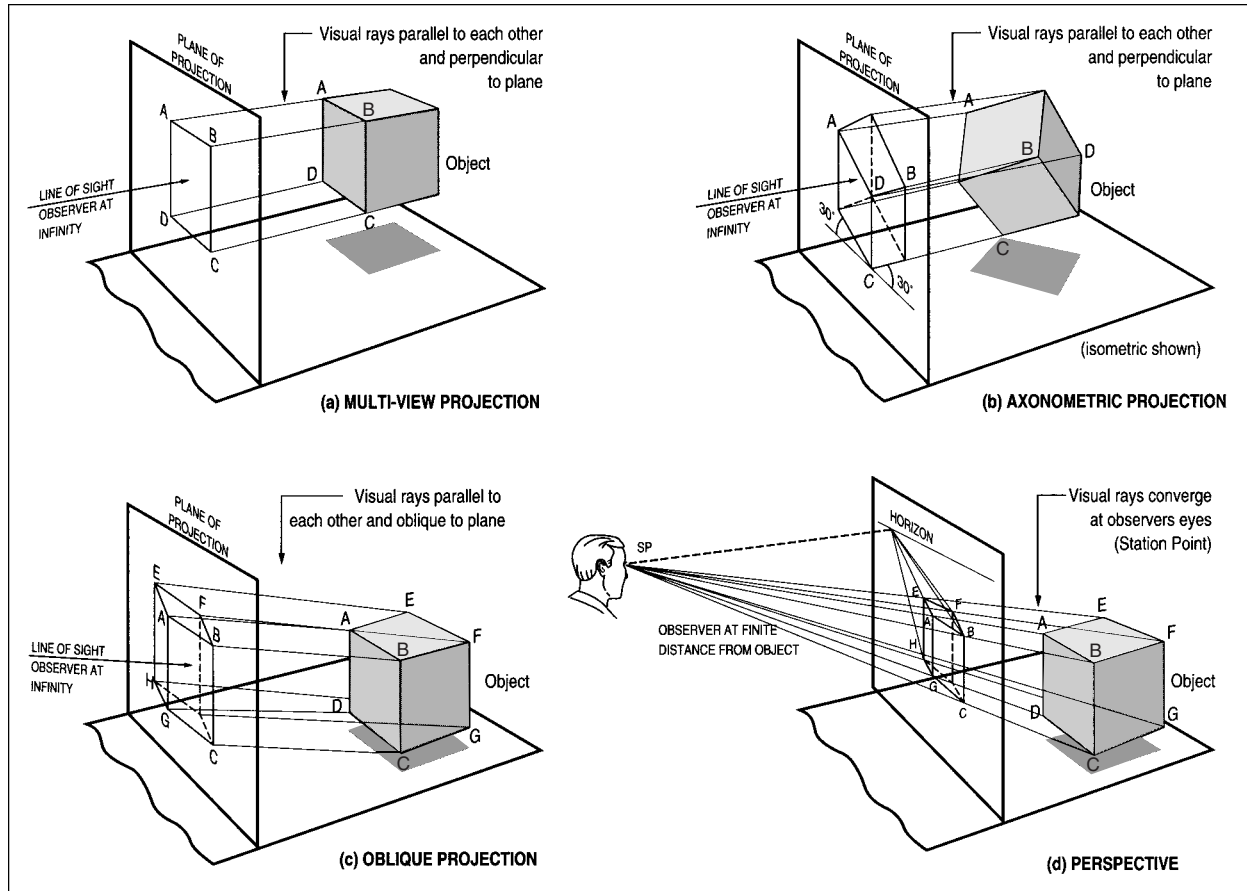
drawing shortens the depth dimension to  $\frac{1}{2}$  of the width and height dimension (see Figure 38.17).

### Microcomputer Graphics

The microcomputer has revolutionized design and drafting. Solutions to design problems and creation design drawings are done almost concurrently with the aid of microcomputers and software application programs. Producing drawings at various scales, viewing the designed product from any perspective in three dimensions and color design drawings essentially can be had at the touch of a button. Revisions to the design and to the drawings can be performed very quickly and efficiently. One distinct advantage of using a computer for design is the ability to generate customized drawings. For example, a drawing showing only proposed roads and existing topography at one scale can be generated, and in a short time another drawing can be generated showing lot lines, sanitary sewer and waterline at a different scale—that is, once the data has been entered into the drawing file.

Microcomputer graphics can be used to link information among the various disciplines. For example, surveying field data can be downloaded to a microcomputer file, which can then be accessed for engineering-design purposes. Conversely, data generated from engineering design can be transferred to surveying equipment to make for easier field measurements. The sharing of information translates to savings throughout the design process.





**FIGURE 38.14** Projection plane for orthographic, axonometric, oblique, and perspective drawings. (Reprinted from Frederick E. Giesecke, 1980, *Technical Drawing*, 7th ed. Reprinted by permission of Pearson Education, Inc., Upper Saddle River, NJ.)

The design and the drawing is only as good as the person who operates the machine, the available input data, as well as the limitations of the software program. Despite the dramatic effects of the microcomputer, one realization frequently overlooked is the over-emphasis placed on the reliability of the output. For the most part, the output from most software programs is accurate and reliable. However, the designer should not blindly accept output design just because a computer generated the solution. A familiarity with the software and its limitations is essential to produce a good design. When a designer's intuition results in a suspicious feeling about a computer-generated design, it is best to follow that intuition and investigate the solution until an acceptable comfort level is attained. Don't be reluctant to challenge the output from a computer. Computers have not changed the nature of the design process, only the efficiency and productivity.

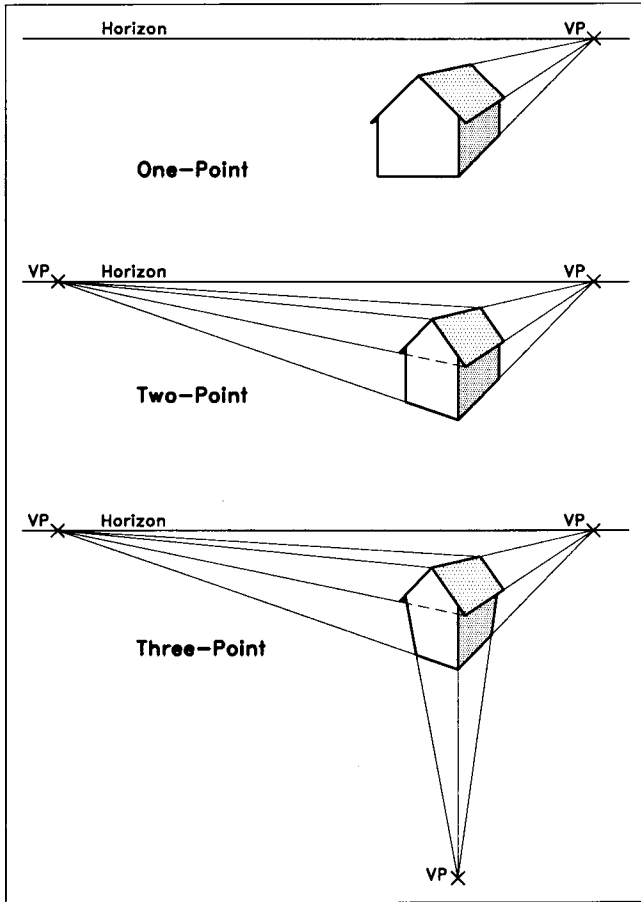
**TYPES OF DRAWINGS USED IN LAND DESIGN**

**Project Planning**

During initial stages of the design process, a small-scale plan view of the project area with the property lines, existing features and topography is used as the base to begin

planning the project. The location and identification of the features are taken from existing records rather than field-run surveys. At the very early planning stages, initial layout of the site is a broad-brush generalization that "roughs-in" the major streets, residential areas, commercial areas, recreational areas, open space areas and other major ideas about the design. Residential and commercial areas are located with only free-hand circles, streets have no geometric smoothness, and ideas for proposed land use are labeled in the general vicinity of their impact. Such an abstract and symbolic drawing is commonly referred to as a bubble diagram or blob diagram (see Figure 38.18).

As the project evolves, the bubble diagram is further refined to what is referred to as the concept plan or concept diagram. The identified residential areas and commercial areas show building location, entrances, parking areas, open space areas, ponds and other amenities. Major roads are refined to a reasonable geometric relationship. Realizing that the building has, in all likelihood, not been designed at this stage, the building footprints shown on the conceptual diagram are only representations of potential building configurations. Parking areas are drawn to within a reasonable area that accounts for the estimated number of re-



**FIGURE 38.15** Example of one-point, two-point and three-point perspective drawing.

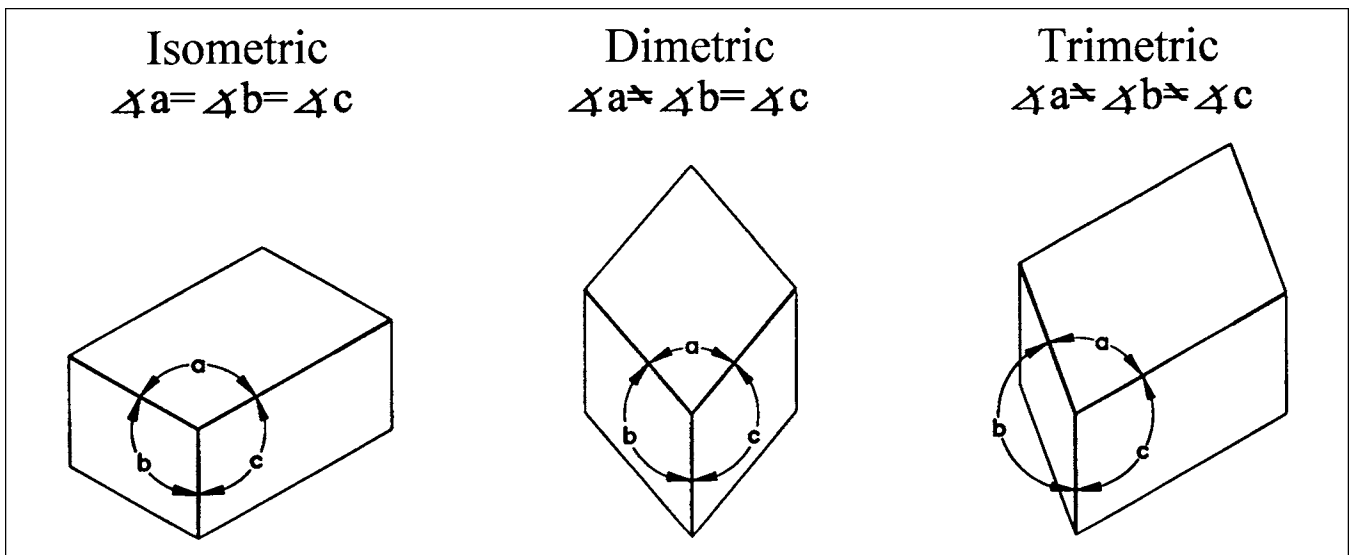
quired parking spaces. On large projects, the conceptual plan shows the project broken down into different phases and the order the phases will be developed. Figure 38.19 is the concept plan for the bubble diagram of Figure 38.18.

The conceptual plan is then modified into separate site-analysis diagrams that illustrate specific concepts about the site. Such diagrams may show views from different directions away from the site, solar orientation, hydrologic and hydraulic issues, or how the site appears from the road or entrance during various stages of completion. Some site-analysis diagrams are sometimes prepared before the concept plan is started. These diagrams evaluate the site for specific existing conditions.

Preliminary-layout drawings typically follow as the next design phase begins. These drawings take the conceptual diagrams to the next level of detail. Lot lines are drawn such that each lot size and configuration conforms to zoning regulations, streets are drawn with proper geometric characteristics that conform to design speed and traffic loads (e.g., minimum widths and minimum radii), preliminary grades for design are indicated by spot elevations, etc.

During early stages of the project, members of the design team frequently meet with civic groups, neighborhood citizens, staff of local planning agencies, environmentalists and other people not adept at reading technical drawings. In order to relay project-design ideas, they prepare various types of presentation drawings. These drawings are enhanced versions of the plan view or three-dimensional perspective drawings sometimes shaded and colored to convey project ideas more clearly. Occasionally they are enlarged for presentation purposes.

An illustrative plan is a plan view of the project that is colored or shaded to depict open space, parking, streams, lakes and other site characteristics. Buildings, trees, lot lines, and other physical features are illustrated to embellish



**FIGURE 38.16** Three types of axonometric projections.

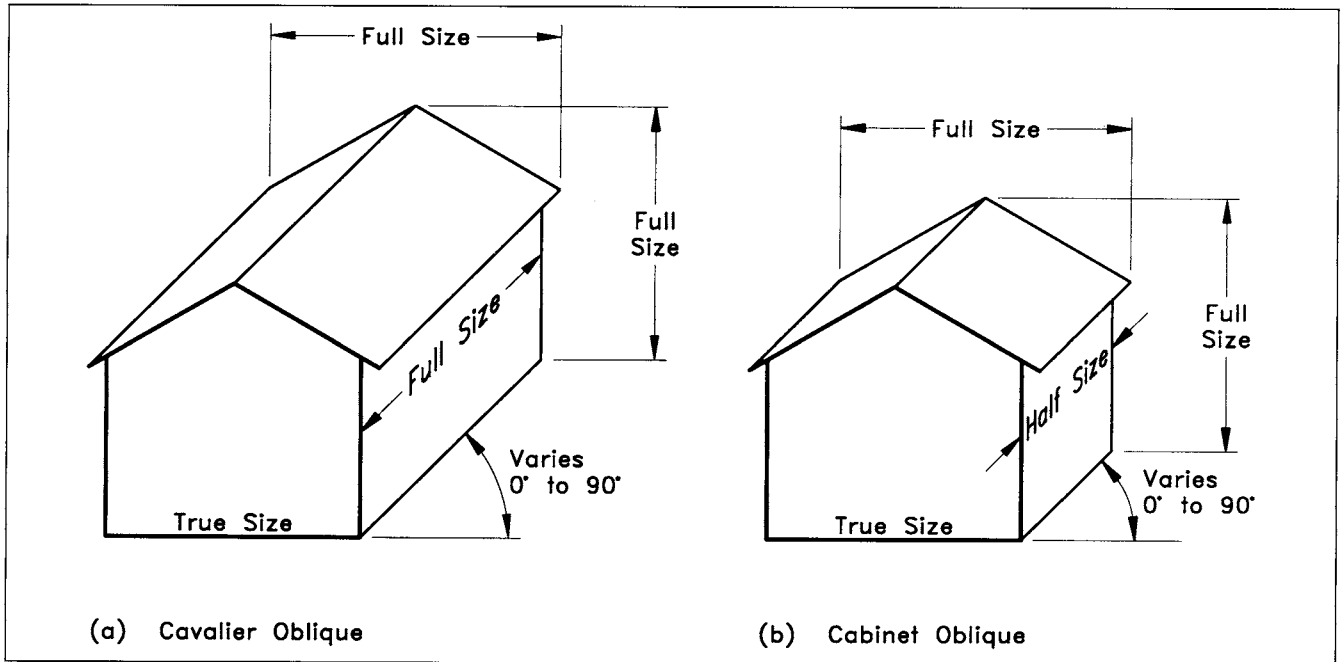


FIGURE 38.17 Cavalier and cabinet oblique drawings.

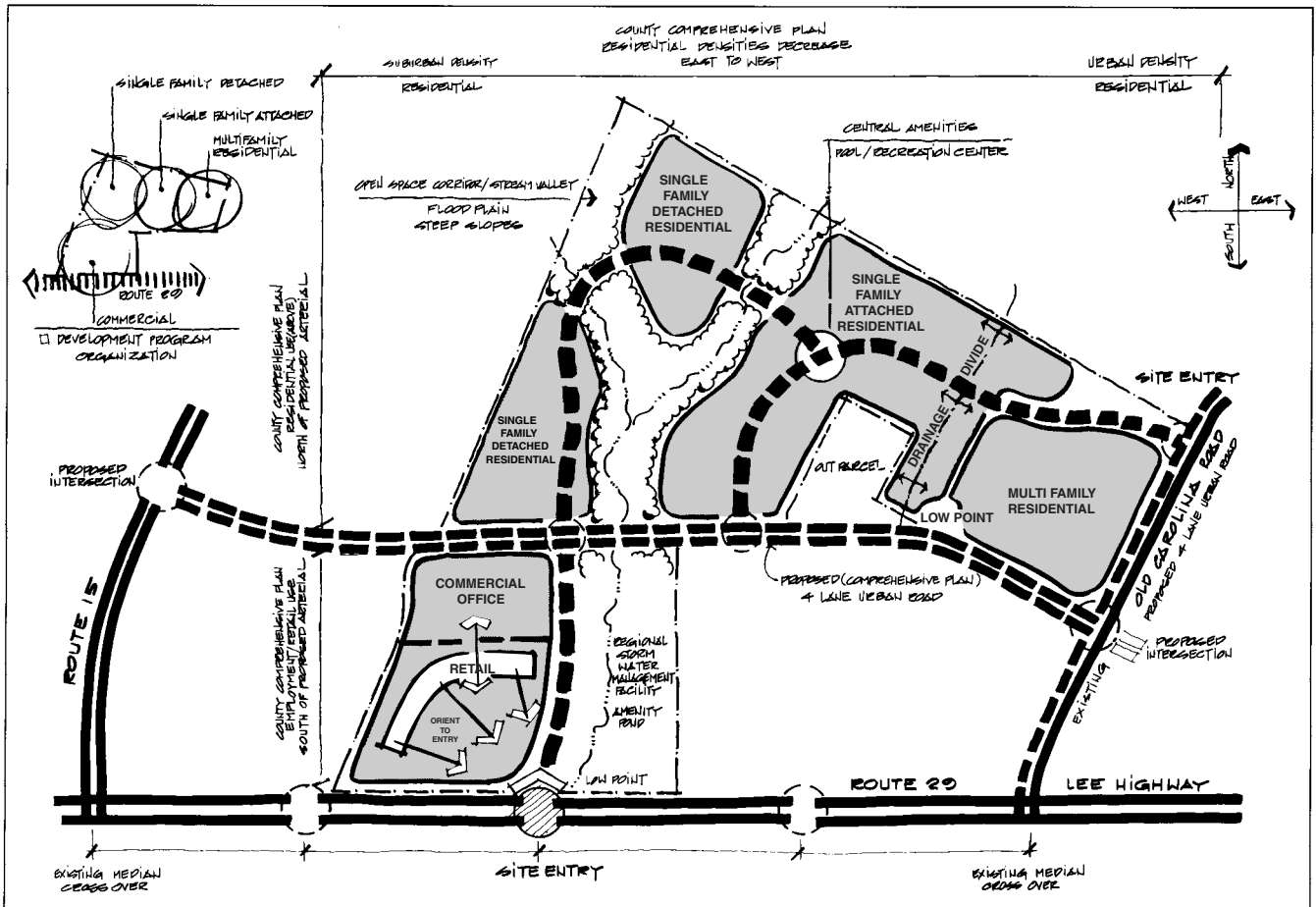


FIGURE 38.18 Bubble diagram.

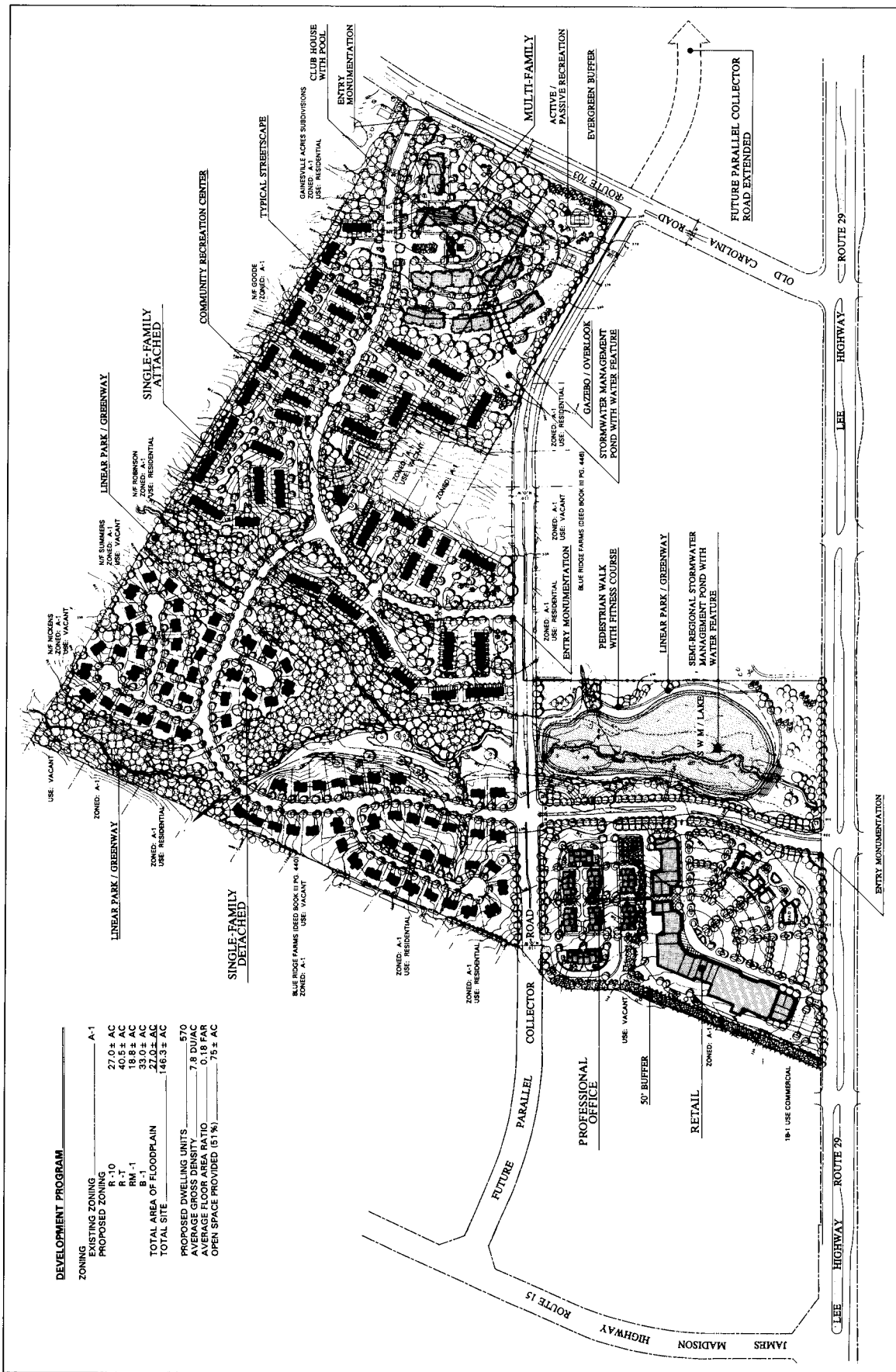


FIGURE 38.19 Concept plan.

the drawing. Perspective drawings of the site, colored and shaded (i.e., renderings), also help the uninitiated visualize the project. Often, one type of drawing may serve several purposes. Figure 38.19 is also an illustrative-type drawing.

### Civil Site Drawings

The civil site drawings are the technical drawings created by engineers that show site improvements such as building location and grading, road design, utilities and storm-sewer network. Many of these drawing become the construction drawings when sealed by the engineer and approved by the appropriate public agencies. Typically, the civil site drawings use orthographic-type drawings to show the site as it exists and the finished project. In rare circumstances, the civil site drawings may include a pictorial type of drawing. On complex projects, drawings may be included to show intermediate stages of the project. For example, on a site where massive earthwork is required there may be several grading plans showing the grading at intermediate stages.

Location of existing and proposed structures such as buildings, streets, and utilities, size and type of materials, existing and proposed grades, appropriate details and computation analysis are all common elements in the full drawing set. Although the drawings' main purpose is to direct the construction, they are also used for quantity take-offs and bidding purposes. Submission of the completed set to various public agencies is mandatory in most localities. Public agencies such as department of transportation, department of public works, water authority and utility companies, fire marshall's office, and others all must approve the design of their respective disciplines. Until such complete approval is obtained, site-construction activities cannot begin. The review and approval by these public agencies is necessary to insure the design conforms to applicable codes and standards. (See Chapter 19, The Plan Submittal, Review and Approval Process, for further discussions.)

A typical construction-drawing set includes:

1. *A cover sheet*—this contains general notes regarding general site/project conditions, vicinity map, sheet index, legend for symbols used in the plan set, block areas for public agency approval stamps and signatures, the name and address of client/developer, and name of the project manager.
2. *Grading plan/site plan*—this is a plan view showing horizontal location of the existing and proposed buildings, roads, utilities, parking areas, storm-sewer network, and existing and proposed contours.
3. *Plan and profile sheets*—these are drawing sheets with part of the sheet showing a plan view and part showing a profile view of the road, utility, storm-drain section etc. Depending on the nature of the project, a plan and profile view could show several facilities on the same sheet. For example, a length of street with water, sanitary, and storm sewer could all be shown on the same plan and profile view. In other cases, such as

a sanitary-sewer outfall running through tracts of land, the plan and profile sheets would show only the sanitary line. Various examples of a plan and profile drawing can be seen throughout the book.

4. *Sediment and erosion-control plan*—this identifies the size, location, and types of various structural and vegetative measures that are used throughout the duration of the project to control erosion and sediment. Typically, this sheet also contains a narrative. Drainage divides are depicted on this sheet to support the erosion and sediment-design decisions.
5. *Calculation sheet*—these sheets show the storm-sewer design calculations relating to inlet and pipe sizing. Calculation sheets also show stage-storage and stage-discharge curves for storm-water management facilities or other calculations for the benefit of the review process and to be used by other design professionals.
6. *Various detail sheets*—most designs use local and national standards for construction. Occasionally no standard exists or the specifics of the site preclude the use of an existing standard. In such cases, the engineer composes his own detail drawings.
7. *Landscaping plan and detail sheet*—these drawings locate and identify the types of trees, shrubs, and other vegetation for the project.
8. *Record plats/easement plats*—although these drawings are not typically thought of as part of the construction set, they may be a required part of the submission set to the public agencies. These plats show the subdivision of the original tract, right-of-way acquisitions and dedications, and the granting of certain property rights (e.g., easements).

The number of drawing sheets in a construction set varies according to the size of the project, the scales selected for the drawings and the requirements of the public agencies. The foregoing sheets are the more common sheets; other drawing sheets are included as the particular project warrants. The project engineer is responsible to see that the set contains all sheets necessary for communicating proper information to the contractors and public agencies. Insufficient or inadequate drawings will delay the approval process and cause headaches for the builder, and this occasionally results in backcharges to the engineer.

### Survey Drawings

Frequently, survey drawings and maps are referred to as plats. Boundary plats, easement plats and subdivision plats are a few types of survey drawings encountered in land development projects. Additionally, surveyors are responsible for topographic maps and as-built plans, sometimes called record drawings. Most survey drawings are a form of a plan view.

Boundary lines, lot lines, easements, etc. at the subdivision level are identified with bearings and distances. In the public-land system, a lot in a subdivision is referenced eventually to the township in which it is located. The public-land system partitions land according to latitudes and longitudes of the Earth. Base latitudes and longitude lines were established. From there, land was blocked off into 6-mile square rectangles. Each township was further divided into 1-mile square rectangles called sections. Sections were divided into halves, quarters, and quarter sections. The intent of establishing this system was to provide a simple method for real property identification to facilitate the apportionment of government lands to private ownership beginning in the early nineteenth century. Thirty-two states use the public-land survey system. The remaining 20 states<sup>2</sup> rely on metes and bounds, lot and block for land descriptions. With the public-land system, any lot can be traced backwards to the township.

Early property boundaries in the “metes and bounds states” were identified by referencing physical features of the property. That is, the property lines were identified as “from the fence post to the oak tree to the pine tree stump to the edge of the swamp.” A surveyor surveyed the property, and, based on the metes and bounds description, established property lines with bearings and distances.

Boundary plats (see Figure 38.20) establish the property lines for the project. The plat shows the monuments set and found, and the bearings and distances for each line. Existing easements or the potential for existing easements is shown. Although the boundary is essential for planning design, often the planning design begins using only plotted-deed descriptions to expedite the design process. When the boundary survey is complete, the design worksheets are updated to the newly-established boundary. For the purposes of conceptual and preliminary site analysis, a plotted-deed boundary is usually adequate as long as the worksheet boundary is revised to reflect currently-established property lines.

Topographical maps are also critical to preliminary design. Again because of timing and the nature of the land development process, topographical surveys are completed later in the process after some of the planning phase has already begun. During preliminary design, topography maps from existing sources suffice. During final design, more accurate topographic information is an absolute necessity.

The subdivision plat (see Figure 38.21) delineates the lot lines with bearings and distances and areas for each lot. Street dedications, rights-of-way, and easements are included on the subdivision plat. This plat is eventually recorded in the land records of the municipality.

Easement plats typically are required for all permanent off-site easements. On some projects, utilities must be ex-

tended to the project site by traversing through private property. The success of the project is determined by the acquisition of the necessary easements to access the existing utilities. The easements permit access by appropriate personnel for construction and maintenance.

Construction drawings typically use dashed lines for existing features on the site and solid lines for proposed features. Different line weights (i.e., thicknesses) depict different categories of the features to make the drawings more readable. Survey plats do not follow this practice. Dashed lines on survey plats are indicative of existing features as well as other items. For example, dashed lines are frequently used for easements (existing and proposed).

### *Shop Drawings*

Inevitably, design drawings cannot cover all construction details. In circumstances where site-specific details are necessary, the contractor supplements the design drawings with shop drawings (sometimes referred to as detail drawings). These drawings convey details necessary to manufacture specific components for the project or detail construction techniques for a site-specific purpose. These shop drawings may have to be approved by the local municipal agency and/or the project engineer. In most cases, the engineer's approval is only to insure the drawings meet contract specifications. The engineer assumes no responsibility for their accuracy, unless the contract states otherwise.

### **Architectural Drawings**

Architectural drawings are used extensively in the land development industry. They basically show the interior and exterior design specific to the building. Planners and engineers coordinate with the architect in order to merge the site design with the building design. Confirmation by the planner/engineer is recommended as to the architect's intentions for grades and elevations of the site and location of the buildings. Site grades, building location and orientation as visualized by the architect may not work because of constraints relating to street and utility location and grades, as well as land acquisition for easements and rights-of-way.

Because of the nature of the design process, plans and drawings change quite frequently. An open communication line between the architect and planner/engineer is essential to minimize conflicts between the architectural design and site design. A change of even 1 ft in elevation on a building or the addition of a window or door changes the site-grading scheme and could severely affect the site design. Therefore, when obtaining information from architectural plans (or from any plans outside the site engineer's office), it is prudent to verify that the information is current.<sup>3</sup>

Architectural drawings for residential dwellings include the following types of drawings: floor plans, framing plans,

<sup>2</sup> Maine, New Hampshire, Vermont, Massachusetts, Connecticut, Rhode Island, New York, New Jersey, Pennsylvania, Maryland, Delaware, Virginia, Kentucky, West Virginia, North and South Carolina, Tennessee, Georgia, Texas, and Hawaii

<sup>3</sup> Some architectural/engineering design firms are so dispersed that it is wise to verify plans when everything has been designed in-house.

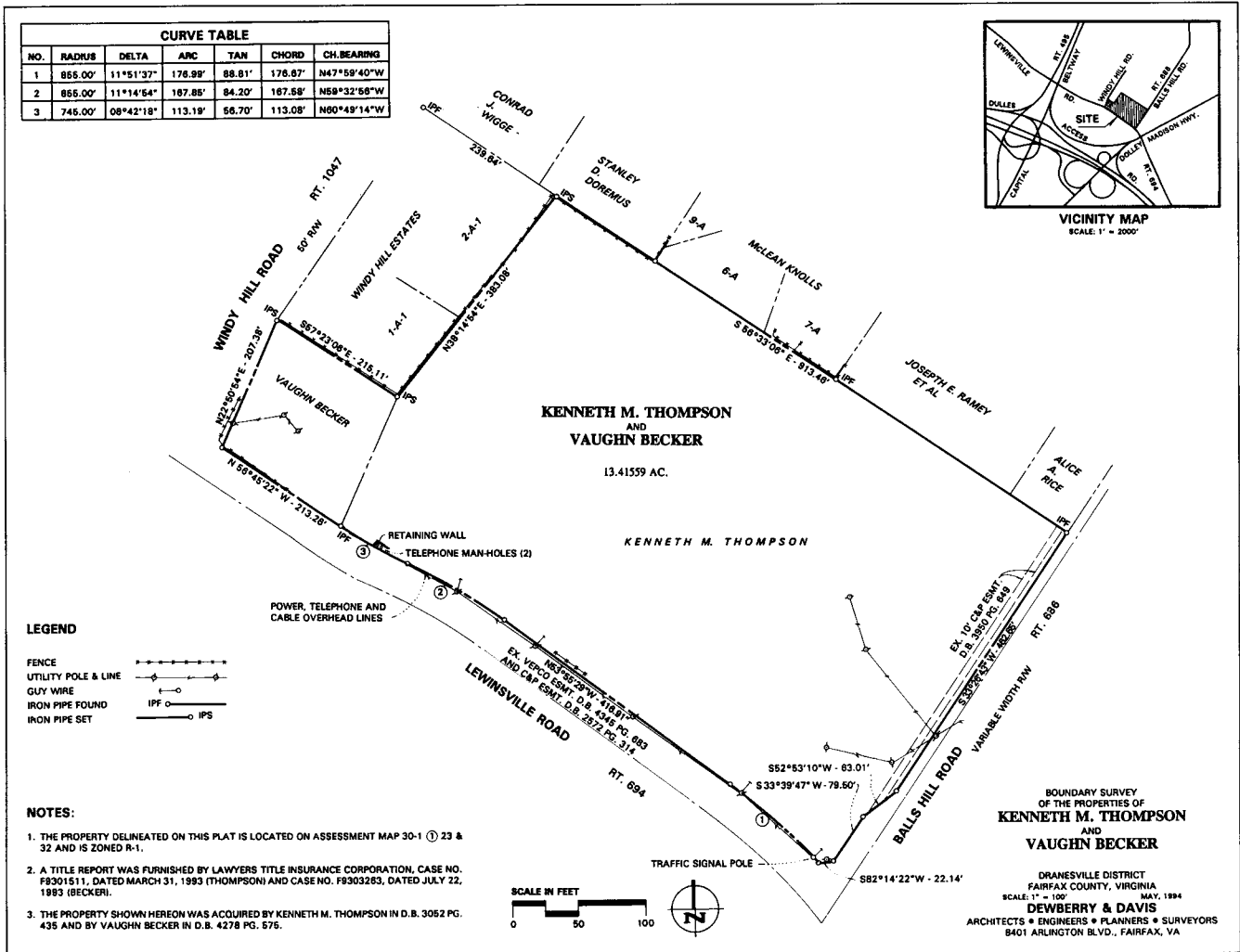


FIGURE 38.20 Boundary plat. Reprinted by permission of Uniwest Construction, Inc.

roof-framing plans, heating and cooling plans, plumbing plans, electrical plans and pictorial presentations. Floor plans are section views that depict items of interest such as the location and dimensions of exterior walls, windows, doors, garages, walks, decks, stoops and patios. Other features shown and dimensioned on the floor plans include interior walls, mechanical equipment and plumbing fixtures. As shown in Figure 38.22, the floor plan is a horizontal sectional view. A floor plan showing typical details and dimensions is shown in Figure 38.23. Typically, there is a floor plan for each floor of the building.

Elevation views (Figure 39.24) show exterior features of one side of the building. The elevation shows the finished appearance of the building with vertical dimensions. Grade lines, windows, doors, and exterior materials are evident from the elevation views. When establishing the grading plan, floor plans and elevation views are referenced frequently for information. An example of a wall section is shown in Figure 38.25.

Another architectural drawing helpful in site design is wall sections. Wall sections, a type of detail view, shows

floor to floor dimensions, exterior grades, foundations, footings, and overhangs, and other features that impact site design.

In commercial projects, other professionals design many of the plans. For example, mechanical engineers and electrical engineers perform the heating and cooling plans and electrical design. During the site-design process, the planners and engineers refer to these plan sets to locate utility service connections and dimensions for construction stake-out surveys.

### MAPS FREQUENTLY USED IN LAND DEVELOPMENT PROJECTS

There are numerous sources for obtaining maps that provide useful information for a project. It is good practice to obtain many of the maps at the outset of the project. They will be invaluable during the course of the project. Additionally, these maps can be kept on file to serve future projects.

The United States Geologic Survey (USGS) is a major source of various maps. The USGS publishes a series of

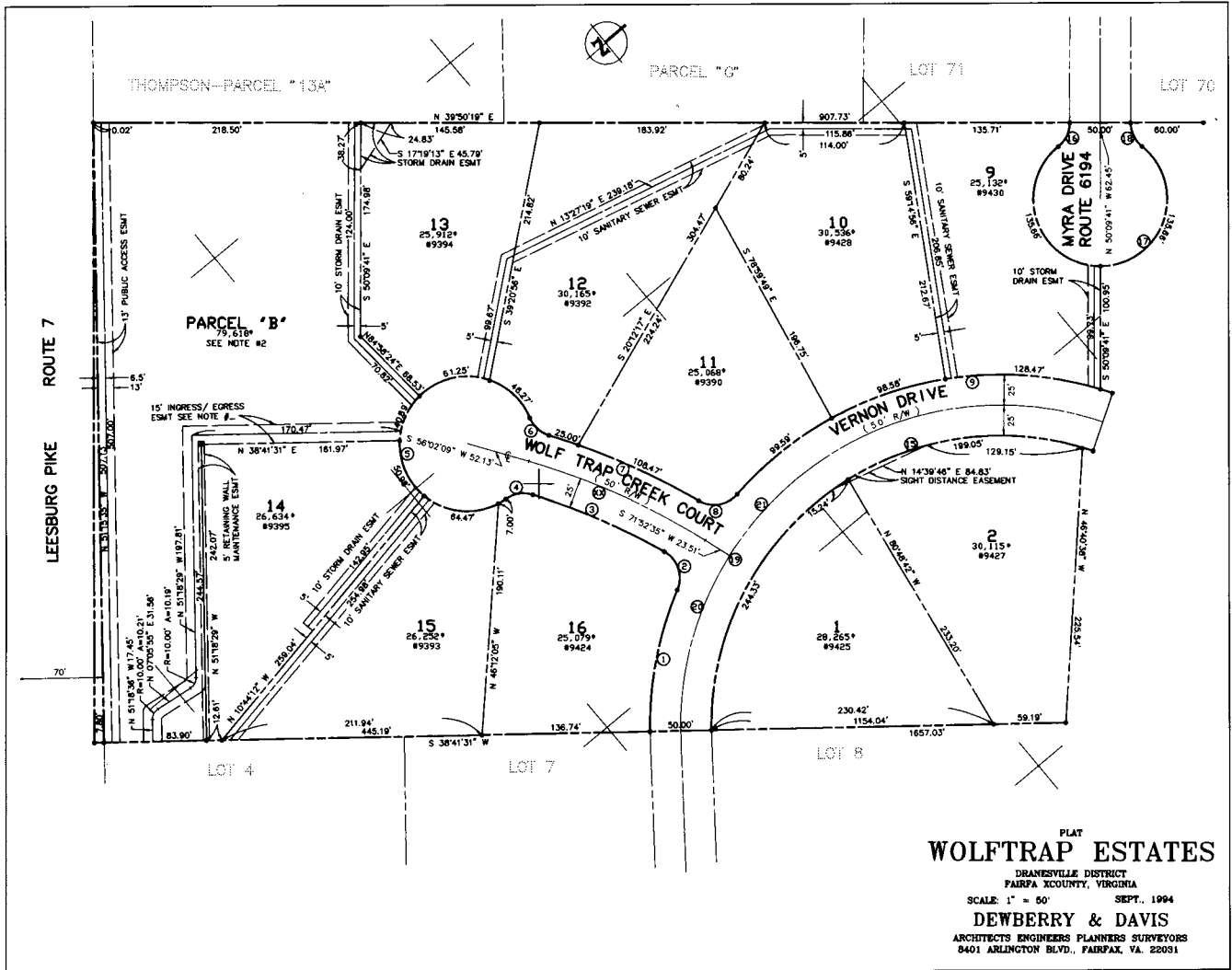


FIGURE 38.21 Subdivision plat.

topographic quadrangle maps, which cover the United States, its territories and possessions. These maps, The National Topographic Map Series, are available in various scales, depending on the area of coverage. Areas bounded by 7.5' of latitude and 7.5' of longitude (referred to as 7.5-

minute maps), 15' maps, and 1 degree maps are available for most areas of the U.S.

Other types of maps available from the USGS include geologic maps, mineral resource maps, water resource maps, land use and land cover maps, and both high and

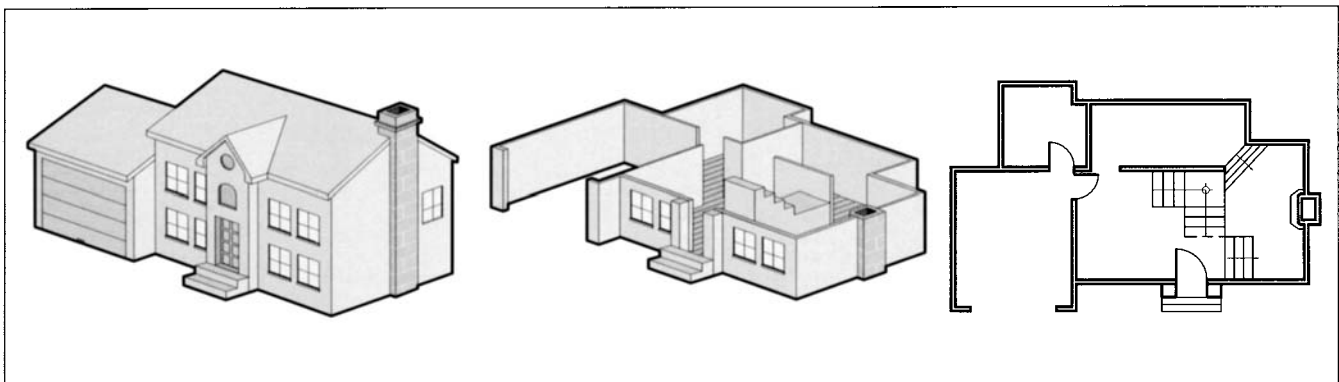


FIGURE 38.22 Creating a floor plan.



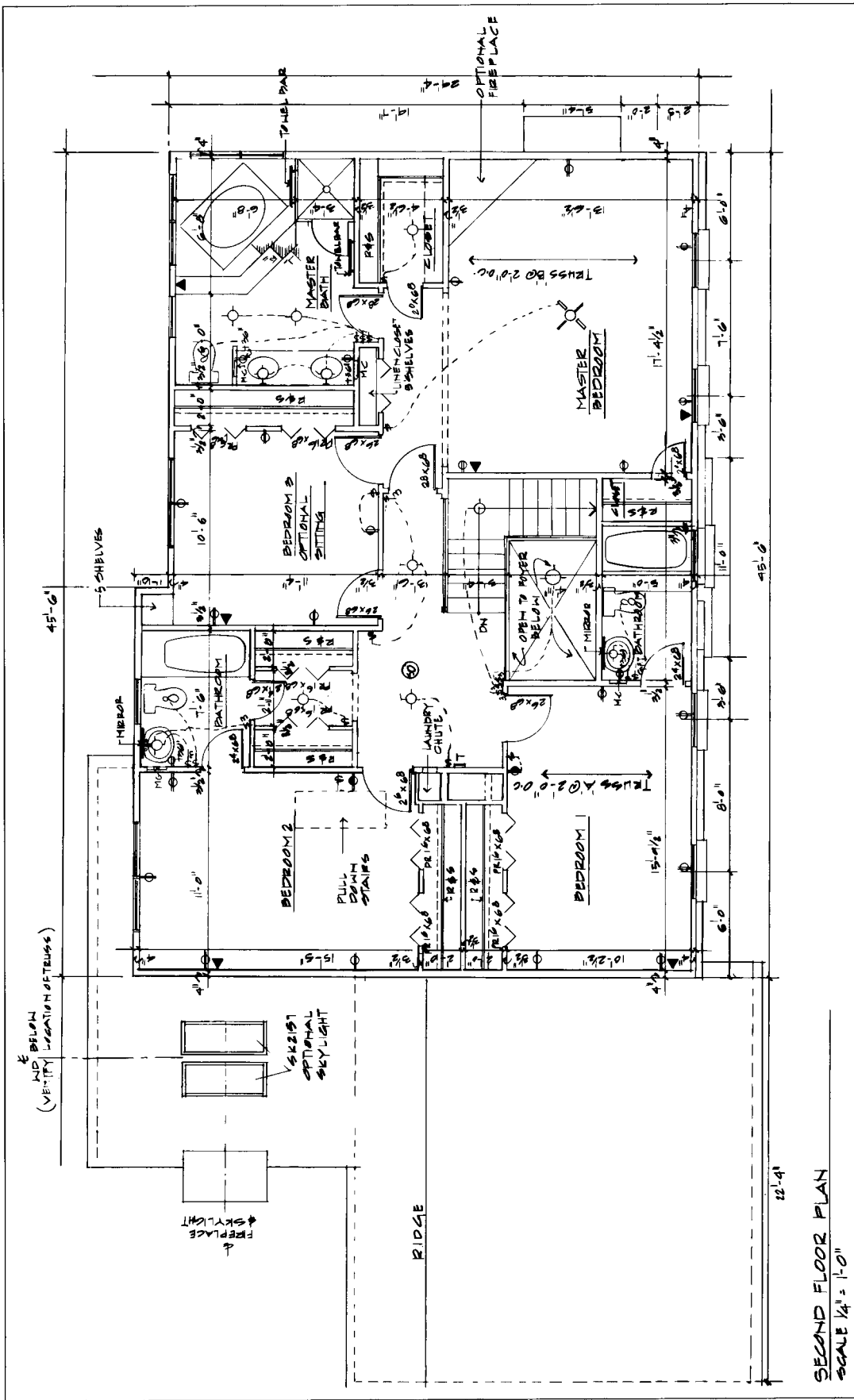


FIGURE 38.23 Typical floor plan.

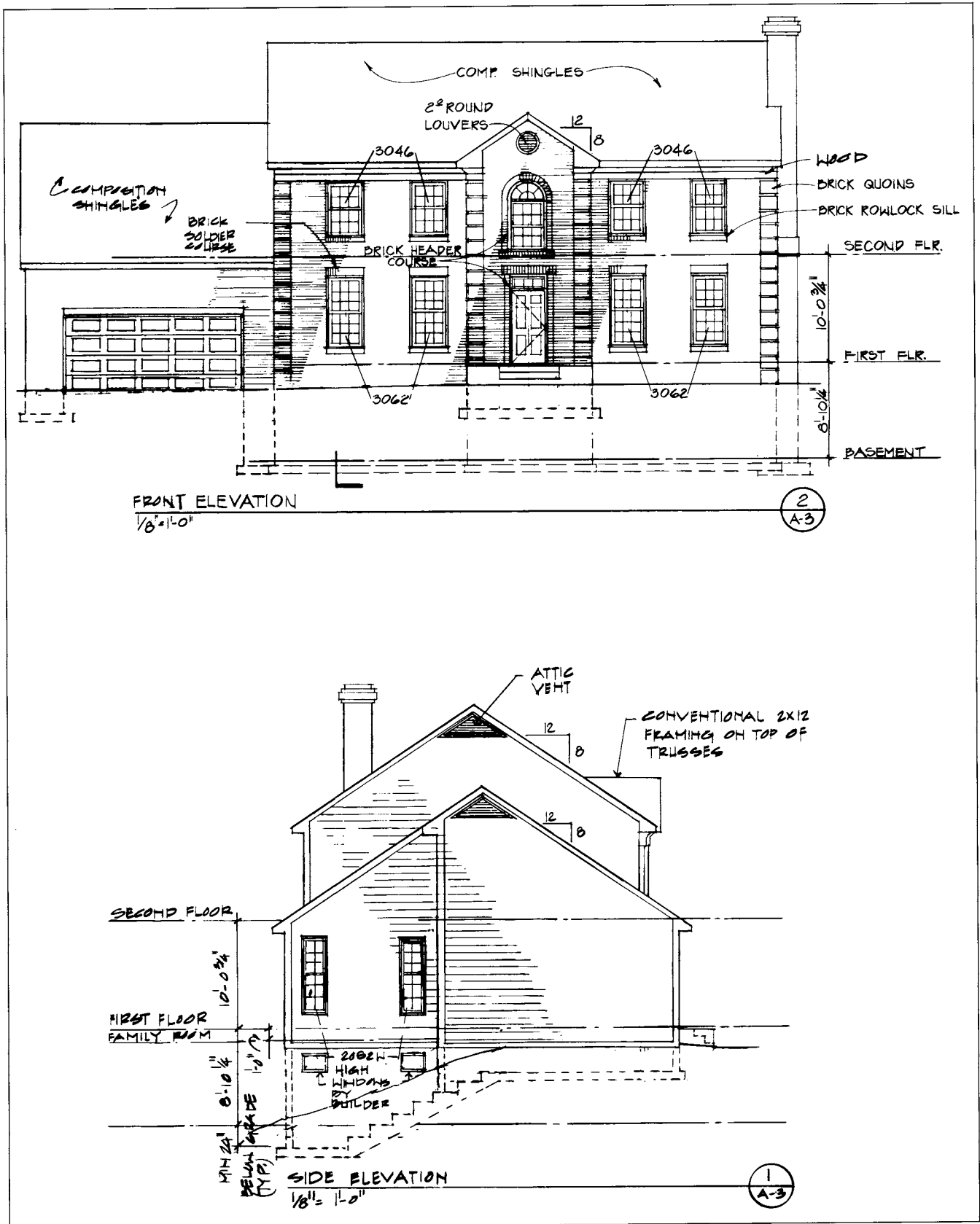


FIGURE 38.24 Architectural Elevations.

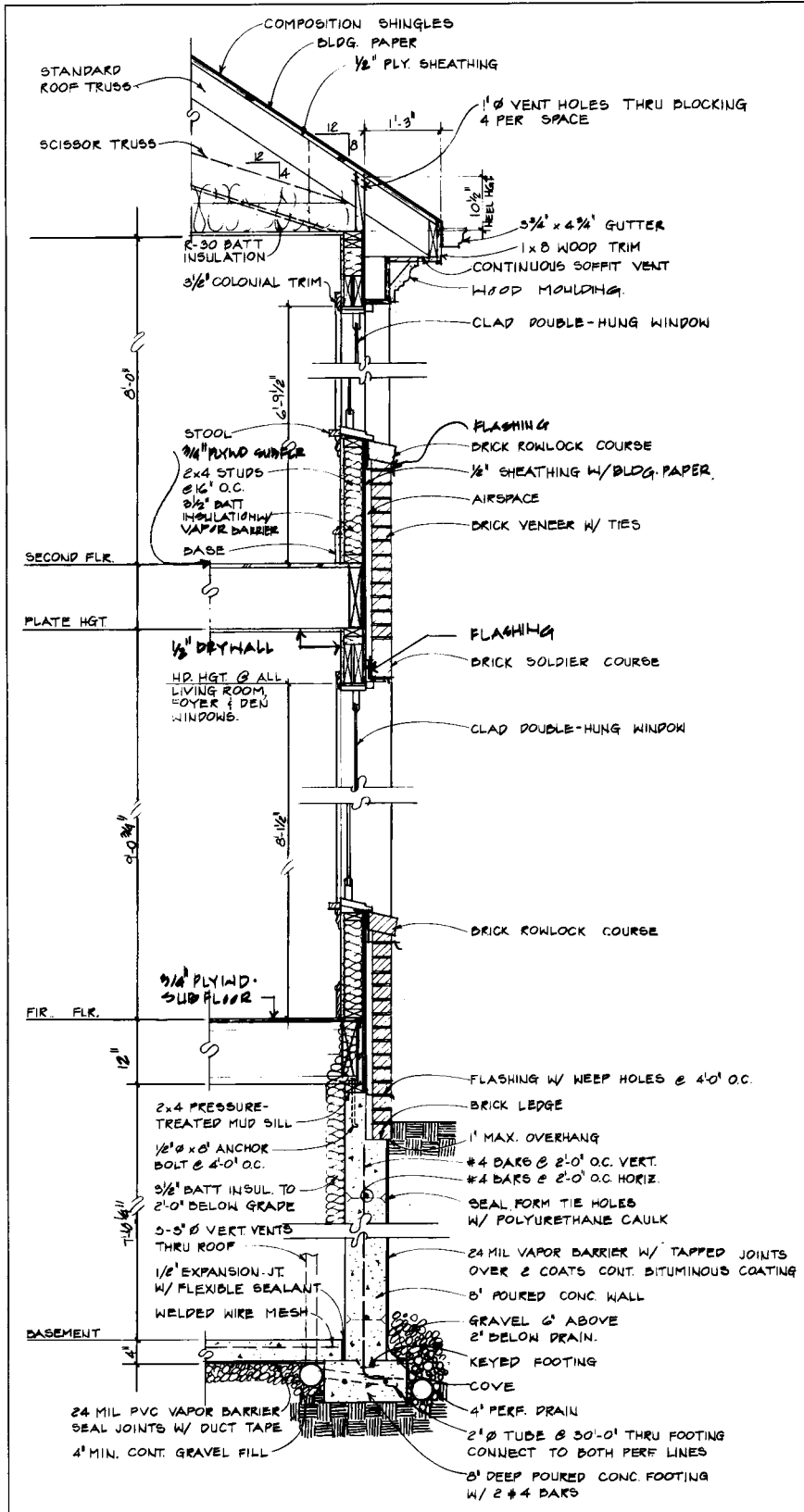


FIGURE 38.25 Wall section.

low-altitude photographic maps. The topographic and geologic-map coverages are identified on state indexes. Other map coverages are outlined in U.S. indexes.

For example, each state index to the topographic maps shows the quadrangle areas by reference code, map name and scale. The more comprehensive catalog, the "State catalog", lists alphabetically, grouped by map type and scale, the names, dates and prices of all available maps. The list includes topographic maps, orthophoto quadrangles, state maps, satellite-image maps and others.

The "Indexes to Geologic Mapping in the United States" comprise a map of each state showing areas for which geologic maps have been published. Each sheet has a color code to identify the scale, along with a text that gives the source of publication, scale, date, and author of each geologic map and a list of USGS reports on the state.

Other small-scale topographic maps are produced by the Army Map Service and the United States Coast and Geodetic Survey (US&GS). The US&GS mainly publishes aeronautical and coastline charts.

Large-scale topographic maps with 5-ft contour intervals are available from the local municipality. These maps provide information usable at the land development level but not all municipalities have these. Large-scale topographic maps may also be accessible from aerial survey firms. Additionally, these firms may have low-altitude aerial photographic maps if the local municipality does not have them either.

Soil surveys, performed by the U.S. Dept. of Agriculture's Soil Conservation Service since the early 1900s, identify soils according to county. This information is presented in map form and text form. The text describes the areal extent, physiography, relief, drainage patterns, climate and vegeta-

tion along with the soil deposits of the area. The soils are described and mapped by the pedologic method. Some of the newer versions of the SCS maps include engineering characteristics of the soils also. Other localized soil maps and reports may be available through local municipal divisions.

Other maps that assist in the planning and design of a land development project include the comprehensive plan map, zoning/land-use maps, political-boundary maps, utility maps, watershed maps, special district maps, and many others that are readily available from the municipality.

## REFERENCES

- American Society of Photogrammetry, *Manual of Photogrammetry*, American Society of Photogrammetry, Falls Church, Va., 1980.
- Ching, Frank, *Architectural Graphics*, 2nd ed., Van Nostrand Reinhold, New York, 1985.
- Croft, Frank M., et. al., *Engineering Graphics*, John Wiley, New York, 1989.
- Giesecke, Frederick E., et. al., *Technical Drawing*, 7th ed., Macmillan, New York, 1980.
- Kicklighter, Clois E. et. al., *Architecture Residential Drawing and Design*, 4th ed., SouthHolland, Ill., 1984.
- Molnar, John, *Building Construction Drafting and Design*, Van Nostrand Reinhold, New York, 1989.
- Prenzel, Rudolf, *Working and Design Drawings*, 2nd ed., Van Nostrand Reinhold, New York, 1989.
- Vaughn, Richard C., *Legal Aspects of Engineering*, 4th ed., Kendall/Hunt, Dubuque, Iowa, 1983.
- Walker, Theodore D., and David D. Davis, *Plan Graphics*, 4th ed., Van Nostrand Reinhold, New York, 1990.
- Wang, Thomas, *Plan and Section Drawing*, Van Nostrand Reinhold, New York, 1979.



## CHAPTER 39

# SOILS

*Douglas D. Frost, P.E.*

### INTRODUCTION

The purpose of this chapter is to familiarize the land development engineer with some of the basic soils issues that are likely to be encountered when he or she begins to plan, design, and alter land use. Knowledge of the basic make-up of soil and its physical characteristics will be helpful to land development engineers when planning site grading, storm-water management structures, roadway and parking area subgrades, retaining structures, and of course foundations for all types of structures. Also included in this chapter are some important engineering properties of soils that will aid land development engineers' design efforts and assist them in understanding soils reports and geotechnical engineering design parameters provided by soils engineers.

From an engineering perspective, soil is the organic and inorganic material of the Earth's surface capable of being displaced by shovel or nominal mechanical efforts. Included in this description are the gasses and liquids within the voids of the solid particles. Soil is an integral part of the design analysis and construction activities. It is used as a foundation base in its natural undisturbed state and as a construction material for embankments. The behavior of soil is very different for each situation. Many structural failures have been attributed to not considering subsurface conditions properly, or to uncertainty in predicting soil behavior. The engineer is concerned with understanding what lies below the ground surface and with soil engineering properties such as strength, consolidation, stability, and permeability and physical properties such as grain-size distribution, moisture content, plasticity, and sometimes corrosivity.

Besides those methods that describe and indicate soil properties for engineering purposes, other methods of de-

scribing, categorizing and identifying soils based on the interests of other professions are available to assist the engineer. The soil scientist (agronomist) is interested in the origin, distribution and classification of soil for agricultural purposes. Geology is the study of the origin, history, and structure of the Earth with a focus on rock and rock formations. While the geologist typically considers soil as the weathering product of rocks, the agronomist's interest in soil is related obviously to agriculture. Soils and their properties are often described by terms specific to these disciplines, and it is helpful if the engineer can relate their terms to his engineering interests.

Geotechnical engineering is the branch of civil engineering that involves the use of soil and rock in engineered construction. While some projects may require careful consideration of rock and ground-water conditions, detailed discussion of engineering geology and ground-water effects are beyond the scope of this chapter.

### SOIL PROPERTIES PHYSICAL & ENGINEERING

#### Phase Diagrams

Soil, in its natural state, exists as a mixture of three distinct phases: water, air (gasses), and the soil itself. The gasses and water occupy the void spaces between the soil particles. If and when the soil is completely saturated, all of the spaces within the soil are filled with water, while a soil that is completely dry has all of its spaces filled only with air (and gasses). In both of these cases, the soil exists as only two phases.

A phase diagram is used to establish weight-volume relationships of the air, water and solid content by separating the constituents into distinct phases. It is a tool used to

define parameters of the soil that are fundamental in establishing the soil's physical and engineering properties, such as unit weight, specific gravity, porosity, void ratio, shear strength, loading and deformation behavior, permeability, consolidation, and shrink-swell behavior.

On the diagram of Figure 39.1, each phase is associated with a volume and weight. Knowing the volume and weight of each constituent provides information on other soil-property parameters derived from the weight-volume relationships.

Assuming the weight of the gasses to be negligible, the total weight,  $W_T$ , of the soil is

$$W_T = W_w + W_s \quad (39.1)$$

where  $W_w$  is the weight of the water and  $W_s$  is the weight of the solids. The total volume,  $V_T$ , is

$$V_T = V_s + V_a + V_w \quad (39.2)$$

where  $V_s$  is the volume of the solids,  $V_a$  is the volume of air (gasses), and  $V_w$  is the volume of the water. The combined volume of gasses and water is the void volume,  $V_v = V_a + V_w$ .

Two parameters that relate the amount of air and water to the amount of solids in the sample are void ratio and porosity. By definition, the void ratio,  $e$ , is

$$e = \frac{V_v}{V_s} \quad (39.3)$$

and porosity  $n$ , expressed as a percent, is

$$n = \frac{V_v}{V_T} \times 100\% \quad (39.4)$$

Please note that the void ratio, commonly expressed as a decimal, can be greater than 1 whereas the porosity cannot exceed 100%.

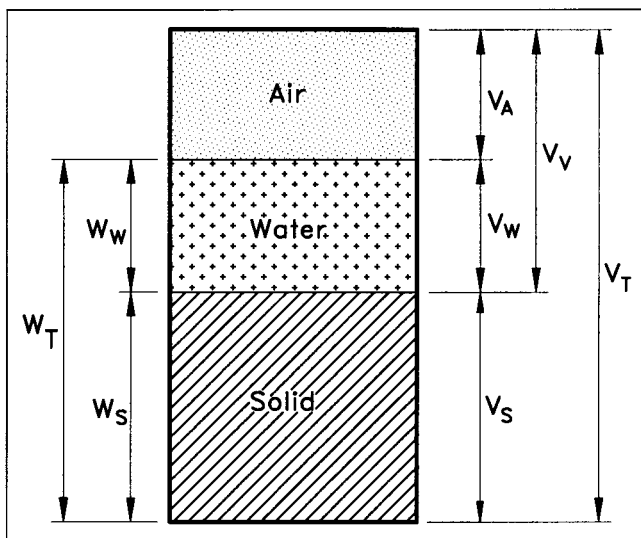


FIGURE 39.1 Three-phase schematic of soil sample.

The degree of saturation,  $S$ , is the relative volume of water in the voids, expressed as a percent. In equation form, the degree of saturation is

$$S = \frac{V_w}{V_v} \times 100\% \quad (39.5)$$

A completely saturated soil sample has no air content, and the phase diagram becomes a two-phase diagram. In such case,  $S$ , the degree of saturation, is 100%.

The water content,  $w$ , is defined as the ratio of the weight of the water to the weight of the solid or

$$w = \frac{W_w}{W_s} \quad (39.6)$$

This parameter greatly influences characteristics of a soil (discussed later), especially in soils containing clays.

Tables 39.1 through 39.4 give the equations for weight-volume relationships using various indexing parameters. Other terms used in the tables and not previously defined are: water content is designated as  $w$ , the specific gravity of the soil is  $G_s$ , density  $\rho$ , and the unit weight is  $\gamma$ . Subscripts  $s$ ,  $w$ , and  $a$  refer to the three phases of solid, water and air respectively.

### Gradation

Terms such as clayey, silty, sandy, and gravelly are descriptive textural terms frequently used to characterize soils. Soil texture is determined by the relative amounts (by dry weight) of individual particle sizes in a sample. Although many agencies and organizations have set up their own limits independently for the various particle sizes, the Unified Soil Classification System is the most widely accepted and has been adopted by the American Society for Testing and Materials. Table 39.5 gives the particle-size range as defined by ASTM.

The size-range of particles is determined through mechanical analysis. The two methods of analysis are sieve analysis (as described in ASTM D-422 for particle sizes larger than 0.075 mm) and hydrometer analysis (ASTM D-1140 for particle sizes less than 0.075 mm).

The sieve analysis consists of passing an oven-dry soil sample of predetermined weight through a nested set of sieves. Each subsequent sieve has progressively smaller openings, as shown in Figure 39.2. The weight of the sample retained on each sieve is expressed as a percentage of the total weight of the dried sample. The number of sieves and their size, identified by a number corresponding to the opening size, are selected to cover the range of particle sizes in the soil sample. Table 39.5 lists the standard sieve series according to ASTM specifications.

Hydrometers are instruments designed to measure the amount of soil (by weight) in suspension. The hydrometer test separates soil-particle sizes through sedimentation, assuming that the soil particles are spherical and their settling velocity follows Stokes' law. The test measures the specific gravity of the soil-liquid solution at a known depth and

TABLE 39.1 Volume Relationships

PROPERTY	DEFINITION	S = 100%	S < 100%				
$V_s$	Vol. of Solids		$\frac{W_s}{G_s \gamma_w}$	$V_T - (V_a + V_w)$	$V_T(1 - n)$	$\frac{V_T}{1 + e}$	$\frac{V_v}{e}$
$V_w$	Vol. of Water		$\frac{W_w}{\gamma_w}$	$V_v - V_a$	$SV_v$	$\frac{SV_T e}{1 + e}$	$SV_s e$
$V_a$	Vol. of Air	0		$V_T - (V_s + V_w)$	$V_v - V_w$	$(1 - S)V_v$	$\frac{(1 - S)V_T e}{1 + e}$ (1 - S)V_s e
$V_v$	Vol. of Voids	$\frac{W_w}{\gamma_w}$	$V_T - \frac{W_s}{G_s \gamma_w}$	$V_T - V_s$	$\frac{V_s n}{1 - n}$	$\frac{V_T e}{1 + e}$	$V_s e$
$V_T$	Total Volume	$V_s + V_w$		$V_s + V_a + V_w$	$\frac{V_s}{1 - n}$	$V_s(1 + e)$	$\frac{V_v(1 + e)}{e}$
$n$	Porosity		$\frac{V_v}{V_T}$	$1 - \frac{V_s}{V_T}$	$1 - \frac{W_s}{G_s V_T \gamma_w}$	$\frac{e}{1 + e}$	
$e$	Void Ratio		$\frac{V_v}{V_s}$	$\frac{V_T}{V_s} - 1$	$\frac{n}{1 - n}$	$\frac{W_w G_s}{W_s S}$	$\frac{G_s V_T \gamma_w}{W_s} - 1$

Taken from the *Civil Engineering Reference Manual, 4th edition*, by Michael R. Lindeburg, with permission from the publisher, Professional Publications, Inc., copyright 1989.

time from the start of sedimentation. This can be related to the amount of soil in suspension at the prescribed depth and time. Application of Stokes' law then determines the percentage of soil by weight finer than a given diameter.

The information obtained from the hydrometer and sieve analysis is presented as a particle-size distribution curve. This curve is a semi-logarithmic plot of the particle diameters (on log scale) along the abscissa with the corresponding percent finer (i.e., total weight less that retained on the particular sieve) along the ordinate on the arithmetic scale. The log scale accentuates the particle-size range better than an arithmetic scale. It is nearly impossible to cover the variations in particle size unless the log of the values is used.

The points are plotted and a smooth curve is drawn through the points. The particle-size distribution curve graphically depicts the range and distribution of particle sizes for the soil sample tested. These curves are also used for presenting grain-size distribution of concrete aggregates, filter media in dam embankments, and for water-well gravel packs.

To illustrate the preceding discussion, consider the data in Table 39.6 showing the results of a sieve analysis of three soil samples of 100 g each.

Particle-size distribution curves for each sample are shown in Figure 39.3. Curve I, a relatively flat curve, shows the soil particle sizes distributed over a wide range. This

TABLE 39.2 Weight Relationships

PROPERTY	DEFINITION	S = 100%	S < 100%		
$W_s$ weight of solids	measured	$\frac{W_T}{(1 + W)}$	$GV\rho_w(1 - n)$	$\frac{W_w G}{eS}$	
$W_w$ weight of water	measured	$wW_s$	$S\rho_w V_v$	$\frac{eW_s S}{G}$	$V \cdot \rho_D \cdot W$
$W_T$ total weight of sample	$W_s + W_w$	$W_s(1 + W)$			

Taken from the *Civil Engineering Reference Manual, 4th edition*, by Michael R. Lindeburg, with permission from the publisher, Professional Publications, Inc., copyright 1989.



TABLE 39.3 Unit Volume Relationships

PROPERTY	DEFINITION	S = 100%	S < 100%				
$\gamma_{\text{Dry}}$	Dry Unit Wt.	$\frac{W_s}{V_s + V_w}$	$\frac{W_s}{V_T}$	$\frac{W_T}{V(1+w)}$	$G_s \gamma_w (1-n)$	$\frac{G_s \gamma_w}{1+e}$	$\frac{G_s \gamma_w}{1 + \frac{wG_s}{S}}$
$\gamma_{\text{wet}}$	Wet Unit Wt.	$\frac{W_s + W_w}{V_s + V_w}$	$\frac{W_s + W_w}{V_T}$	$\gamma_D(1+w)$	$G_s \gamma_w (1-n) * (1+w)$	$\frac{(G_s + Se) \gamma_w}{1+e}$	$\frac{(1+w) \gamma_w}{\frac{w}{S} + \frac{1}{G}}$
$\gamma_{\text{sat}}$	Saturated Unit Wt.	$\frac{W_s + W_w}{V_s + V_w}$	$\frac{W_s + V_v \gamma_w}{V_T}$	$\frac{W_s}{V} + \frac{\gamma_w e}{1+e}$	$\frac{(G_s + e) \gamma_w}{1+e}$	$\frac{(1+w) \gamma_w}{w + \frac{1}{G_s}}$	

Taken from the *Civil Engineering Reference Manual, 4th edition*, by Michael R. Lindeburg, with permission from the publisher, Professional Publications, Inc., copyright 1989.

type of soil is called a well-graded soil. Curve II shows a soil sample composed of two uniform soils (represented by curves on either side of the horizontal segment), with an absence of particle sizes in the 0.85-mm to 0.425-mm size range. Such a soil is referred to as gap-graded. The relatively steep curve of soil sample III indicates a poorly graded soil—that is, nearly uniform particle sizes. A term analogous to the engineering concept of gradation is “sorted”. Geologists use this term to describe the uniformity of the particle sizes in a soil. A well-graded engineering soil is a poorly sorted geologic soil.

Grain-size distribution curves provide clues as to the type of soil deposits and the geologic origin of the sample. For instance, some glacial soils contain a wide range of particle sizes. This type of soil would appear as a well-graded distribution curve. Alluvial and eolian soils might appear as poorly-graded soils because of the mechanics of natural sorting by flowing water and wind.

Terms associated with soil gradations used to identify soil properties further include:

- Effective size—grain diameter at 10% finer ( $D_{10}$ )
- Uniformity coefficient ( $C_u$ ), given as

$$C_u = \frac{D_{60}}{D_{10}} \quad (39.7)$$

- Coefficient of gradation ( $C_c$ )

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} \quad (39.8)$$

The  $D_{10}$  and  $C_u$  parameters are a means to compare particle-distribution curves for different soils without plotting them on the same graph. The uniformity coefficient indicates the shape of the curve and  $D_{10}$  positions along the curve. An

TABLE 39.4 Additional Relationships

PROPERTY	DEFINITION	S = 100%	S < 100%			
$w$	Moisture Content	$\frac{n}{(1-n)G_s}$	$\frac{W_w}{W_s}$	$\frac{W_T}{W_s} - 1$	$\frac{Se}{G_s}$	$S \left( \frac{\gamma_w}{\gamma_D} - \frac{1}{G_s} \right)$
$S$	Degree of Saturation	1.00	$\frac{V_w}{V_v}$	$\frac{W_w}{V_v \gamma_w}$	$\frac{wG_s}{e}$	$\frac{w}{\frac{\gamma_w}{\gamma_D} - \frac{1}{G}}$
$G_s$	Specific Gravity		$\frac{W_s}{V_s \gamma_w}$		$\frac{Se}{w}$	

Taken from the *Civil Engineering Reference Manual, 4th edition*, by Michael R. Lindeburg, with permission from the publisher, Professional Publications, Inc., copyright 1989.

**TABLE 39.5 USA Sieve Series (ASTM Specification E-11-70) Showing the Corresponding Soil-Grain Size (ASTM D-2487)**

STANDARD <sup>B</sup>	Alternative	Description	Standard	Alternative	Description	
125 mm	5 in.	Cobbles*	2.00 mm	No. 10		
106 mm	4.24 in.		1.70 mm	No. 12		
100 mm <sup>D</sup>	4 in. <sup>D</sup>		1.4 mm	No. 14		
90 mm	3½ in. <sup>E</sup>		1.18 mm	No. 16		Medium Sand
75 mm	3 in.	1.00 mm	No. 18			
63 mm	2½ in.	Coarse gravel	850 μm <sup>F</sup>	No. 20		
53 mm	2.12 in.		710 μm	No. 25		
50 mm <sup>D</sup>	2 in. <sup>D</sup>		600 μm	No. 30		
45 mm	1¾ in.		500 μm	No. 35		
37.5 mm	1½ in.		425 μm	No. 40		
31.5 mm	1¼ in.		355 μm	No. 45		
26.5 mm	1.06 in.		300 μm	No. 50		
25 mm <sup>D</sup>	1 in. <sup>D</sup>		250 μm	No. 60		Fine Sand
22.4 mm	7⁄8 in.		212 μm	No. 70		
19 mm	¾ in.		180 μm	No. 80		
16 mm	5⁄8 in.	150 μm	No. 100			
13.2 mm	0.530 in.	Fine gravel	125 μm	No. 120		
12.5 mm <sup>D</sup>	½ in. <sup>D</sup>		106 μm	No. 140		
11.2 mm	7⁄16 in.		90 μm	No. 170		
9.5 mm	¾ in.		75 μm	No. 200		
8.0 mm	5⁄16 in.		63 μm	No. 230		
6.7 mm	0.265 in.		53 μm	No. 270		Fines
6.3 mm <sup>D</sup>	¼ in. <sup>D</sup>		45 μm	No. 325		
5.6 mm	No. 3½		38 μm	No. 400		
4.75 mm	No. 4	32 μm	No. 450			
4.0 mm	No. 5	Coarse Sand	25 μm <sup>D</sup>	No. 500		
3.35 mm	No. 6		20 μm <sup>D</sup>	No. 635		
2.80 mm	No. 7					
2.36 mm	No. 8					

Copyright ASTM. Reprinted with permission.

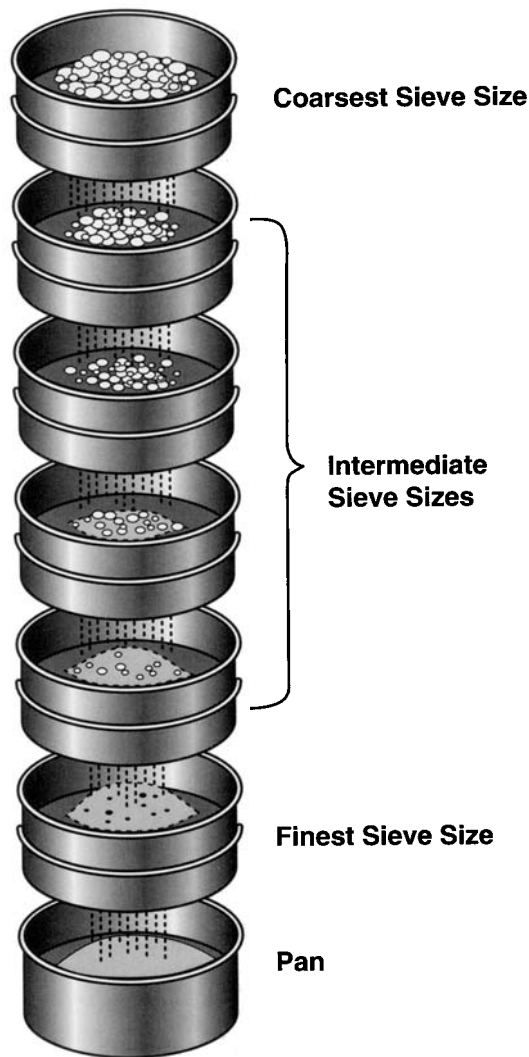
<sup>B</sup>These standard designations correspond to the values for test sieve apertures recommended by International Standards Organizations, Geneva, Switzerland.

\*Cobbles range from 300 mm to 90 mm; Particles greater than 300 mm are designated as boulders.

<sup>D</sup>These sieves are not in the standard series but they have been included because they are in common usage.

<sup>E</sup>These numbers (93½ to 635) are the approximate number of openings per linear in. but it is preferred that the sieve be identified by the standard designation in millimetres or micrometers.

<sup>F</sup>1000 μm=1 mm.



**FIGURE 39.2** Sieve analysis.

ideal uniform soil (i.e., very poorly graded) has a uniformity coefficient of 1. Soils with  $C_u$  less than 2 are considered uniform or poorly graded. Well-graded soils generally have  $C_u$  values of 4 or more.

Soil gradation typically is specified for controlled backfill material, dam embankments, soil filters (to prevent piping), as well as subbase for roads and foundations.

### Atterberg Limits

Cohesive soils are composed of fine-grained (<0.002-mm) particles, often flaky in shape rather than spherical or blocky. The interaction of the fine particles and water molecules causes these soils to exhibit some degree of plasticity. Plastic materials deform without cracking (under small stress) and maintain the deformed shape after the stress has been relaxed. Elastic materials, on the other hand, deform when subjected to small stresses but rebound to their original shape when the stress is relaxed. The strength of cohesive soils usually depends on the water content. A

cohesive soil has considerably more strength at a low water content than at high water content.

Depending on the moisture content of a cohesive soil, its behavior ranges from a solid (at very low moisture content), to a semisolid, to a plastic, and finally to a liquid phase at very high moisture contents. The moisture content, in percent, for which the behavior transitions from solid to a semisolid is defined as the shrinkage limit. As a semisolid soil sample is dried, its mass volume decreases to a minimum below which no further reduction mass volume occurs with any further drying. The moisture content, for which the soil transitions from the semisolid to plastic phase, is called the plastic limit. As the moisture content increases, the soil transforms into the liquid phase. The moisture content at which this transformation occurs is the liquid limit. Figure 39.4 shows the relationship of the water content and the consistency range of cohesive soils. The difference between the liquid limit and the plastic limit is known as the plasticity index (PI).

The moisture content defining the shrinkage limit, plastic limit, and the liquid limit, sometimes referred to collectively as the Atterberg limits, is a measure of the consistency of the cohesive soil. The plasticity index is a measure of the range of water content over which the soil remains plastic. Plasticity of fine-grained soils is caused by the adsorbed water on the particles.

The type of clay minerals and the proportional amounts of clay minerals the soil affect the liquid limit and the plastic limit. Based on their crystal structure and chemistry, clays are classified into distinct groups. A few of the more common clay mineral groups include Kaolinite, Halloysite, Illite, and Montmorillonite. Less common are Vermiculite and Chlorite. The activity,  $A$ , is an index that identifies the swelling potential of clay soils. Activity is defined as

$$BBB = \frac{PI}{(\% < 0.002 \text{ mm}) \eta} \quad (39.9)$$

where the denominator represents the fraction (by weight) of the sample smaller than 0.002 mm. The activity of kaolinite is less than 1, illite has an intermediate activity range of 1 to 2 and montmorillonite clays have an activity of greater than 4.

A wide variation the liquid limit is typical among cohesive soils. Values as high as 80 to 100 are not uncommon, although values of 40 to 60 are more representative of such soils. The liquid limit range for silty soils is typically 25 to 50. Plastic limits of silts and clays do not vary as widely as the liquid limits; the normal range is between 15 and 30. Silty soils are less plastic than clay and tend to be at the higher end of this range. The plasticity index can be as high as 70 to 80 for very plastic clays, although more common values for the plasticity index for clays are between 20 and 40. The plasticity index for silty materials normally ranges between 10 and 20. Table 39.7 gives ranges for the liquid

**TABLE 39.6 Particle-Size Distribution of Three Soil Samples**

SIEVE #	SAMPLE I		SAMPLE II		SAMPLE III	
	% RETAIN	% PASS.	% RETAIN	% PASS.	% RETAIN	% PASS.
8	0	100	0	100	0	100
10	1.0	99.0	2.5	97.5	0	100
14	3.0	96.0	7.5	90.0	2.0	98.0
16	5.0	91.0	18.5	71.5	12.0	86.0
20	3.5	87.5	12.5	59.0	22.0	64.0
25	6.0	81.5	3.0	56.0	43.0	21.0
30	5.5	76.0	0.5	55.5	15.0	6.0
40	13.0	63.0	2.0	53.5	6.0	0.0
50	13.0	50.0	12.0	41.5	0.0	0.0
70	12.0	38.0	17.0	24.5		
100	17.0	21.0	13.0	11.5		
140	11.0	10.0	8.0	3.5		
200	8.0	2.0	3.5	0.0		
230	2.0	0.0	0.0	0.0		

limit, plastic limit and shrinkage limit for various clay minerals.

The results from studies done by Casagrande on the relationship of the plasticity index to the liquid limit for a variety of soils lead to the formation of a plasticity chart. The chart, Figure 39.5, delineates inorganic clays from inorganic silts by the empirically-derived A-line. The U-line defines the upper limit of the relationship of the plasticity index to the liquid limit for any soil found. This plasticity chart has been modified for use as part of ASTM D-2487, "Classification of Soils for Engineering Purposes."

Typical index values of various soils are shown in Table 39.8.

### Permeability

The study of geotechnical engineering requires a thorough understanding of the sometimes complex integration of soil, water and air that makes up the three-phase soil schematic discussed in the beginning of this chapter. Variation in a soil's water content can alter its physical and strength characteristics dramatically. For instance, one can easily walk or even drive on wet sand and dry clay. But doing the same on wet clay and dry sand would prove very difficult (Liu, 1937).

Water flow through soil or any other porous material is an engineering property called "permeability." Permeability

is an important property of soil and influences the design and constructability of nearly all land development projects. A few examples of projects in which permeability should be considered are:

- Seepage of water through dams and levees
- Water well recharge times and dewatering of excavations
- Rate of settlement of fine-grained soils
- Roadway subgrade and drainage design
- Drainage, capillary, and frost action

For engineering purposes, the permeability of a soil is based on "Darcy's Law" and is discussed in terms of the "constant of proportionality ( $k$ )," which is commonly referred to as the coefficient of permeability and has the units of velocity.

Because soils are often deposited in relatively horizontal layers, vertical and lateral permeability of soils varies widely even in the most homogeneous soil deposits. Non-homogeneous layering in soil deposits increases this natural variability. The determination of a soil's coefficient of permeability ( $k$ ) is made one of several ways. Approximate values can be obtained from laboratory testing using the

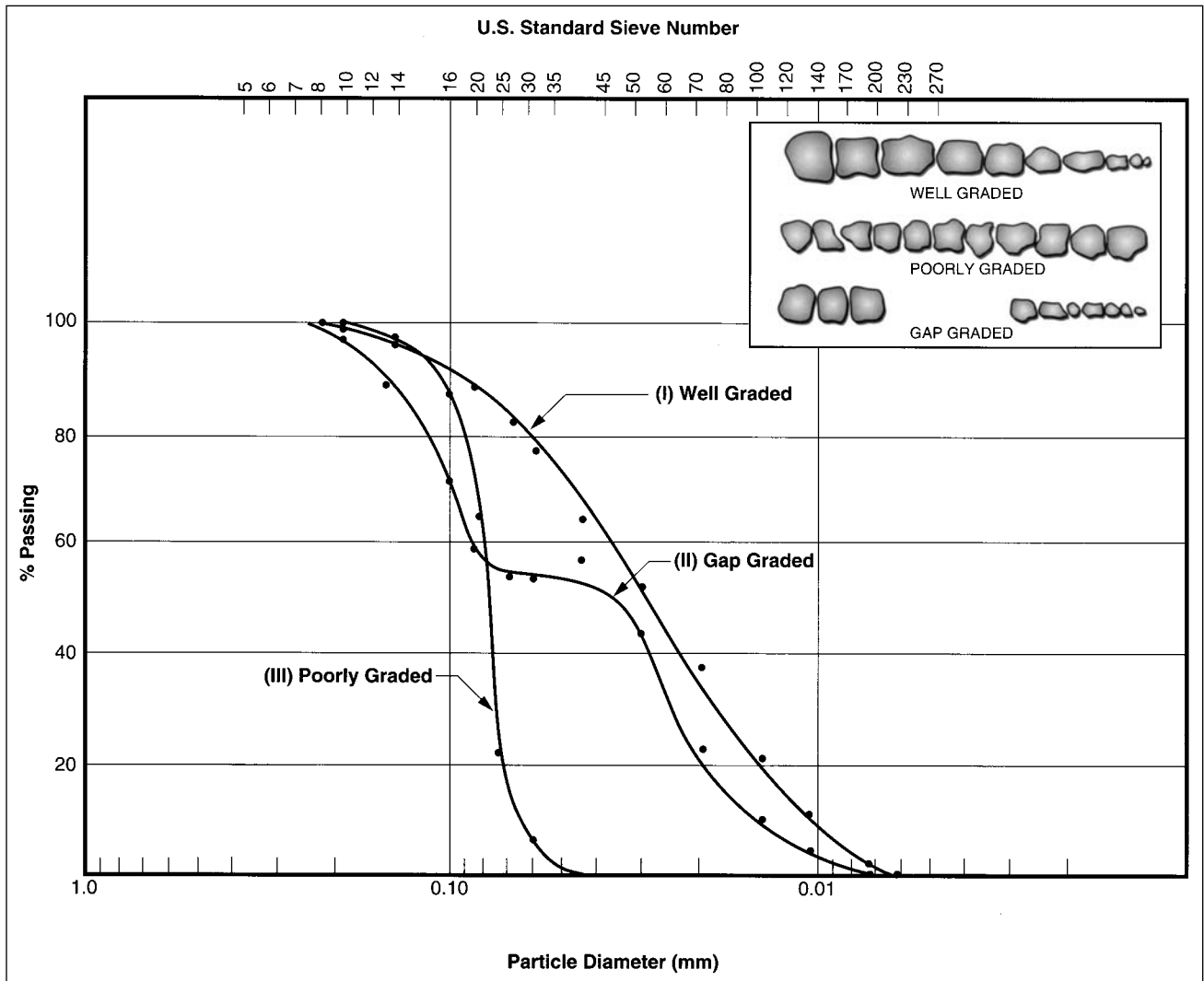


FIGURE 39.3 Particle-size distribution curves.

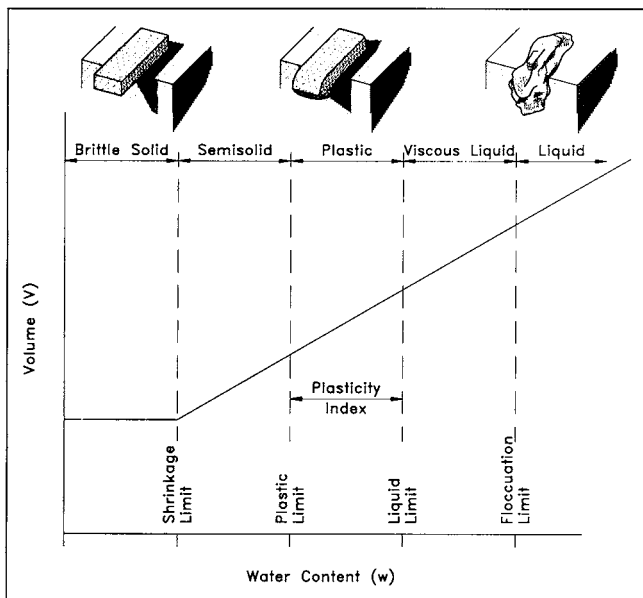


FIGURE 39.4 Atterberg limits—phase diagram.

constant-head (ASTM D-2434) or falling-head permeability test. Table 39.9 provides some information on the range of the coefficient of permeability, drainage characteristics, and applicable testing methods for determining  $k$  values for common soil classes.

Field pump testing utilizing an extraction well in concert with a series of radial-spaced observation wells can be used for large-scale dewatering design. Single-well tests, such as “slug” testing, can also be used to estimate permeability. Finally, there are some empirical formulas that can be used to obtain rough orders of magnitude of the coefficient of permeability.

For clean cohesionless soils Hazen (1911) proposed

$$k = C(D_{10})^2 \tag{39.10}$$

where

- $k$  = coefficient of permeability in mm/sec
- $D_{10}$  = effective grain size mm
- $C$  = a constant ranging from 10 to 15

**TABLE 39.7 Atterberg Limit Values for Clay Minerals\***

MINERAL	LIQUID LIMIT	PLASTIC LIMIT	SHRINKAGE LIMIT
Montmorillonite	100–900	50–100	8.5–15
Nontronite	37–72	19–27	
Illite	60–120	35–60	15–17
Kaolinite	30–110	25–40	25–29
Hydrated halloysite	50–70	47–60	
Dehydrated halloysite	35–55	30–45	
Attapulgite	160–230	100–120	
Chlorite	44–47	36–40	
Allophane	200–250	130–140	

From *Principles of Geotechnical Engineering, 3rd edition*, by B. M. Das © 1994. Reprinted with permission of Brooks/Cole, an imprint of the Wadsworth Group, a division of Thomson Learning.

### Shear Strength

The stability of a soil is largely dependent on its shearing strength, which is a measure of the resistance to sliding between soil particles. Internal friction and cohesion are the two components that combine to produce the sliding resistance. Cohesion is the magnetic-like attraction between particles caused by the atomic structure of the individual particles. Cohesion is inherent in all clay as well as some other fine-grained soils. Friction is the resistance of the particles rubbing across one another. Frictional resistance depends on the physical characteristics of the soil grains, such as particle angularity, surface roughness, sphericity, gradation, and relative density. The measure of this resistance is referred to as the angle of internal friction.

The concept of shear strength is best illustrated with the following description of the direct-shear test (ASTM D-3080). A soil sample is placed in a shear box, a device that is split along a horizontal plane through the middle, as shown in Figure 39.6. A normal force is applied to the top of the box by using weights. A horizontal force is then gradually applied to the top half of the box until the soil sample fails. The normal force divided by the area of the sample is the normal stress  $\sigma$ ; the horizontal force divided by the sample area is the shear stress  $\tau$ . This procedure is repeated for various values of the normal force and the results are plotted as a curve of  $\sigma$  versus  $\tau$ .

A granular, cohesionless soil will produce the curve of Figure 39.7(a). This type of curve shows that the shear resistance is linearly proportional to the normal force. The constant of proportionality, designated as  $\phi$ , is the angle of internal friction. These soils have no shear strength when

the normal stress is zero and, therefore, will not hold together unless confined. The curve of Figure 39.7(b) shows a soil that exhibits some degree of cohesion, indicated by the vertical shift in the curve.

Because of the apparatus configuration, the direct-shear test imposes failure along a pre-described plane rather than the weakest plane in the soil sample. Additionally, the distribution of shear strain is not uniform over the sample, and this results in progressive failure. The results of these shortcomings in the test may give higher than actual values. The direct-shear test is especially useful for determining the angle of internal friction ( $\phi$ ) for compacted backfill soils adjacent to retaining walls. Other shear tests include ring or double direct and triaxial, the latter of which gives the most consistent results.

The voids between soil particles are called pore spaces. The water and the corresponding pressure are referred to as pore water and pore pressure. Water flowing through the soil skeleton, although usually very slow, affects the transmission of the resisting forces among the soil particles, thus influencing the compression and shear resistance of the soil. Within the context of soil strength, two terms commonly used are drained and undrained shear strength. The drained strength is the tested strength of a saturated soil sample as the water is allowed to dissipate out of the soil while the loading is applied. This strength parameter is applicable for use when the underlying soils are granular (i.e., pervious). Hence, as the surface load (e.g., shallow footing) is applied to an upper-saturated soil layer, the water in the soil would permeate into the granular soil. However, if the lower soil layers are impermeable, the undrained strength parameter is used in the design analysis.

The shearing strength of a soil is given by the Coulomb equation,

$$s = c + \sigma \tan \phi \quad (39.11)$$

where

$s$  = shearing strength

$c$  = cohesion

$\sigma$  = normal stress on the shear plane

$\phi$  = angle of internal friction

Shearing strength of a soil is a combination of the cohesive properties and the friction properties. For pure clays the shearing strength is only dependent on cohesion (i.e.,  $\phi = 0$ ). Recognize, however, that cohesion is not constant. Factors affecting cohesion are changes in water content, rate and time of loading, chemical composition of the clay, and numerous other factors. Hence, slope stability for such soils depends on a number of conditions.

Because the internal friction of granular soils is from the particle-to-particle resistance, the shearing strength increases relation to the confining pressure to which they are subjected. This is evident from the Coulomb equation and Figure 39.7(a). For a given granular soil where  $c = 0$  and  $\phi$  is constant, as the normal stress,  $\sigma$ , increases, the shear-

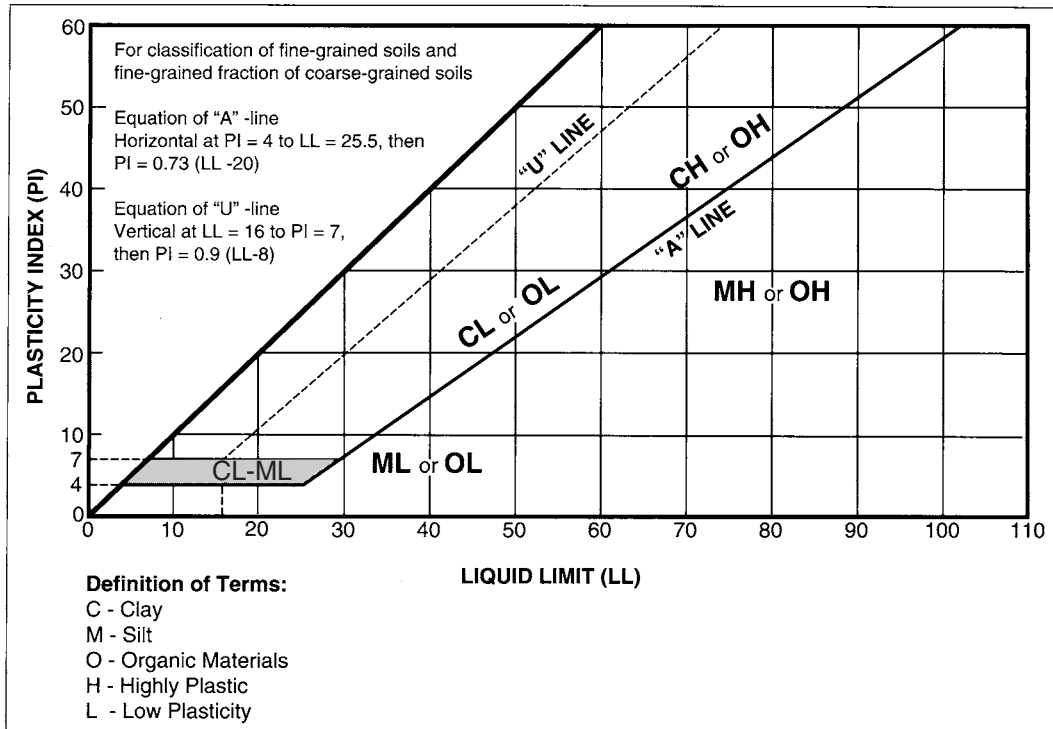


FIGURE 39.5 Plasticity chart. Copyright ASTM. Reprinted with permission.

ing strength also increases. Conversely, the shearing strength of a pure clay soil with  $\varphi = 0$  is independent of the normal stress and attributable solely to the cohesive properties of the soil.

### Compressibility

Nearly everyone has seen pictures of the Leaning Tower of Pisa, listing to one side and looking like it could topple at any second. Settlement has haunted the designers and builders of this world famous structure for more than 700 years. Countless other structures and civil projects have suffered a similar if not more catastrophic fate (Sowers 1977.)

Structures built on soil are subject to settlement. Most often settlement results from increased stress, which causes compressive deformation (i.e., a reduction in the soil's void ratio) in the supporting soil matrix because of the weight of the new structures. Other causes of settlement include dynamic forces, changes the ground-water table elevation, and nearby excavations (Liu, 1937).

Settlement generally takes the form of relatively immediate subsidence and long-term, time-dependent substance. Compression (consolidation) is the result of soil particles re-arranging themselves into a tighter, more compact state. For dry soils, which are made up of soil particles and air voids (a two-phase system), settlement would occur quite rapidly under the application of an increased load. The air voids are easily compressed as the soil grains continue to slide past each other until equilibrium is achieved between the increased stress and the resisting friction generated from adjacent soil grains. This type of settlement behavior is referred to as immediate or distortion settlement ( $S_d$ ).

Soils, which are characterized by a three-phase system composed of soil, air voids, and water (or when a soil matrix is 100% saturated), undergo a time-dependent consolidation since the applied load is carried initially by the soil and water. With the passing of time, the water is squeezed out of the soil, allowing the grains to move closer together.

The rate at which the water is displaced is governed by the soil's permeability. Soils with high permeability (i.e., coarse-grained soils) require relatively short time periods to consolidate, and most of the settlement is complete by the time construction operations are completed (Liu, 1937).

Soils of low permeability (i.e., fine-grained clay soils) can experience consolidation settlements for many years and, in fact, may continue to be compressed well beyond the useful life of the structure. This is especially true of organic or peat soils. Note also that the rate of consolidation for a given soil is also related to the thickness of the soil layer, since that controls the distance the pore water has to travel to escape the soil layer.

Settlement associated with the displacement of pore water out of a soil layer is referred to as "primary consolidation settlement" ( $S_p$ ). Following the primary consolidation, additional consolidation settlement may occur, although it will be at a reduced rate and magnitude. This additional, often-negligible settlement is known as "secondary compression" ( $S_s$ ) and results from the reorientation and breaking of the clay-to-clay bonds. Secondary compression is also referred to as soil "creep."

The total settlement ( $S$ ) to be anticipated is obtained by summing the aforementioned settlement components and is represented by the following formula (Winterkorn 1975.)

**TABLE 39.8 Table of Values of Soil Index Properties**

	PARTICLE SIZE AND GRADATION				VOIDS <sup>1</sup>				UNIT WEIGHT <sup>2</sup> (L.B.CU.FT)						
	APPROX. SIZE RANGE (MM.)		APPROX. $D_{10}$ (MM)	APPROX. RANGE UNIF. COEF., $C_u$	VOID RATIO		POROSITY (%)		DRY WT., $\gamma_{dry}$		WET WT., $\gamma_{wet}$		SUB. WT., $\gamma_{sub}$		
	$D_{max}$	$D_{min}$			$F_{max}$ (LOOSE)	$F_{cr}$	$F_{min}$ (DENSE)	$n_{max}$ (LOOSE)	$n_{min}$ (DENSE)	MIN. (LOOSE)	MAX. (DENSE)	MIN. (LOOSE)	MAX. (DENSE)	MIN. (LOOSE)	MAX. (DENSE)
<b>Granular Materials</b>															
1. Uniform Materials															
a. Equal spheres (theoretical values)															
	—	—	—	1.0	0.92	—	0.35	47.6	26.0	—	—	—	—	—	—
b. Standard Ottawa Sand															
	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	93	131	57	69	73
c. Clean, uniform sand (fine or medium)															
	—	—	—	1.2 to 2.0	1.0	0.80	0.40	50	29	83	84	136	52	73	73
d. Uniform, inorganic silt															
	0.05	0.005	0.012	1.2 to 2.0	1.1	—	0.40	52	29	80	81	136	51	73	73
2. Well-graded materials															
a. Silty sand															
	2.0	0.005	0.02	5 to 10	0.90	—	0.30	47	23	87	88	142	54	79	86
b. Clean, fine to coarse sand															
	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	86	148	53	86	86
c. Micaceous sand															
	—	—	—	—	1.2	—	0.40	55	29	76	77	138	48	76	92
d. Silty sand & gravel															
	100	0.005	0.02	15 to 300	0.85	—	0.14	46	12	89	90	155 <sup>3</sup>	56	92	92
<b>Mixed Soils</b>															
1. Sandy or silty clay															
	2.0	0.001	0.003	10 to 30	1.8	—	0.25	64	20	60	100	147	38	85	85
2. Skip-graded silty clay with stones or rock fragments															
	250	0.001	—	—	1.0	—	0.20	50	17	84	115	151	53	89	89
3. Well-graded gravel, sand, silt & clay mixture															
	250	0.001	0.002	25 to 1000	0.70	—	0.13	41	11	100	125	156 <sup>4</sup>	62	94	94
<b>Clay Soils</b>															
1. Clay (30 to 50% clay sizes)															
	0.05	0.5u	0.001	—	2.4	—	0.50	71	33	50	94	133	31	71	71
2. Colloidal clay ( $-0.002$ mm. $\geq 50\%$ )															
	0.01	10A	—	—	12	—	0.60	92	37	13	71	128	8	66	66
<b>Organic Soils</b>															
1. Organic silt															
	—	—	—	—	3.0	—	0.55	75	35	40	87	131	25	69	69
2. Organic clay (30 to 50% clay sizes)															
	—	—	—	—	4.4	—	0.70	81	41	30	81	125	18	62	62

<sup>1</sup> Granular materials may reach  $e_{max}$  when dry or only slightly moist. Clays can reach  $e_{max}$  only when fully saturated.

<sup>2</sup> Granular materials reach minimum unit weight when at  $e_{max}$  and with hygroscopic moisture only. Clays reach minimum unit wet weight when fully saturated at  $e_{max}$ . The unit submerged weight of any saturated soil is the unit wet weight minus the unit weight of water.

<sup>3</sup> Applicable for very compact glacial till. Unusually high unit weight values for tills are sometimes due not only to an extremely compact condition but to unusually high specific gravity values.

<sup>4</sup> Applicable for hardpan.

GENERAL NOTE: Tabulation is based on  $G = 2.65$  for granular soil,  $G = 2.7$  for clays, and  $G = 2.6$  for organic soils.

From *Basic Soils Engineering, 2nd edition*, Hough, B. K. Copyright 1969. Reprinted by permission of John Wiley & Sons, Inc.



TABLE 39.9 Permeability Characteristics of Soils

PERMEABILITY CHARACTERISTICS OF SOILS AND THEIR METHODS OF MEASUREMENT *												
Coefficient of Permeability $k$ , cm/s (log scale)												
	10 <sup>2</sup>	10 <sup>1</sup>	10 <sup>0</sup>	10 <sup>-1</sup>	10 <sup>-2</sup>	10 <sup>-3</sup>	10 <sup>-4</sup>	10 <sup>-5</sup>	10 <sup>-6</sup>	10 <sup>-7</sup>	10 <sup>-8</sup>	10 <sup>-9</sup>
Drainage				Good drainage			Poor drainage			Practically impervious		
Types of soil	Clean gravel		Clean sand Clean sand and gravel mixtures			Very fine sands; organic and inorganic silts; mixtures of sand, silt, and clay; glacial till stratified clay deposits; etc.			"Impervious soils, e. g., homogeneous clays below zone of weathering			
Direct determination of coefficient of permeability	Direct testing of soil in its original position (e. g. field-pumping tests)											
	Constant-head permeameter											
	Falling-head permeameter											
Indirect determination of coefficient of permeability	Computations from grain size distribution, porosity, etc.											
						Horizontal capillarity test			Computations from time rate of consoli- dation and rate of pressure drop at constant volume			

\* After Casarande and Fadum (1940), from Hunt (1984)

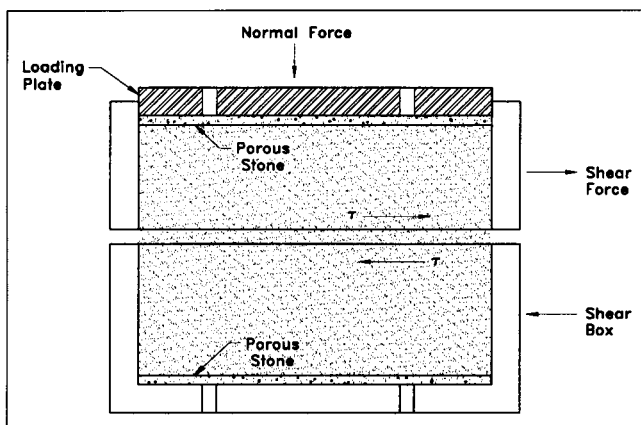
$$S = S_d + S_c + S_s \quad (39.12)$$

where

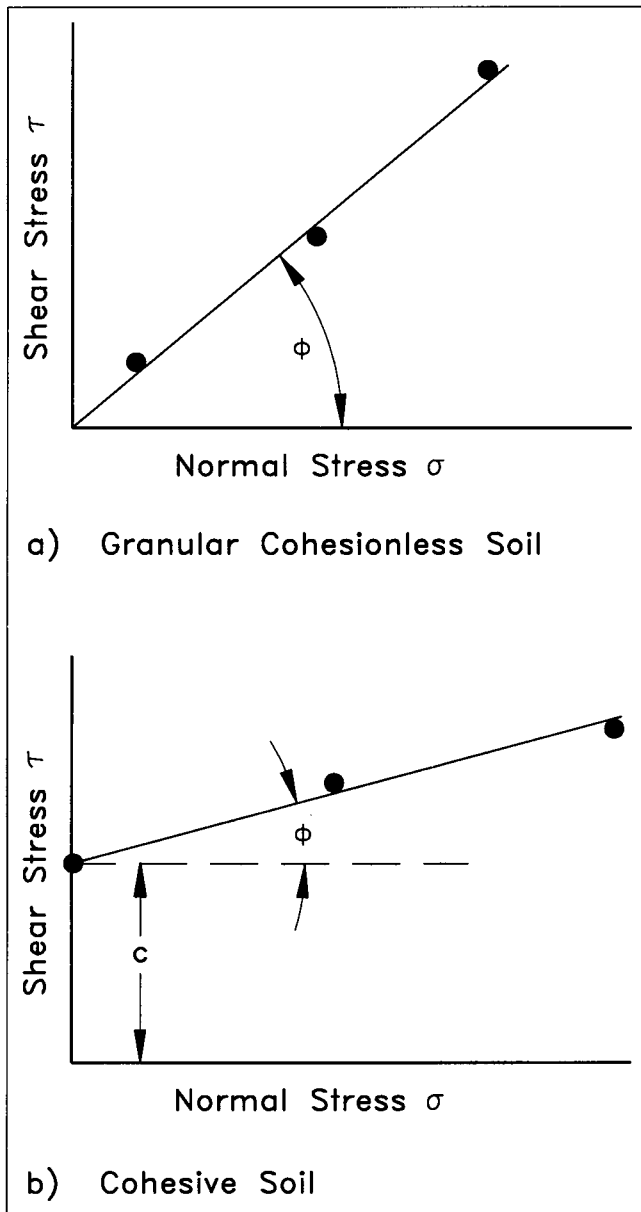
- $S$  = total settlement
- $S_d$  = distortion settlement
- $S_c$  = primary consolidation settlement
- $S_s$  = secondary compression settlement

### METHODS FOR CLASSIFYING SOIL

Soil classification is a means to describe soil behavior according to the interests of a particular discipline. From an engineering perspective, although a soil is non-homogeneous and anisotropic (reacts differently under different forces), there are methods available to categorize soils systematically for construction purposes. Although soil classification provides a general sense of how the soil will likely behave, based on its engineering properties, this alone cannot be the sole criterion for measuring the soil's behavior. A detailed investigation of the soil, using subsurface exploration methods and other detailed soil investigations, is



**FIGURE 39.6** Direct-shear test apparatus. From *Principles of Geotechnical Engineering, 3rd edition*, by B. M. Das © 1994. Reprinted with permission of Brooks/Cole, an imprint of the Wadsworth Group, a division of Thomson Learning.



**FIGURE 39.7** Shear strength for (a) granular soil (b) cohesive soil.

needed to supplement the predicted behavior based on generalized classification. Specific laboratory and field tests quantify indexing parameters and provide reasonable assurance of the anticipated performance of the soil as a construction medium.

### Pedologic Classification

Pedology is the study and analysis of soil profiles and includes soil chemistry, soil mineralogy, and soil physics. The uncertainty in estimating soil behavior, as discussed previously, is attributable to the fact that soil is non-homogeneous and non-isotropic. The non-homogeneous, non-isotropic characteristics of soil are the result of the way soils originate. The gradual conversion of rock to soil is the

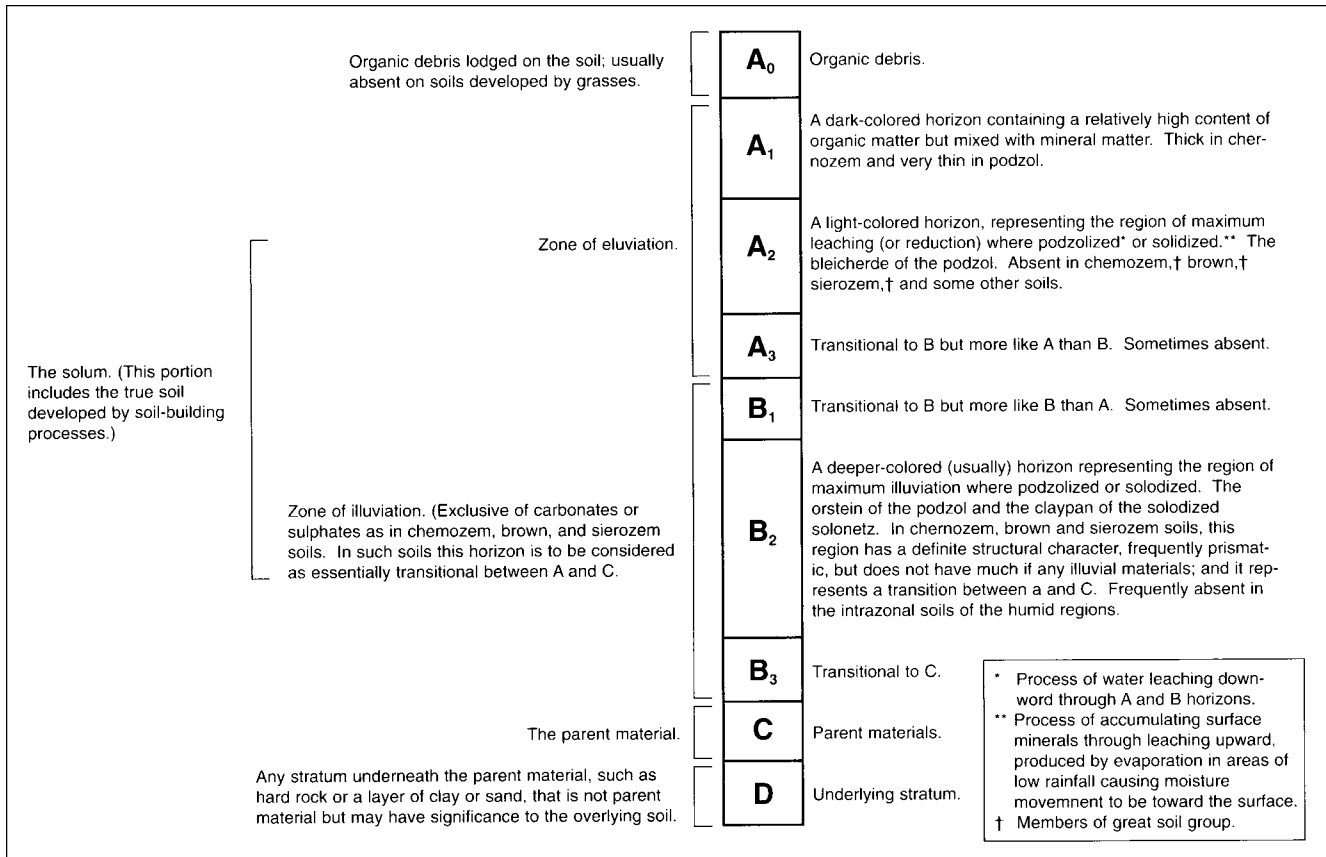
result of both the physical and chemical weathering process. Physical weathering results when rock is fragmented as a result of the freeze-thaw cycles of the water found in the soil's cracks and pore spaces. Abrasion also contributes to the breakdown of rocks into fragments and soil. Chemical weathering, considered perhaps as the most important of the two processes, is the result of chemically-active agents such as water, carbon dioxide ( $\text{CO}_2$ ), free oxygen ( $\text{O}_2$ ), and organic acids reacting with the mineral composition of the rock.

Soil formation is an evolutionary process involving the breakdown of parent-rock material by weathering from its interaction with environmental factors, such as climate, topography, and vegetation. Residual soils are those formed through natural decomposition of rock material. These soils are susceptible to being transported great distances by wind, water, ice, and gravity. New soils are formed when these transported soils are deposited, mix with the local soils and then undergo further weathering and consolidation. Conceivably, these new, mixed soils can themselves be transported, and the soil-formation process repeats itself.

The various methods of soil formation give soil its areal and vertical limitations. Transitions from one soil formation to another may be gradual or abrupt depending on formation processes, transport and deposition modes. In the vertical direction, deposits of transported soils result in strata layers. Soils formed in place (i.e., residual soils) create layers referred to as horizons.

A soil horizon is a layer of soil, approximately parallel to the surface of the Earth, that differs from adjacent, genetically-related horizons in physical, chemical or biological characteristics. The vertical variation of the horizons forms the soil profile. Each soil profile is comprised of three major horizons, within which there may be sub-horizons. The deepest horizon, the C-horizon, contains the relatively unweathered parent material. This is either residual material or material that has been transported through geologic processes. The upper A-horizon is comprised mostly of organic litter. Between the A and C horizon is the B-horizon. This horizon is partly soil from the A-horizon that has been washed and leached downward. It includes the true soil developed by the soil-forming processes. Each of the A and B horizons may be further subdivided into other horizons if other specific distinctions are present. These sub-horizons are denoted with numerical subscripts to the letter designation (e.g.,  $A_1$ ,  $A_2$ ,  $B_1$ ,  $B_2$  . . . etc.)

The depth of a soil profile depends on the processes that affect its formation. Both age and topography affect the depth of weathering. Profiles develop when moisture moves downward through the horizons. Steep slopes or low-rainfall areas result in shallow profile development. Older, more developed profiles occur on flat terrain where greater amounts of moisture movement are prevalent. Figure 39.8 illustrates a typical soil profile. Some of the sub-horizons, identified by the numerical subscripts, are transition zones between the adjacent major horizons. Typical of residual



**FIGURE 39.8** Typical soil profile. Reprinted with permission from PCA, 1973, *Soil Primer*, Skokie, Ill.: Portland Cement Association.

soils is the variation of gradation with depth. Particle sizes range from the fine-grained particles at horizons near the surface down to the rock fragments of the underlying parent material in the lower horizons.

Soil scientists describe soil horizons in terms of color, texture, structure, consistency, root and pore distribution, and the relative arrangement with adjacent soil layers. From the pedological perspective, the same parent material subjected to the same environmental conditions will produce identical soil profiles. This premise is the basis by which pedologists combine soils into related groups. Although there are different systems of pedologic classification, the one use by the U.S. Soil Conservation Service and reflected in the abundant county soil survey reports available nationwide was developed initially by the U.S. Department of Agriculture (USDA). Soil-survey reports are very useful during initial land-planning processes. The SCS classification system categorizes soil through six levels. Each subsequent level is more descriptive, and thus more definitive, for classification. Beginning with the broadest to narrowest the categories are:

1. Order: the 10 orders are differentiated according to soil morphology and genetic base
2. Suborder: divided primarily on the basis of chemical and physical properties that reflect (a) presence of

waterlogging or (b) genetic differences caused by climate and vegetation

3. Great groups: based on certain diagnostic horizons
4. Subgroup: three kinds of subgroups that relate to (a) the central concept of the great group, (b) transitional forms to the higher groups, (c) properties not representative of the great group or transitions
5. Family: based on plant growth properties such as texture, mineralogical composition and temperature
6. Series: the basic unit of classification consisting of soils essentially alike in all major profile characteristics except the texture of the A horizon. There are about 12,000 series names in the U.S.

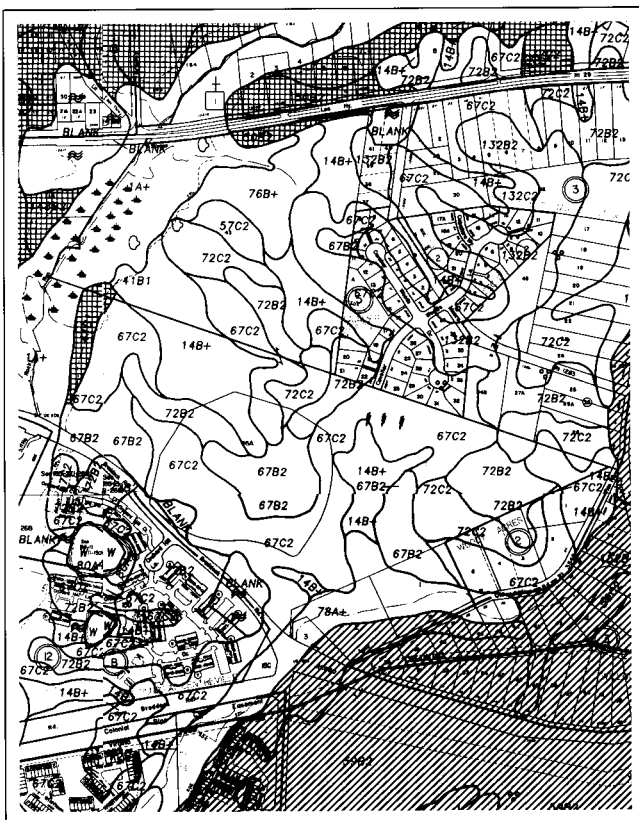
Soil series names typically are associated with local place names or a natural feature where the soil was recognized initially. The soil series names are the major component shown on detailed soil maps by the Soil Conservation Service (SCS). Essentially, a single-series name identifies the following features about a soil:

1. Parent material
2. Special features of parent material
3. Topographic position

4. Natural drainage qualities
5. Profile characteristics

Along with the series name, its texture or its geologic classification may also describe a soil. The properties associated with this classification then can be related to its engineering aspects. Such things as grain size, geology of the parent materials (including approximate depth to bedrock in residual soil), hydrology of the soil, relative infiltration rate, topography in terms of degree of slope and position in the landscape, and the performance of the soil as construction material based on past experiences with that same soil can be associated with the series name.

The soil designations can help determine where to perform extensive subsurface explorations and possibly how deep the explorations will need to be. Soil reports, published by the SCS, available in most areas, describe specific characteristics about the local soil series and their relationship to engineering design problems. Such characteristics as depth-to-water-table, erosion potential, rock excavation, need for subgrade stabilization and a host of other engineering interests are identified or can be inferred from the agricultural soil data. As an example, Figure 39.9 shows an SCS soils map. The accompanying table (Table 39.10) identifies the map soil codes by series name and correlates the soil series with engineering properties.



**FIGURE 39.9** CS map showing soil series.

## Geologic Classification

Physical properties of some soils can be inferred from the mode of transportation displacing the soils and the geologic factors that influenced this formation. The following is a brief description of the principal types of transported soils and geologic factors that influence their formation. Table 39.11 summarizes geologic description of soils.

*Alluvial and lacustrine soils* are formed by sedimentation. Alluvial soils are those deposited from running water and lacustrine soils are deposits in lakes. Deposits of alluvial soils tend to be well sorted because of the transport process of streams and rivers. A stream's capacity to transport sediment is a function of the velocity. Higher velocities move larger particles; hence, the particle size of the deposit is indicative of the channel velocity. Since the velocity can transport only specific particle sizes, those that are larger than the threshold size settle out. Because of this natural sorting process, alluvial and lacustrine deposits may be sources of gravel or sand quarries. Some areas of alluvial soils may have high water tables. The soils tend to be mainly medium-to-fine sands, silts and clays with poor drainage qualities. They tend to be soft, loose in density, and highly compressible and typically do not provide a good foundation.

*Aeolian soils* are wind-transported soils. These soils typically are found in semi-arid to arid regions. Dunes, which are sand deposits formed by wind, occur mainly in coastal zones and desert areas. Since dunes migrate in the direction of the prevailing winds, their formation consists of flat slopes on the windward side and steep slopes on the leeward side.

Finer silts transported by wind are called loess. This soil is characterized by loosely-arranged grains of sand, silt and clay, with silt as the character size. When dry, the grains are held together by a cementing agent, typically calcite, which may make up as much as 10% of the total soil. The loose structure with its numerous root holes usually produces vertical cleavage and high vertical permeability. Upon saturation, loess becomes unstable and compressible.

*Glacial soil* deposits are mostly heterogeneous mixtures of particle sizes ranging from boulders to clay. The soils have been eroded, pushed, or carried along with the glacier movement. Moraines describe those soil deposits that have been pushed or scraped along with the glacier. Glacial drifts are the soil deposits precipitated out of the ice or from the melt waters. Outwash are deposits carried away by the melting glacial waters. The character of glacial soils varies considerably, but in general, most glacial soils have good-to-fair bearing capacities and good-to-fair drainage.

## Textural Classification

From an engineering perspective, the primary interest soil is as a construction medium; that is, whether a soil will be effective as a foundation base, provide a stable embankment or cut slope, or serve as adequate backfill material. The

**TABLE 39.10 Typical Engineering Characteristics of Soils**

SOIL SERIES	SOIL No.	PARENT MATERIAL	POSITION FROM GROUND SURFACE		MOISTURE DENSITY		GRAIN SIZE DISTRIBUTION										PLASTICITY INDEX	CLASSIFICATION	
			DEPTH (in.)	HORIZON	MAXIMUM DRY DENSITY	OPTIMUM MOISTURE	PERCENTAGE PASSING SIEVE					PERCENTAGE SMALLER THAN						AASHTO	UNIFIED
							No. 3-in. (4.76 mm)	No. 4 (4.76 mm)	No. 10 (2.0 mm)	No. 40 (.42 mm)	No. 200 (0.74 mm)	.05 mm	.02 mm	.005 mm	.002 mm	LIQUID LIMIT			
Orange	59	Greenstone	0-9 9-23	Ap B21, B22	113	14	100	96	92	88	81	77	60	30	17	23	3	A-4(8)	ML
			23-41	B2m	91	28	100	97	93	90	77	54	72	30	19	29	9	A-4(8)	CI
Othello <sup>3</sup>	39	Coastal Plain sediments	2-10 12-30 35-40	A Bg C	120	11	100	100	99	99	62	57	—	19	12	18	4	A-4(5)	ML-CL
					113	15	100	100	99	71	68	—	38	31	35	21	A-6(12)	CL	
					112	13	—	—	100	38	28	—	10	9	NP	NP	A-4(1)	SM	
Penn <sup>2</sup> (fine sandy loam) <sup>4</sup>	67	Sandstone	0-8 8-15+	A C	—	—	—	—	—	—	—	—	—	—	18	2	A-2	SM	
					—	—	—	—	—	—	—	—	—	—	26	5	A-4	ML-CL	
Penn (loam) <sup>4</sup>	75	Siltstone and shaly sandstone	0-8 8-19	A C	113	14	100	99	93	84	78	—	26	16	27	4	A-4(8)	ML-CL	
					115	14	—	100	96	87	77	—	22	15	26	3	A-4(8)	ML	
Penn <sup>3</sup> (Shaly silt loam) <sup>4</sup>	77	Shale and sandstone	3-14 14-36	B and C C	112	15	89	72	55	51	51	—	23	13	31	7	A-4(3)	ML-CL	
					115	15	100	40	32	28	27	—	14	8	29	7	A-2-4(0)	GM-GC	
Penn (silt loam) <sup>4</sup>	73	Triassic shale and siltstone	0-8 8-19 19+	Ap C1 D	113	14	100	99	93	84	78	50	26	16	27	4	A-4(8)	ML-CL	
					115	14	—	100	96	87	77	46	22	15	26	3	A-4(8)	ML	
					117	12	100	35	28	24	10	—	—	—	32	11	A-2-6(0)	GC	
Raritan <sup>3</sup>	92	Alluvium on Triassic terrace shales and sandstone	0-8 8-37 37+	A B R	—	—	—	—	—	—	—	—	—	—	33	8	A-4	ML-CL	
					—	—	—	—	—	—	—	—	—	—	75	50	A-7-5	CH-ML	
					—	—	—	—	—	—	—	—	—	—	28	7	A-2-4	GM-GC	

Readington	273	Triassic shale and siltstone	0-7	Ap	100	21	100	99	93	84	82	68	42	30	43	16	A-7-6(11)	ML-CL
			12-28	B22	110	18	100	99	96	92	89	68	47	38	45	24	A-7-6(5)	CL
			34-48	C	113	16	97	95	91	86	84	68	43	32	38	19	A-6(12)	CL
Rowland	12	Alluvium from Triassic soils	0-8	Layer 1	112	16	100	98	91	75	70	49	23	14	27	4	A-4(8)	ML-CL
			14-21	Layer 2	112	15	100	100	94	85	76	51	24	16	27	5	A-4(8)	ML-CL
			21-36	Layer 3	115	15	100	100	96	52	44	32	20	15	25	3	A-4(3)	ML
Sassafraz	54	Sand, silt, and clay of Coastal Plain	4-10	A2	122	10	100	100	97	41	38	27	15	9	15	2	A-4(1)	SM
			14-26	B2	117	14	100	100	97	46	45	37	27	24	27	10	A-4(2)	SC
			38-50	C	111	12	100	100	97	13	13	13	11	10	NP	NP	A-2-4(0)	SM
Wahadkee	5	Alluvium	0-8	A	95-105	24-28	100	100	85-100	50-95	—	—	—	—	25-52	11-22	A-6, A-7	CL, MH
			8-92	B			100	100	90-100	50-85	—	—	—	—	30-45	11-20	A-6 A-7	ML, CH
Woodstown <sup>3</sup>	34	Coastal Plain sediments	6-11	A	117	12	100	100	90	35	36	—	15	12	20	5	A-4(0)	SM-SC
			22-36	B	125	10	100	100	89	38	38	—	19	14	20	6	A-4(1)	SM-SC
			36-60	C	121	10	100	100	86	20	19	—	11	9	NP	NP	A-2-4(0)	SM
Worsham	8	Colluvium from schist and granite	0-7	A	110	16	100	100	93	72	65	—	33	26	31	9	A-4(7)	ML-CL
			10-46	B	104	20	100	100	97	86	82	—	58	48	60	34	A-7-6(20)	CH
			46-62	C	113	14	100	100	92	68	62	—	41	35	48	26	A-7-6(14)	CL

<sup>1</sup>Samples were tested in accordance with standard procedures of the American Association of State Highway Transportation Officials (AASHTO) in a cooperative program involving State Highway Departments, Universities or Colleges, and the Bureau of Public Roads. Moisture density tests were performed in accordance with AASHTO Test Designation T99.57; Mechanical analyses according to the American Association of State Highway Transportation Officials Designation T88. AASHTO classification is based on "Standard Specifications for Highway Materials and Methods of Sampling and Testing" (Pt. 1, Ed. 7); "The Classification of Soils and Soil Aggregate Mixtures for Highway Construction Purposes." AASHTO Designation M145-49. Unified classification is based on the "Unified Soil Classification System," Technical Memorandum No. 3 357, Volume 1, Waterways Experiment Stations, Corps of Engineers March 1953.

<sup>2</sup>Soils of Prince William County, "Report No. 8; Prepared by Prince William County in cooperation with the Agronomy Department of Virginia Polytechnic Institute and the U.S.D.A. Soil Conservation Service.

<sup>3</sup>"Engineering Soil Classification for Residential Development", F.H.A. No. 373; revised November, 1961; compiled and edited by the Federal Housing Administration, Architectural Standards Division, Technical Studies Staff from data prepared by the Virginia Polytechnic Institute, Bureau of Public Roads, and State Highway Departments, Universities, and Colleges.

<sup>4</sup>Soil Series (ex. Bucks) are often separated during field mapping according to soil types (ex. Bucks Loam, and Bucks Silt Loam). The soil type is determined by the texture of the surface horizon, which often indicates differences in the soil parent material, and throughout the soil profile. These differences may affect the engineering properties of the Soil Series.

<sup>5</sup>"Soil Survey of Hanover County, Virginia", prepared by the U.S.D.A. Soil Conservation Service in cooperation with Virginia Polytechnic Institute and State University.

<sup>6</sup>Lab data obtained from Geotechnical Engineering Report, considered to be typical of "Marine clays" in Fairfax County, Virginia Liquid Limits typically range from 50-90, with Plasticity Indices greater than 30.

TABLE 39.11 Summary of the Principal Geologic Soil Deposits

MAJOR DIVISION	PRINCIPAL SOIL DEPOSITS	PERTINENT ENGINEERING CHARACTERISTICS
<i>Transported Soils</i>		
<u>Alluvial</u>	<u>Floodplain deposits.</u> Deposits laid down by a stream within that portion of its valley subject to inundation by floodwaters.	
Material transported and deposited by running water.	<u>Point bar.</u> Alternating deposits of arcuate ridges and swales (lows) formed on the inside or convex bank of mitigating river bends. Ridge deposits consists primarily of silt and sand; swales are clay-filled.	Generally favorable foundation conditions; however, detailed investigations are necessary to locate discontinuities. Flow slides may be a problem along riverbanks. Soils are quite pervious.
	<u>Channel fill.</u> Deposits laid down in abandoned meander loops isolated when rivers shorten their courses. Composed primarily of clay; however, silty and sandy soils are found at the upstream and downstream ends.	Fine-grained soils are usually compressible. Portions may be very heterogeneous. Silty soils generally present favorable foundation conditions.
	<u>Back swamp.</u> The prolonged accumulation of flood water sediments in flood basins bordering a river. Materials are generally clays but tend to become more silty near riverbank.	Relatively uniform in a horizontal direction. Clays are usually subject to seasonal volume changes.
	<u>Alluvial Terrace deposits.</u> Relatively narrow, flat-surfaced, river-flanking remnants of floodplain deposits formed by entrenchment of rivers and associated processes.	Usually drained, oxidized. Generally favorable foundation conditions.
	<u>Estuarine deposits.</u> Mixed deposits of marine and alluvial origin laid down in widened channels at mouths of rivers and influenced by tides of body of water into which they are deposited.	Generally fine-grained and compressible. Many local variations in soil conditions.
	<u>Alluvial-Lacustrine deposits.</u> Material deposited within lakes (other than those associated with glaciation) by waves, currents, and organo-chemical processes. Deposits consist of unstratified organic clay or clay in central portions of the lake or typically grade to stratified silts and sands in peripheral zones.	Usually very uniform in horizontal direction. Fine-grained soils generally compressible.
	<u>Deltaic deposits.</u> Deposits formed at the mouth of rivers, resulting in shoreline extension.	Generally fine-grained and compressible. Many local variations in soil condition.
	<u>Piedmont deposits.</u> Alluvial deposits at foot or hills or mountains. Extensive plains or alluvial fans.	Generally favorable foundation conditions.
..... <u>Glacial</u> Material transported and deposited by glaciers, or by melting water from the glacier.	<u>Glacial till.</u> An accumulation of debris, deposited beneath at the side (lateral moraines), or at the lower limit of a glacier (terminal moraine). Material lowered to ground surface in an irregular sheet by melting glacier is known as a ground moraine.	Consists of material of all sizes in various proportions from boulders and gravel to clay. Deposits are unstratified. Generally present favorable foundation conditions; but rapid changes in conditions are common.

**TABLE 39.11 Summary of the Principal Geologic Soil Deposits (Continued)**

MAJOR DIVISION	PRINCIPAL SOIL DEPOSITS	PERTINENT ENGINEERING CHARACTERISTICS
	<u>Glacio-Fluvial deposits.</u> Coarse and fine-grained material deposited by streams of meltwater from glaciers. Material deposited on ground surface beyond terminal of glacier is known as an outwash plain. Gravel ridges known as kames and eskers.	Many local variations. Generally present favorable foundation conditions.
	<u>Glacio-Lacustrine deposits.</u> Material deposited within lakes by meltwater from glaciers. Consisting of clay in central portions of lake and alternate layers of silty clay or silt and clay (varved clay) in peripheral zones.	Very uniform in a horizontal direction.
<u>Marine Material</u> transported and deposited by ocean waves and currents in shore and offshore areas.	<u>Shore deposits.</u> Deposits of sands and/or gravels formed by the transporting, destructive, and sorting action of waves on the shoreline. <u>Marine clays.</u> Organic and inorganic deposits of fine-grained material.	Relatively uniform and of moderate to high density.  Generally very uniform in composition. Compressible and usually very sensitive to remodeling.
<u>Eolian Material</u> transported and deposited by wind.	<u>Loess.</u> A calcareous, unstratified deposit of silts or sandy or clayey silt traversed by a network of tubes formed by root fibers now decayed.  <u>Dune sands.</u> Mounds, ridges, and hills of uniform fine sand characteristically exhibiting rounded grains.	Relatively uniform deposits characterized by ability to stand in vertical cuts. Collapsible structure. Deep weathering or saturation can modify characteristics.  Very uniform grain size; may exist in relatively loose conditions.
<i>Sedimentary Soils</i>		
<u>Residual Material</u> formed by disintegration of underlying parent rock or partially indurated material.	<u>Residual sands and fragments of gravel size</u> formed by solution and leaching of cementing material, leaving more resident particles; commonly quartz. <u>Residual clays</u> formed by decomposition of silicate rocks, disintegration of shales, and solution of carbonates in limestone. With few exceptions becomes more compact, rockier, and less weathered with increasing depth. At intermediate stage may reflect composition, structure, and stratification of parent rock.	Generally favorable foundation conditions.  Variable properties requiring detailed investigation. Deposits present favorable foundation conditions except in humid and tropical climates, where depth and rate of weathering are very great.
<u>Organic</u>	<u>Peat.</u> A somewhat fibrous aggregate of decayed and decaying vegetation matter having a dark color and odor of decay.	Very compressible. Entirely unsuitable for supporting building foundations.



TABLE 39.11 Summary of the Principal Geologic Soil Deposits (Continued)

MAJOR DIVISION	PRINCIPAL SOIL DEPOSITS	PERTINENT ENGINEERING CHARACTERISTICS
Accumulation of highly organic material formed in place by the growth and subsequent decay of plant life.	<u>Muck</u> . Peat deposits that have advanced in decomposition to such extent that the botanical character is no longer evident.	
<u>Colluvial Material</u> transported and deposited by gravity.	<u>Talus</u> . Deposits created by gradual accumulation of unsorted rock fragments and debris at base of cliffs. <u>Hillwash</u> . Fine colluvium consisting of clayey sand, sand silt, or clay. <u>Landslide deposits</u> . Considerable masses of soil or rock that have slipped down, more or less as units, from their former position on steep slopes.	Previous movement indicates possible future difficulties. Generally unstable foundation conditions.
<u>Pyroclastic Material</u> ejected from volcanoes and transported by gravity, wind, and air.	<u>Ejecta</u> . Loose deposits of volcanic ash, lapilli, bombs, etc. <u>Pumice</u> . Frequently associated with lava flows and mud flows, or may be mixed with nonvolcanic sediments.	Typically shardlike particles of silt size with larger volcanic debris. Weathering and redeposition produce highly plastic, compressible clay. Unusual and difficult foundation condition.

effectiveness of a soil for these purposes may be determined initially by its classification, using properties such as particle-size distribution and plasticity.

Particle-size distribution (i.e., the gradation of a soil sample) is a form of textural classification. In the textural sense, soil is classed as gravel, sand, silt and clay. These textural categories separate the soil according to grain size. Although these textural terms are commonly used by various organizations and agencies, the size limits for each particle class vary. Table 39.12 shows differences in particle-size limits for several selected agencies. Using the size limits of the USDA (sand: 2.0 mm–0.05 mm, silt: 0.05 mm–0.002 mm, clay: <0.002 mm), Figure 39.10 can be used to classify the soil once the relative weight (in percent) of each constituent is known.

Textural classification relates the physical properties of soils, particle size, shape, and soil composition. The particle-size distribution method is most effective for categorizing cohesionless (i.e., coarse-grained) soils, whereas cohesive soil (e.g., clays) characteristics are best described by properties related to their plasticity. Two of the most

widely-used methods for classifying soils, taking into account the particle-size distribution and plasticity, are the Unified Classification System (UCS) and the American Association of State Highway and Transportation Officials (AASHTO) System.

#### Unified Soil Classification System (ASTM D-2487)

The Unified Soil Classification (USC) system, described in ASTM D-2487, groups soil into two broad categories: coarse-grained soils and fine-grained soils. The letters *G*, *S*, *C* and *M* designate textural categories of gravel, sand, clay, and silt, respectively. Additionally, this system uses *W* and *P* to indicate a well-graded or poorly-graded soil, and *H* to indicate high or *L* for low plasticity. The underlying assumption for the USC system is that the engineering properties of any coarse-grained soil depends on the particle-size distribution. For fine-grained soils, the engineering properties are identified by the soil's plasticity.

To determine the classification using the USC system, a sieve and/or hydrometer analysis test, as described previously, is run and the results are compared to the appropriate

**TABLE 39.12 Particle-Size Limits for Selected Agencies**

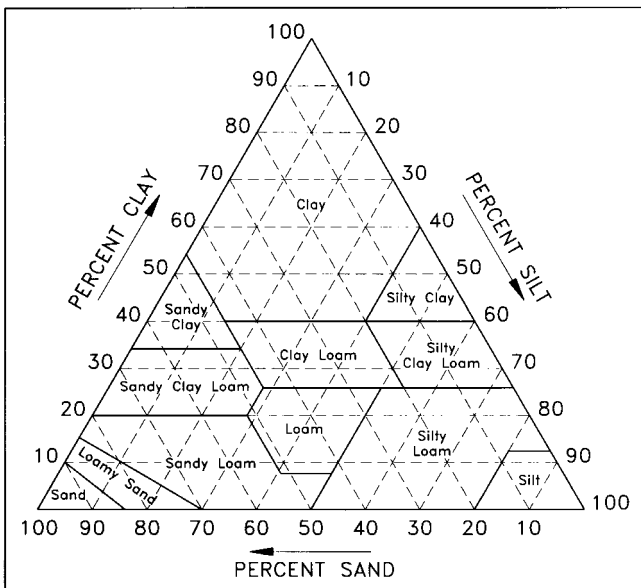
<b>American Society for Testing Materials</b>	Colloids	Clay	Silt	Fine Sand	Coarse Sand	Gravel			
	Colloids	Clay	Silt	Fine Sand	Coarse Sand	Fine Gravel	Medium Gravel	Coarse Gravel	Boulders
<b>American Association of State Highway and Transportation Officials Soil Classification</b>	Colloids	Clay	Silt	Fine Sand	Coarse Sand	Fine Gravel	Medium Gravel	Coarse Gravel	Boulders
	Clay	Silt	Very Fine Sand	Fine Sand	Medium Sand	Coarse Sand	Very Coarse Sand	Fine Gravel	Coarse Gravel
<b>U.S. Department of Agriculture Soil Classification</b>	Clay	Silt	Fine Sand	Coarse Sand	Gravel				
<b>Civil Aeronautics Administration Soil Classification</b>	Clay	Silt	Fine Sand	Coarse Sand	Gravel				
<b>Unified Soil Classification (Corps of Engineers, Department of the Army, and Bureau of Reclamation)</b>	Fines (silt or clay)			Fine Sand	Medium Sand	Coarse Sand	Fine Gravel	Coarse Gravel	Cobbles
	Fines (silt or clay)			Fine Sand	Medium Sand	Coarse Sand	Fine Gravel	Coarse Gravel	Cobbles
Sieve Size									
.001 .002 .003 .004 .006 .008 .01 0.02 0.03 0.04 0.06 0.08 0.1 0.140 0.2 0.3 0.4 0.6 0.8 1.0 2.0 10 4 6.0 8.0 10 1/2" 3/4" 20 30 40 60 80									
Particle Size, mm									

Reprinted with permission from Fang, Hsai-Yang, ed. 1991. *Foundation Engineering Handbook*, 2nd ed., New York: Van Nostrand Reinhold.

flow chart. First the soil is classified as to whether it is a coarse-grained soil or fine-grained soil. This is based on the weight fraction larger or smaller than the No. 200 sieve (0.075-mm).

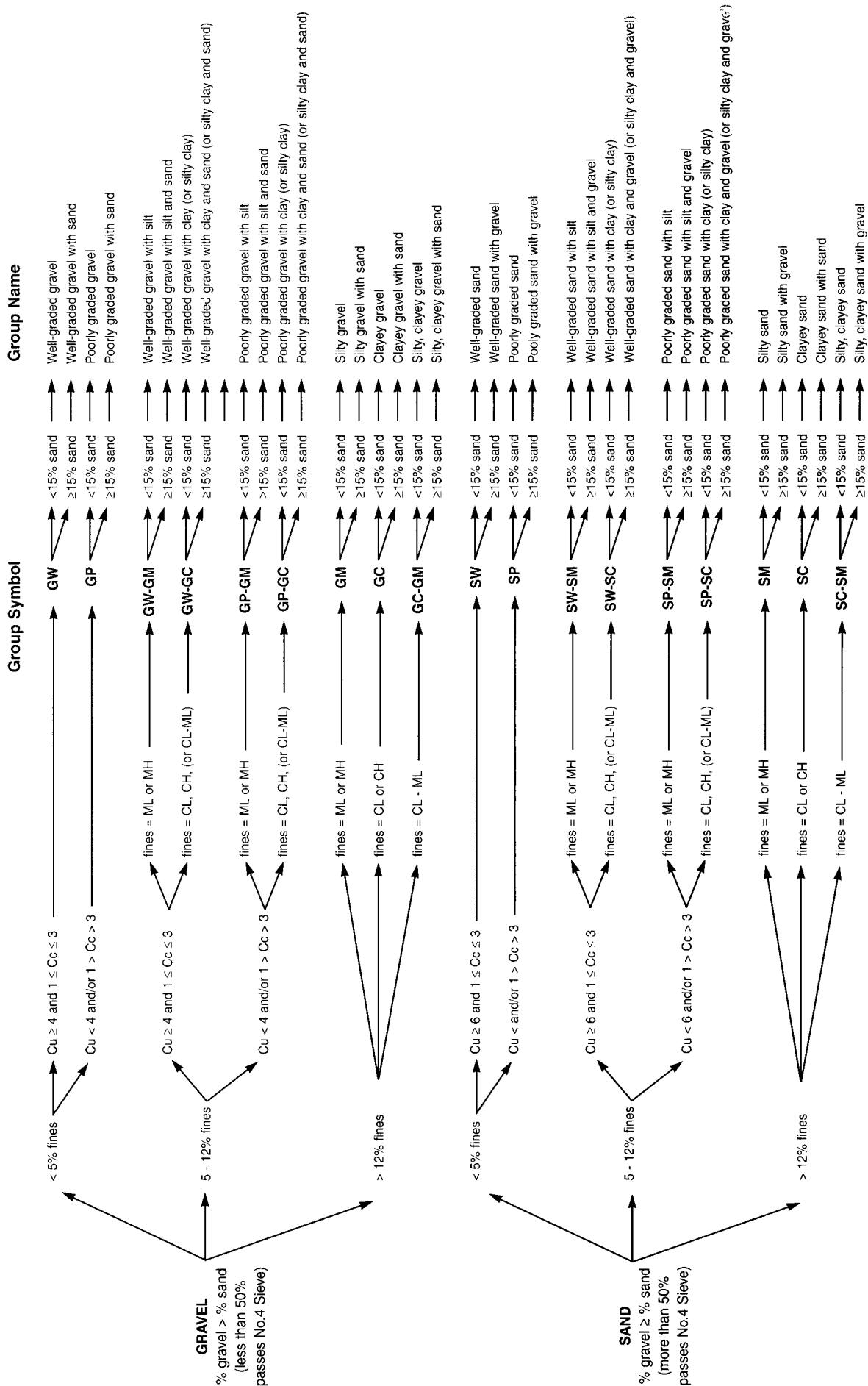
Coarse-grained soils are those that have 50% or more of the soil sample larger than 0.075 mm (No. 200 sieve). As shown the flowchart of Figure 39.11, coarse-grained soils are divided into either sands or gravels. If more than 50% of the coarse fraction is larger than 4.75 mm (No. 4 sieve), the sample is a gravel. For a sample with more than 50% smaller than the No. 4 sieve, the soil is a sand. The symbols G and S identify the soil as a gravel or sand. Gravels and sands are further classified according to gradation, including the amount and type of fines<sup>1</sup> in the sample. If the amount of fines is less than 12%, the uniformity coefficient,  $C_u$ , and gradation coefficient,  $C_c$ , are used to classify the soil further. For a sample with the amount of fines >12%, the plasticity chart of Figure 39.5 is used to classify the sample further.

The soil is classified as a fine-grained soil if 50% or more passes the No. 200 sieve. Figure 39.12 is used to determine the fine-grained soil classification. These soils are classed as clays (C) or silts (M) (M is from the Swedish word "mo" for

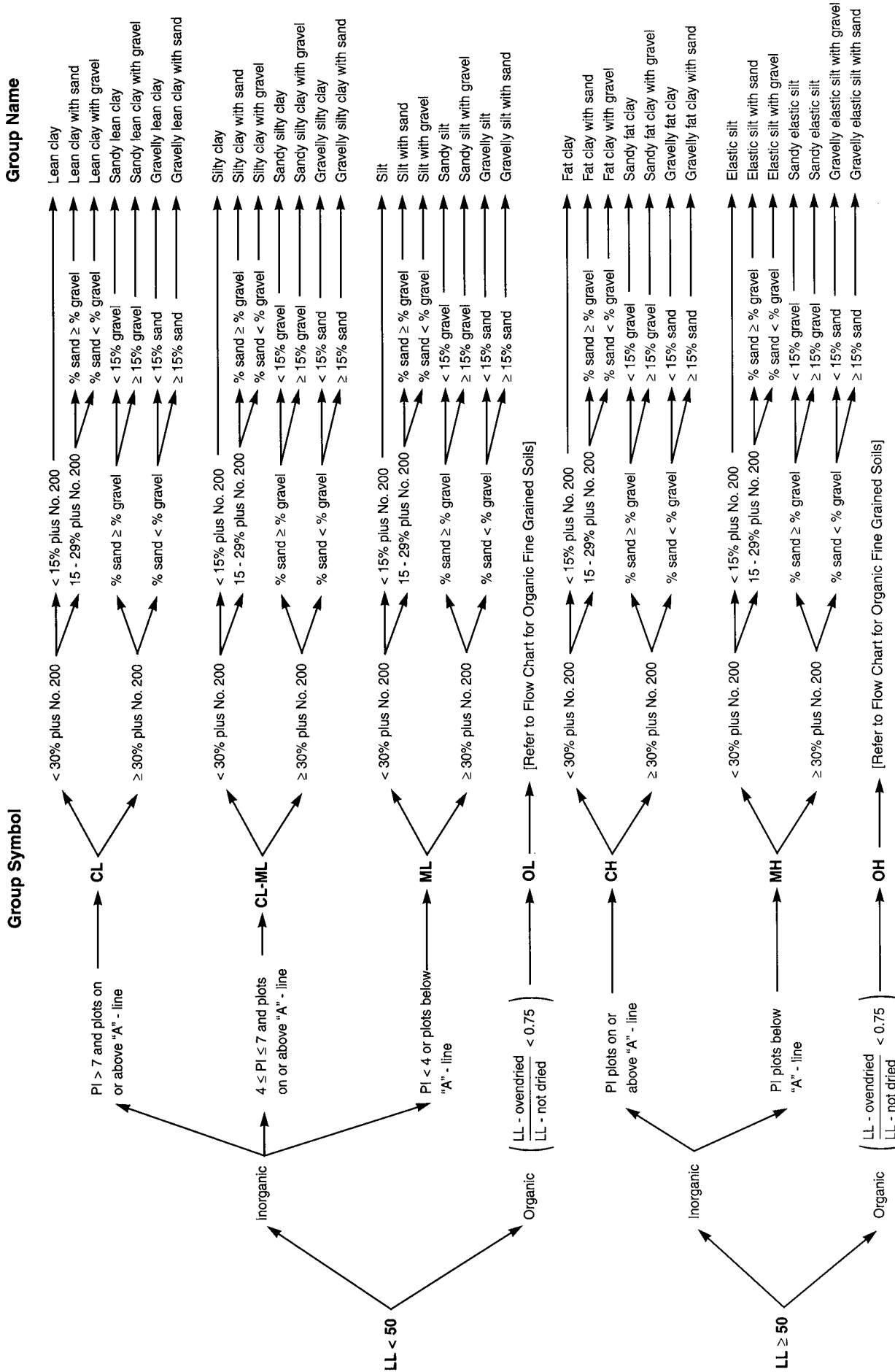


**FIGURE 39.10** USDA textural classification chart.

<sup>1</sup> Fines are soil particles smaller than 0.075 mm (No. 200 sieve).



**FIGURE 39.11** USC flow chart for classifying coarse-grained soils. Copyright ASTM. Reprinted with permission.



**FIGURE 39.12** USC flow chart for classifying fine-grained soil. Copyright ASTM. Reprinted with permission.

silt) according to the liquid limit and the plasticity index from Figure 39.5. Organic, silty and clayey soils are divided according to the liquid limit ( $LL$ ). Those with high  $LL$  (i.e.,  $> 50$ ) are  $CH$  or  $MH$ . A liquid limit less than 50 results in a soil designation as  $CL$  or  $ML$  ( $L$  indicates low plasticity). The flow chart for organic fine-grained soils (Figure 39.13) is used if

$$\frac{LL - \text{oven dried}}{LL - \text{not dried}} < 0.75 \quad (39.13)$$

Table 39.15 provides general guidelines on the properties of soils classified by the USC system.

### AASHTO Classification System (ASTM D-3282)

This system is described in ASTM D-3282 or AASHTO M65. This method of classification, popular with state and county highway departments, is used to classify soils based on laboratory test results. The American Association of State Highway and Transportation Officials (AASHTO) classifies soils into seven groups, designated as A-1 through A-7, based on the particle-size distribution, liquid limit and plasticity index. Classifications A-1 through A-3 are basically granular material with 35% or less of the particles smaller than 0.075 mm. Soil classifications A-4, A-5, A-6, and A-7 are silt and clay-type soils where more than 35% of the soil is finer than 0.075 mm (No. 200 sieve).

This system uses an empirically-determined, group-index parameter to classify the soil into a subgroup of one of the seven initial groups. The group index ( $GI$ ) is defined as

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10) \quad (39.14)$$

where  $F$  = % of soil particles passing 0.075 mm (No. 200) sieve expressed as a whole number (this percentage is based only on the material passing the 3-in. (75 mm) sieve);  $LL$  is the liquid limit in percent; and  $PI$  is the plasticity index expressed also as a percent. The first term of the equation is the partial group index determined from the liquid limit. The second term is the partial group index determined from the plasticity index. The following list applies in determining the group index number:

1. The group index is always entered as a number greater than or equal to zero. If the  $GI$  computes to be less than 0, the  $GI$  is taken as zero. The group index of soils the A-1-a, A-1-b, A-2-4, A-2-5 and A-3 is always zero.
2. The group index number is rounded to the nearest whole number.
3. For soils in the A-2-6 and A-2-7 subgroups, the  $GI$  is calculated using only the second term (Eq. (39.14)) containing the  $PI$  portion of the equation.

After the  $GI$  is computed, Figure 39.14 is used to determine the soil classification. To determine the correct soil group, read the diagram from left to right. The soil group that includes the range of the particle-size distribution and Atterberg limits of the sample is the correct group. In practice, the designation is given followed by the  $GI$  in parentheses (e.g., A-5 (9)).

Correlation between the USC system and the AASHTO classifications is shown in Table 39.13.

### SOIL COMPACTION

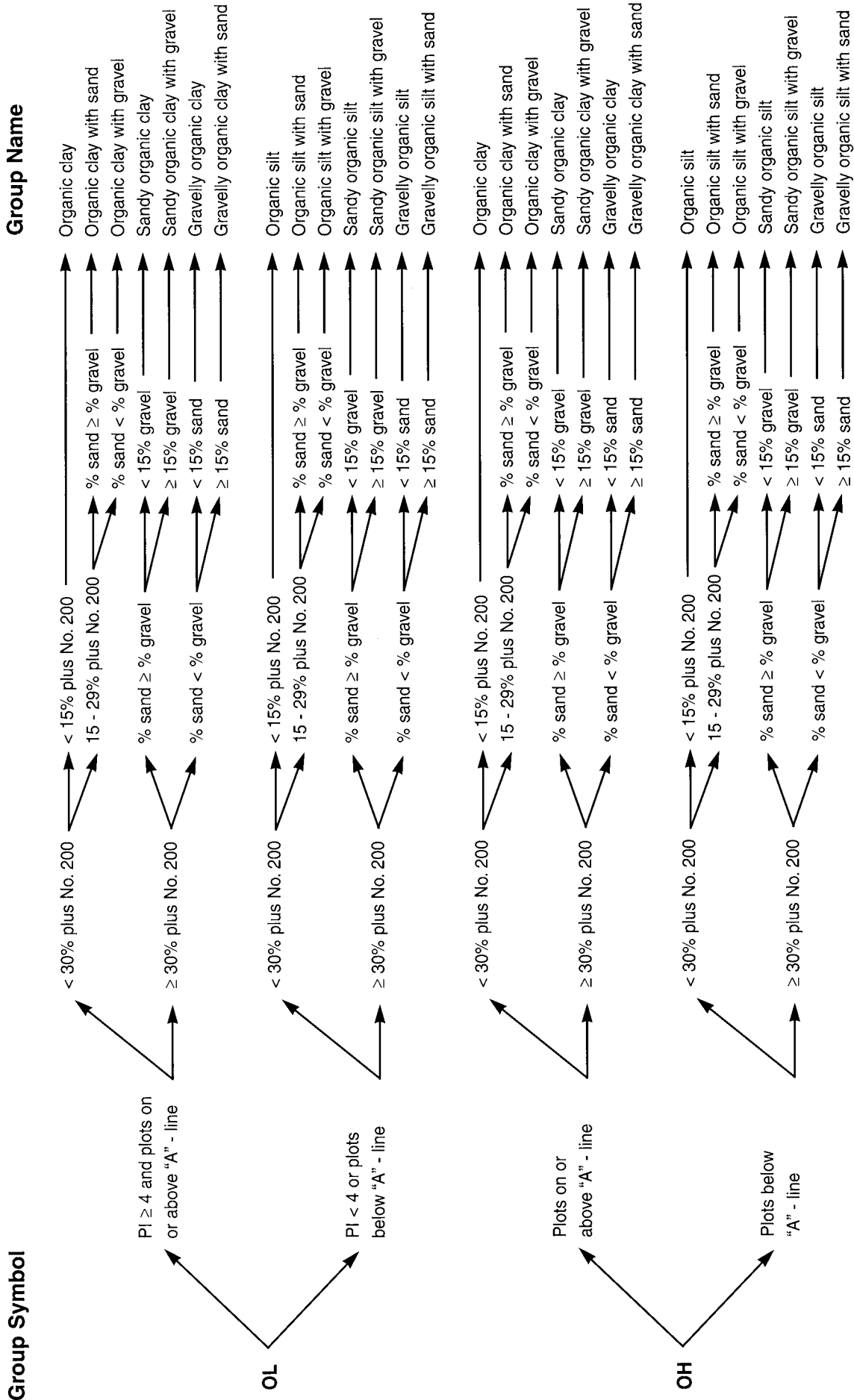
The stability of any structure founded on soil depends on the strength and stability of the soil itself. Engineering properties for in-situ soil can be markedly different than for the same soil that has been excavated and used elsewhere. In some cases the existing soil in its natural state is not suitable as a foundation base, whereas it may be if recompacted or prepared properly. In other cases, excavated soil may not be suitable for any reuse beyond landscaping purposes. The land design engineer relies on the recommendations of a geotechnical expert on the strength and stability of a soil and what methods might be used for improving its performance. Methods for increasing soil strength include adding material or chemicals to the soil or increasing the density by controlled compaction. Ground improvement techniques are discussed later in this chapter. For many land development projects, the land design engineer typically is concerned with the following recurring situations:

1. Bearing capacity and compressibility of the existing soil to sustain the structure loads of buildings or pavement
2. Bearing capacity and compressibility of excavated soil that is reused on the site
3. Slope stability of soil used as embankment
4. Slope stability of soil in cut areas

The strength of a soil depends on the void ratio and the moisture content of silty and clayey soil. Compaction usually increases the strength characteristics of soil, decreases the settlement potential, and increases slope stability of embankments by decreasing the void ratio and increasing the density of the soil. Minimum compaction requirements are identified in the specifications or dictated by local standards. Usually, the geotechnical consultant recommends compaction requirements in the instances where none exist in local standards.

### Compaction

Increasing soil density occurs by: rearrangement of the soil particles; fracture of the grains or bonds between them and then rearrangement; or bending the soil particles and their adsorbed layers. To do any of this requires energy—a compactive effort. The amount of energy needed depends on



**FIGURE 39.13** USC flow chart for classifying organic fine-grained soils. Copyright ASTM. Reprinted with permission.

CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES*											
General Classification	Granular Materials (35% or Less Passing No. 200)							Silt-Clay Materials (More than 35% Passing No. 200)			
	A-1		A-3	A-2				A-4	A-5	A-6	A-7
Group classification	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5
Sieve analysis, percent passing:											
No. 10	50 max.	—	—	—	—	—	—	—	—	—	—
No. 40	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40:											
Liquid limit	—	—	—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.	—	N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min. <sup>b</sup>
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

\* AASHTO (1986, 1988).  
<sup>b</sup> Plasticity index of A-7-5 subgroup is equal or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

**FIGURE 39.14** AASHTO classification system. From *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 14th edition*. Copyright 1992 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

**TABLE 39.13 - a Comparison of the AASHTO System with the Unified System\***

SOIL GROUP IN AASHTO SYSTEM	COMPARABLE SOIL GROUPS IN UNIFIED SYSTEM		
	MOST PROBABLE	POSSIBLE	POSSIBLE BUT IMPROBABLE
A-1-a	GW, GP	SW, SP	GM, SM
A-1-b	SW, SP, GM, SM	GP	—
A-3	SP	—	SW, GP
A-2-4	GM, SM	GC, SC	GW, GP, SW, SP
A-2-5	GM, SM	—	GW, GP, SW, SP
A-2-6	GC, SC	GM, SM	GW, GP, SW, SP
A-2-7	GM, GC, SM, SC	—	GW, GP, SW, SP
A-4	ML, OL	CL, SM, SC	GM, GC
A-5	OH, MH, ML, OL	—	SM, GM
A-6	CL	ML, OL, SC	GC, GM, SM
A-7-5	OH, MH	ML, OL, CH	GM, SM, GC, SC
A-7-6	CH, CL	ML, OL, SC	OH, MH, GC, GM, SM

Copyright ASTM.  
 Reprinted with permission.  
 After T. K. Liu (1967).  
 From *Principles of Geotechnical Engineering, 3rd edition*, by B. M. Das © 1994. Reprinted with permission of Brooks/Cole, an imprint of the Wadsworth Group, a division of Thomson Learning.

**TABLE 39.13-b Comparison of the Unified System with the Unified System\***

SOIL GROUP IN UNIFIED SYSTEM	COMPARABLE SOIL GROUPS IN AASHTO SYSTEM		
	MOST PROBABLE	POSSIBLE	POSSIBLE BUT IMPROBABLE
GW	A-1-a	—	A-2-4, A-2-5, A-2-6, A-2-7
GP	A-1-a	A-1-b	A-3, A-2-4, A-2-5, A-2-6, A-2-7
GM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6	A-4, A-5, A-6, A-7-5, A-7-6, A-1-a
GC	A-2-6, A-2-7	A-2-4, A-6	A-4, A-7-6, A-7-5
SW	A-1-b	A-1-a	A-3, A-2-4, A-2-5, A-2-6, A-2-7
SP	A-3, A-1-b	A-1-a	A-2-4, A-2-5, A-2-6, A-2-7
SM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6, A-4, A-5	A-6, A-7-5, A-7-6, A-1-a
SC	A-2-6, A-2-7	A-2-4, A-6, A-4, A-7-6	A-7-5
ML	A-4, A-5	A-6, A-7-5	—
CL	A-6, A-7-6	A-4	—
OL	A-4, A-5	A-6, A-7-5, A-7-6	—
MH	A-7-5, A-5	—	A-7-6
CH	A-7-6	A-7-5	—
OH	A-7-5, A-5	—	A-7-6
PT	—	—	—

Copyright ASTM.

Reprinted with permission.

\*After T. K. Liu (1967).

the soil material and the method used to apply the energy. Intra-particle bonds of cohesive soils resist the compactive efforts. Increasing the water content to an optimum value decreases the cohesive resistance and, hence, increases the effects of the compactive efforts. Distortion and reorientation of the particles affects the density of such soils. Resistance to compactive efforts of cohesionless soils is attributable to the intergranular friction that is aided by capillary tension. Increased density in these soils occurs mostly through rearrangement of the particles. Again, increasing the moisture content may increase the effects of the compactive efforts by decreasing capillary tension.

The degree of compaction is measured relative to the maximum dry-unit weight of the soil. Water added to the soil during compaction acts as a lubricant. The soil particles slip over each other and form a more dense arrangement. For a given soil and constant-compaction energy, the dry-unit weight increases as the moisture content increases up to a certain point. The increase unit weight is the result of fitting more and more soil particles into the same volume,

in effect decreasing the void spaces. Beyond a certain moisture content the unit weight of the soil begins to decrease because of the incompressibility of water. At this stage the moisture in the soil begins to occupy the space that could have been occupied by the soil particles. The moisture content, which maximizes the unit weight of the soil with a constant-compactive effort, is referred to as the optimum moisture content (OMC).

### Proctor Tests

The two most widely-used standard laboratory tests to determine the maximum dry-unit weight and optimum moisture content of a soil are the standard Proctor test and the modified Proctor test. The standard Proctor test (ASTM D-698, AASHTO T-99) consists of compacting a soil sample in a mold having a volume of  $\frac{1}{30}$  ft.<sup>3</sup> The compaction is done by dropping a weight of 5.5 lb from a height of 12 in. onto the soil sample. The weight is dropped 25 times onto each of three equal depth layers. For the given soil sample, the unit weight ( $\gamma$ ) is



$$\gamma = \frac{W}{V_m} \quad (39.15)$$

where  $W$  is the weight of the sample and  $V_m$  is the volume of the mold. The moisture content in percent is determined by the difference of the weight of the sample as tested and its weight after drying, as given by Eq. (39.16).

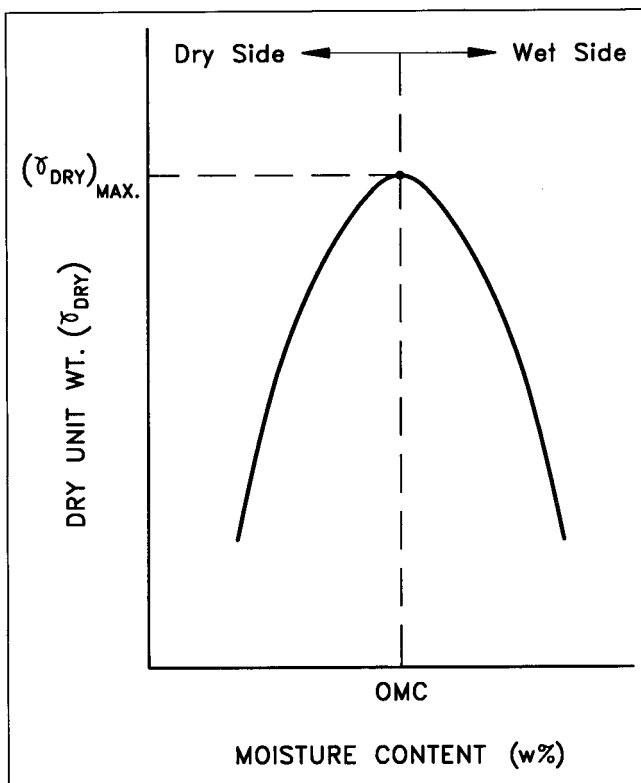
$$w = \frac{W_m - W_d}{W_d} \times 100 \quad (39.16)$$

where  $W_m$  is the weight of the moist sample and  $W_d$  is the weight of the sample after drying. The dry-unit weight ( $\gamma_d$ ) of the soil is then,

$$\gamma_d = \frac{\gamma}{1 + w} \quad (39.17)$$

where  $w$  is the moisture content expressed as a decimal.

The test is performed several times. Prior to running each test, the moisture content of the soil sample is increased by adding more water. The results of the test are plotted on a graph, with the moisture content on the ordinate and the dry-unit weight on the abscissa. This moisture-density relationship is referred to as a compaction curve. The compaction curve is normally bell-shaped, with a distinct maximum dry-unit weight at certain moisture content, as illustrated Figure 39.15. It must be noted, however, that not all soils have such distinct peaks.



**FIGURE 39.15** Standard Proctor compaction test results for silty clay sample.

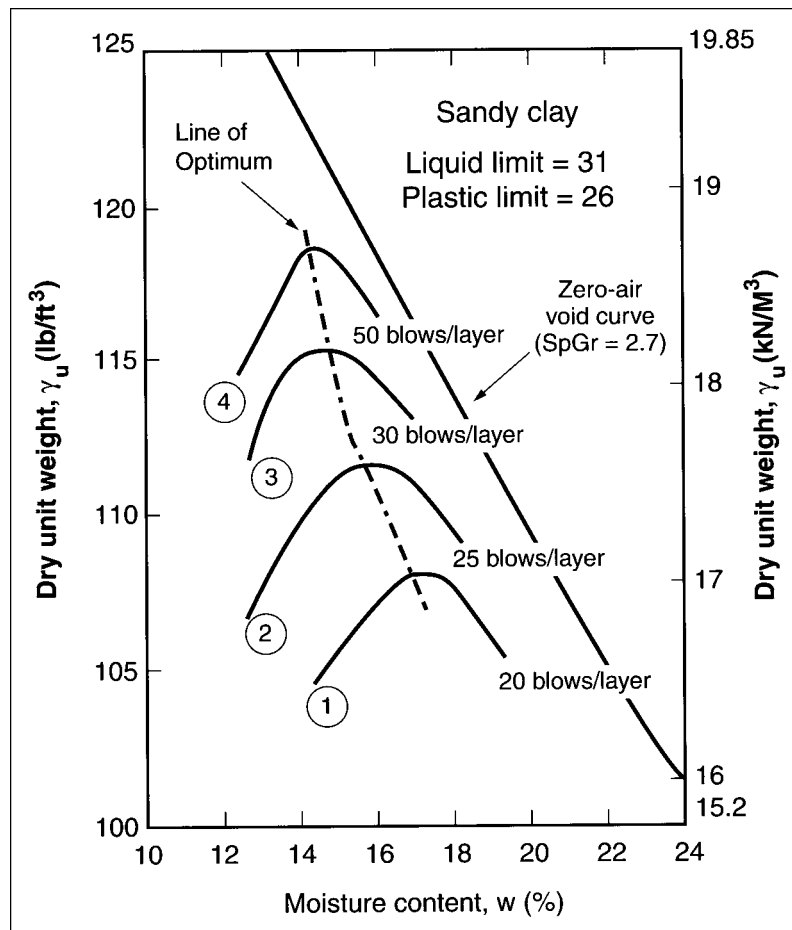
The curve of Figure 39.15 shows that for a given density, below the maximum, there are two corresponding values for the water content. Even though the theoretical density of the two values is the same, there is a difference in properties of a cohesive soil compacted on the dry side of OMC (i.e.,  $w < \text{OMC}$ ) compared to the same soil compacted on the wet side of OMC (i.e.,  $w > \text{OMC}$ ). These differences are attributable to the arrangement of the soil particles after compaction. At the lower water content, the soil particles have a flocculent, unorganized structural arrangement. At the higher water content, the platy clay particles become parallel, yet the soil is dispersed because of the presence of more moisture.

For the same compactive effort, a cohesive soil compacted on the dry side of OMC will usually be more permeable than the same soil compacted to the same density on the wet side of OMC. The unconfined compression strength is lower for cohesive soils compacted on the wet side of OMC as compared to the strength of a soil compacted on the dry side for the same density. The potential swell of clays increases with density and decreases with molding moisture. A clay compacted with a moisture content drier than the plastic limit has a greater potential for swell than if the clay soil is compacted wet. Limiting compaction and maintaining the moisture content at or above the plastic limit attains minimization of swell.

As illustrated in Figure 39.16, an increase in the compaction effort results in an increase in the maximum dry-unit weight of the soil. Additionally, the increased compactive effort results in the maximum dry-unit weight of the soil occurring at a lower moisture content. Since the time the Proctor test was first instituted, the capability to increase the compactive effort in the field has improved because of the increase in size and weight of compacting equipment. To account for the heavier machinery, the Proctor test has been changed slightly. The modified Proctor test (ASTM D-1557) simulates this increased capability by using a 10-lb hammer from a drop height of 18 in. The number of hammer blows is 25 per layer, and the number of equal-depth layers increases to 5. The results are an increase in the maximum dry-unit weight of the soil at a lower moisture content.

### Controlled Fill

For most soils, the soil will compact to its theoretically greatest density at an optimum moisture content. This optimum moisture content (OMC) is around 8% for sands, 15% for silts, and 15% to 20% for clays. Fill areas that support structures must be placed under rigidly controlled conditions in order to insure suitable bearing capacity and tolerable settlements. Typically, the controlled fill areas are identified on the plan sheets as "controlled fill—see note \*\*\*" where note \*\* refers to the compaction specification requirements relative to the maximum dry density and OMC of the fill material. Additionally, a maximum-lift (layer) thickness will also be specified.



**FIGURE 39.16** Effect of compaction energy on dry-unit weight. From *Principles of Geotechnical Engineering, 3rd edition*, by B. M. Das © 1994. Reprinted with permission of Brooks/Cole, an imprint of the Wadsworth Group, a division of Thomson Learning.

The specification for compaction might appear as “all fill shall be compacted to minimum 95% of the theoretical maximum density and within  $\pm 2\%$  of the optimum moisture content in accordance with the specifications of ASTM D-698.” Here, the theoretical maximum density is the density obtained from laboratory tests (i.e., standard Proctor). The 2% is the allowable tolerance of the moisture content from optimum. This allowable tolerance may vary somewhat depending on the compaction curve of the soil and the type of project. Specifying a percentage rather than a unit weight allows for the inherent variability of soils. This insures a compactive effort in the field comparable to the laboratory test. The required density of the placed fill depends on the purpose of the compacted soil. Considerable cost savings can be realized by matching the density to the purpose. Table 39.14 provides recommendations for compaction requirements.

Spreading a layer of soil to an appropriate thickness and using the proper equipment to compact the layer helps to control compaction during construction. When compaction of that layer is completed, another layer is spread and the equipment compacts the next layer. This process is repeated until the fill area is brought to the desired elevation. The

thickness of the soil layers, also known as “lifts,” depends on the type of soil. Cohesive soils typically are placed in lifts of 6 to 8 in. Lifts for well-graded gravel and sands are 10 to 12 in. The practice of compacting soils in small, uniform lifts helps maintain a uniform strength throughout the fill depth.

Controlling moisture content during construction is relatively easy for sandy soils and those that are drier than OMC. Dry soils are sprinkled with water before or as the soil is being compacted. The soil needs to be thoroughly mixed for even distribution of moisture. Soils wetter than the OMC can be spread to dry or mixed with other drier suitable soils before being placed. In general, plastic (clayey) soils are more sensitive to moisture content and must be compacted within a smaller range of the OMC to attain maximum density.

Specifications for controlled fill require that certain materials be excluded. These materials reduce the ultimate attainable strength or exacerbate settlement problems. Materials, such as soils, with excessive organic contents ( $>10\%$ ), trees, and vegetation cause excessive localized settlements after degradation. Large boulders, cobbles, flat

**TABLE 39.14 Compaction Requirements\***

<b>FILL UTILIZED FOR:</b>	<b>REQUIRED DENSITY % OF STANDARD PROCTOR</b>	<b>TOLERABLE RANGE OF MOISTURE ABOUT OPTIMUM, %</b>	<b>MAXIMUM PERMISSIBLE LIFT THICKNESS, COMPACTED (in.)</b>	<b>SPECIAL REQUIREMENTS</b>
Support of structure	95	-2 to +2	12	Fill should be uniform. Blending or processing of borrow may be required. For plastic clays, investigate expansion under saturation for various compaction moisture and densities of loads equal to those applied by structure, to determine condition to minimum expansion. Clays that show expansive tendencies generally should be compacted at or above optimum moisture to a density consistent with strength and incompressibility required of the fill.
Lining for canal or small reservoir	90	-2 to +2	6	For thick linings, GW-GC, GC, SC are preferable for stability and to resist erosive forces. Single size silty sands with PI less than five generally are not suitable. Remove fragments larger than 6 in. before compaction.
Earth dam greater than 50 ft high	95	-1 to +2	12(+)	Utilize least pervious materials as central core and coarsest materials in outer shells. Core should be free of lenses, pockets, or layers of pervious material and successive lifts well bonded to each other. Amounts of oversize exceeding 1% of total material should be removed from the borrow prior to arrival on the embankment.
Earth dam less than 50 ft high	92	-1 to +3	12(+)	In small dams that lack elaborate zoning, materials that are the most vulnerable to cracking and piping should be compacted to 98% density at moisture content from optimum to 3% in excess of optimum.
Support of pavements: Highways	95% within top 2.0 ft; 90% below top 2.0 ft	-2 to +2	8(+)	Place coarsest borrow materials at top of fill. Investigate expansion of plastic clays placed near pavement subgrade to determine compaction moisture and density that will minimize expansion and provide required soaked CBR values.
Backfill surrounding structure	90	-2 to +2	8(+)	Where backfill is to be drained, provide pervious coarse-grained soils. For low walls, do not permit heavy rolling compaction equipment to operate closer to the wall than a distance equal to about $\frac{2}{3}$ the unbalanced height of fill at any time. For highwalls or walls of special design, evaluate the surcharge produced by heavy compaction equipment by the methods of Chapter 3 and specify safe distances back of the wall for its operations.

**TABLE 39.14** Compaction Requirements\* (Continued)

<b>FILL UTILIZED FOR:</b>	<b>REQUIRED DENSITY % OF STANDARD PROCTOR</b>	<b>TOLERABLE RANGE OF MOISTURE ABOUT OPTIMUM, %</b>	<b>MAXIMUM PERMISSIBLE LIFT THICKNESS, COMPACTED (in.)</b>	<b>SPECIAL REQUIREMENTS</b>
Backfill in pipe or utility trenches	90	-2 to +2	8(+)	Material excavated from trench generally is suitable for backfill if it does not contain organic matter or refuse. If backfill is fine grained, a cradle for the pipe is formed in natural soil and backfill placed by tamping to provide the proper bedding. Where free-draining sand and gravel is utilized, the trench bottom may be finished flat and the granular material placed saturated under and around the pipe and compacted by vibration.
Drainage blanket or filter	90	Thoroughly wetted	8	Ordinarily vibratory compaction equipment is utilized. Blending of materials may be required for homogeneity. Segregation must be prevented in placing and compaction. For compaction adjacent to and above drainage pipe, use hand tamping or light travelling vibrators.
Subgrade of excavation for structure	95	-2 to +2	—	For uniform bearing or to break up pockets of frost-susceptible material, scarify the upper 8 to 12 in. of the subgrade, dry or moisten as necessary and recompact. Certain materials, such as heavily preconsolidated clays that will not benefit by compaction, or saturated silts and silty fine sands that become quick during compaction, should be blanketed with a working mat of lean concrete or coarse-grained material to prevent disturbance or softening. Depending on foundation conditions revealed in exploration, a substantial thickness of loose soils may have to be removed below subgrade and recompact, or compacted in place by vibration, or pile driving.
Rock fill		Thoroughly wetted	2 to 3 ft.	For fill containing sizes no larger than ft, place in layers not exceeding 24 in., thoroughly wetted and compacted by travel or heavy crawler tractors in spreading. Material with sizes up to 2 ft may be placed in 3 ft lifts. Placing should be such that the maximum size of rock increases toward the outer slopes. Rocks larger than 1 yd <sup>3</sup> in volume should be embedded on the slope.

## Notes:

1. Density and moisture content refer to "Standard Proctor" test values, (ASTM D 698)

2. Generally, a fill compacted dry of OMC will have higher strength and a lower compressibility even after saturation.

3. Compaction of "Coarse-grained, granular soil" is not sensitive to moisture content so long as bulking moisture is avoided. Where practicable, they should be placed saturated and compacted by vibratory methods.

\*NAVFAC DM7.2.

shaly fragments, and construction debris also adversely affect compaction.

Table 39.15 lists various soils and where their use may be most effective. Table 39.16 summarizes the USC classification of soils for use in roads and airfields.

### Methods for Monitoring Field Compaction

Specifications typically dictate the desired results for compaction in order to insure a result that will have a reliable performance. Part of the assurance, dictated by specifications, involves the field testing of in-place fill. Several standard methods exist for field testing the in-place soil to determine if the compaction efforts are producing specification quality.

The sand cone test (ASTM D-1556) and the rubber balloon test (ASTM D-2167) use material of known density to fill a small-excavated hole in the compacted area. In the sand cone test the excavated hole is filled with sand. The balloon test inserts a balloon into the hole and fills it with water. In either case the volume of the hole can be determined from the amount of sand or water used to fill the excavated hole. The excavated material is then weighed, oven dried, and weighed again to determine the water content. The results are then compared to the specification.

The most popular method of determining compaction uses a nuclear density gage to measure the density and water content. The gage consists of a radiation source, radiation detector, and counter. The gage emits radiation into the soil and the reflected emissions are counted. Knowing how the density and water reflect the emissions determines the unit weight and moisture content of the soil. The advantages of this method are that the test does not require a hole and that the results are available immediately.

An example will illustrate how data from the sand cone test measures compaction.

**EXAMPLE:** The test results from a sand cone test are shown in Table 39.17. Determine if the compaction meets the required specifications. The project specifications require compaction to 95% of the maximum dry density (MDD) with moisture content to be within 2% of OMC as determined by the Proctor test. Laboratory test results indicate the MDD as 119.5 lb/ft<sup>3</sup> at an OMC of 13.0%. At the project site, two tests were made. The sand used in the sand cone test had a density of 114.5 lb/ft<sup>3</sup>.

Specification requirements for this soil:

Dry density,  $\gamma_d = 0.95 \times 119.5 = 113.5$  lb/ft<sup>3</sup>

Moisture content,  $w$ : Min 11.0%, Max 15.0%

For test #1:

$$Vol_1 = \frac{9.05 \text{ lb}}{114.5 \text{ lb/ft}^3} = 0.079 \text{ ft}^3 \quad (39.18)$$

$$w_1 = \frac{9.66 - 8.75}{8.75} \times 100 = 10.4\%$$

$$\gamma_{d1} = \frac{8.75 \text{ lb}}{0.079 \text{ ft}^3} = 110.8 \text{ lb/ft}^3$$

For test #2:

$$Vol_2 = \frac{7.25 \text{ lb}}{114.5 \text{ lb/ft}^3} = 0.063 \text{ ft}^3 \quad (39.19)$$

$$w_2 = \frac{8.45 - 7.41}{7.41} \times 100 = 14.0\%$$

$$\gamma_{d2} = \frac{7.41 \text{ lb}}{0.063 \text{ lb/ft}^3} = 117.6 \text{ lb/ft}^3$$

The results show that compaction in the area of test #1 does not meet specifications. The moisture content is less than 2% of optimum and the unit weight is less than 95% MDD. Test #2 meets the compaction requirements.

### Compaction Equipment

In the laboratory, soil type and moisture content are the main factors affecting maximum density. However, in the field other factors, which cannot be as rigidly controlled as in the laboratory, affect the actual degree of compaction. Such factors as the thickness of the lift, intensity of pressure applied, area of applied pressure, and number of times pressure is applied all determine the ultimate unit weight attained at the project site. Effective compaction is attained by applying a large enough force to reorient and densify the soil particles; however, too large of a force can cause the soil mass itself to be sheared.

Rollers are the primary method for field compaction. Rollers compact the soil by expelling air from the voids and forcing the soil particles into a compact arrangement. Common rollers are: smooth-drum roller, pneumatic rubber-tired roller, sheepfoot roller, and vibratory roller. Figure 39.17 shows a vibratory smooth drum and sheepfoot roller working in tandem to compact an engineered fill. Rollers are available in various sizes and weights. Roller weights can be varied somewhat by filling the drums or body with fluid or sand. The type of roller used depends on soil characteristics, maneuvering space, intended function of the embankment, and economic considerations.

Smooth-drum rollers are best suited for finish rolling of subgrades. The drum rollers provide 100% coverage with contact pressures up to 55 lb/ft<sup>2</sup> in. Although these rollers can be used for all but rocky soils, they are most effective on sandy and silty soils. Smooth-drum rollers do not produce high unit weights of compaction if used on thick lifts and are ineffective for compacting cohesive soils in earth dams. The smooth soil surfaces between lifts are potential planes of weakness for the lateral hydrostatic forces and seepage paths.

The pneumatic rubber-tired roller is a heavily-loaded wagon with rows of four to six tires spaced closely together. Contact pressures in these rollers range from 100 to 150 psi and provide 70% to 80% coverage. Heavy rubber-tired rollers are not recommended for initial rolling of highly plastic clays. Best effects of these rollers are on soils from clean sand to silty clays. Compaction is achieved through the combined effects of the pressure and kneading action.

**TABLE 39.15 Engineering Uses for USC Soils**

TYPICAL NAMES OF SOIL GROUPS		RELATIVE DESIRABILITY FOR VARIOUS USES*													
		IMPORTANT PROPERTIES					ROADWAYS								
		PERMEABILITY WHEN COMPACTED	COMPRESSION WHEN COMPACTED AND SATURATED	WORKABILITY AS A CONSTRUCTION MATERIAL	ROLLED EARTH DAMS		CANAL SECTIONS		FOUNDATIONS		FILLS				
GROUP SYMBOLS	WHEN COMPACTED	SATURATED	COMPACTED AND SATURATED	CONSTRUCTION MATERIAL	HOMOGENEOUS EMBANKMENT	CORE	SHELL	EROSION RESISTANCE	COMPACTED EARTH LINING	SEEPAGE IMPORTANT	SEEPAGE NOT IMPORTANT	FROST HEAVE NOT POSSIBLE	FROST HEAVE POSSIBLE	SURFACING	
Well-graded gravels, gravel-sand mixtures, little or no fines	GW	pervious	excellent	negligible	excellent	—	1	1	—	—	1	1	1	1	3
Poorly graded gravels, gravel-sand mixtures, little or no fines	GP	very pervious	good	negligible	good	—	2	2	—	—	3	3	3	3	—
Silty gravels, poorly graded gravel-sand-silt mixtures	GM	semipervious to impervious	good	negligible	good	2	4	4	4	1	4	4	4	9	5
Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	impervious	good to fair	very low	good	1	1	3	1	2	6	5	5	1	1
Well-graded sands, gravelly sands, little or no fines	SW	pervious	excellent	negligible	excellent	—	3 if gravelly	6	—	—	2	2	2	2	4
Poorly graded sands, gravelly sands, little or no fines	SP	pervious	good	very low	fair	—	4 if gravelly	7 if gravelly	—	—	5	6	4	4	—
Silty sands, poorly graded sand-silt mixtures	SM	semipervious to impervious	good	low	fair	4	5	8 if gravelly	5 erosion critical	3	7	8	10	6	6

**TABLE 39.15 Engineering Uses for USC Soils**

		RELATIVE DESIRABILITY FOR VARIOUS USES*															
		IMPORTANT PROPERTIES						ROADWAYS									
TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	PERMEABILITY WHEN COMPACTED	SHEARING STRENGTH WHEN COMPACTED AND SATURATED	COMPRESSIBILITY WHEN COMPACTED AND SATURATED	WORKABILITY AS A CONSTRUCTION MATERIAL	ROLLED EARTH DAMS			CANAL SECTIONS			FOUNDATIONS			FILLS		
						WHEN COMPACTED	SATURATED	HOMOGENEOUS EMBANKMENT	CORE	SHELL	EROSION RESISTANCE	COMPACTED EARTH LINING	SEEPAGE IMPORTANT	SEEPAGE NOT IMPORTANT	FROST HEAVE NOT POSSIBLE	FROST HEAVE POSSIBLE	EROSION RESISTANCE
Clayey sands, poorly graded sand-clay mixtures	SC	impervious	good to fair	low	good	3	2	—	5	2	4	8	2	7	6	2	
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	semipervious to impervious	fair	medium	fair	6	6	—	—	6 erosion critical	6	9	10	11	—		
Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL	impervious	fair	medium	good to fair	5	3	—	9	3	8	10	9	7	7		
Organic silts and organic silt-clays of low plasticity	OL	semipervious to impervious	poor	medium	fair	8	8	—	—	7 erosion critical	7	11	11	12	—		
Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	semipervious to impervious	fair to poor	high	poor	9	9	—	—	—	8	12	12	13	—		
Inorganic clays of high plasticity, fat clays	CH	impervious	poor	high	poor	7	7	—	10	8 volume change critical	9	13	13	8	—		
Organic clays of medium to high plasticity	OH	impervious	poor	high	poor	10	10	—	—	—	10	14	14	14	—		
Peat and other highly organic soils	Pt	—	—	—	—	—	—	—	—	—	—	—	—	—	—		

From Soil Mechanics, Lambe, T.W. and R.V. Whitman. Copyright 1979. Reprinted by permission of John Wiley & Sons, Inc.  
 \*No. 1 is most desirable, no. 14 is least desirable.

**TABLE 39.16 Soil Characteristics Pertinent to Roads and Airfields**

MAJOR DIVISIONS	LETTER	NAME	VALUE AS				POTENTIAL FROST ACTION	COMPRESSION AND EXPANSION	DRAINAGE CHARACTERISTICS	COMPACTION EQUIPMENT	TYPICAL DESIGN VALUES		
			FRONT ACTION	WHEN NOT SUBJECT TO FROST ACTION	WHEN NOT SUBJECT TO FROST ACTION	WHEN NOT SUBJECT TO FROST ACTION					UNIT DRY WEIGHT (lb./ft. <sup>3</sup> )	CBR	SUBGRADE MODULUS (lb./in. <sup>2</sup> )
Coarse-grained soils Gravel and gravelly soils	GW	Well-graded gravels or gravel-sand mixtures, few or no fines	Excellent	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125-140	40-80	300-500	
			Good to excellent	Good	Fair to good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110-140	30-60	300-500	
	GM d*	Silty gravels, gravel-and-silt mixtures	Good to excellent	Good	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepfoot roller, close control of moisture	125-145	40-60	300-500	
Sand and sandy soils	U*	Clayey gravels, gravel-sand clay mixtures	Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepfoot roller	115-135	20-30	200-500	
			Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepfoot roller	130-145	20-40	200-500	
Sand and gravelly soils	SW	Well-graded sands or gravelly sands, few or no fines	Good	Fair to good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	110-130	20-40	200-400	
			Fair to good	Fair	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	105-135	10-40	150-400	
Silty sands, sand silt mixtures	SM d*	Silty sands, sand silt mixtures	Fair to good	Fair to good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepfoot roller, close control of moisture	120-135	15-40	150-400	
			Fair to good	Fair to good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepfoot roller, close control of moisture	120-135	15-40	150-400	



TABLE 39.16 Soil Characteristics Pertinent to Roads and Airfields (Continued)

MAJOR DIVISIONS	LETTER	NAME	VALUE AS		POTENTIAL FROST ACTION	COMPRESSIBILITY AND EXPANSION	DRAINAGE CHARACTERISTICS	COMPACTION EQUIPMENT	TYPICAL DESIGN VALUES		
			SUBGRADE WHEN NOT SUBJECT TO FRONT ACTION	WHEN NOT SUBJECT TO FROST ACTION					UNIT DRY WEIGHT (lb/ft <sup>3</sup> )	CBR	SUBGRADE MODULUS (lb/in. <sup>2</sup> )
	<i>U</i> *		Fair	Poor to fair	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepfoot roller	100–130	10–20	100–300
	SC	Clayey sands, sand-clay mixtures	Poor to fair	Poor	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepfoot roller	100–135	5–20	100–300
Fine-grained soils											
Silts and clays 1.1 is less than 50	ML	Inorganic silty and very fine sands, rock floor, silty or clayey silts with slight plasticity	Poor to fair	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepfoot roller, close control of moisture	90–130	15 or less	100–200
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepfoot roller	90–130	15 or less	50–150
	OL	Organic silts and organic silt clays of low plasticity	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired roller, sheepfoot roller	90–105	5 or less	50–100
Silts and clays 1.1 is greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller, rubber-tired roller	80–105	10 or less	50–100
	CH	Inorganic clays of high plasticity, fat clays	Poor to fair	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	90–115	15 or less	50–150
	OH	Organic clays of medium to high plasticity, organic soils	Poor to very poor	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	80–110	5 or less	25–100
Highly organic soils	Pt	Peat and other highly organic soils	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical			

Courtesy of The Asphalt Institute. *Soils Manual for Design of Asphalt Pavement Structures*. MS-10, College Park, Maryland 1986.

*U*\* indicates  $LL \leq 25$  and  $PI \leq 5$ , otherwise *U*\*.

**TABLE 39.17 Sand Cone Test Results**

	WET WT.	DRY WT.	SAND WT.
HOLE #1	9.66 lb	8.75 lb	9.05 lb
HOLE #2	8.45 lb	7.41 lb	7.25 lb

Sheepsfoot rollers have drum wheels with a large number of protrusions. The drums can be filled with water to provide the contact pressure. The area of each of the protrusions range from 5 to 12 in.<sup>2</sup> and provide contact pressures of 200 to 1000 psi. Sheepsfoot rollers are most effective in compacting clayey soils. Initial passes of the roller over the fill area compact the lower portion of the lift, while compaction at the top and middle of the lift occurs after subsequent passes.

Vibratory rollers are effective in compacting granular soils and rockfill material for earth dams. The rollers compact the soil through dead weight and the dynamic force of the vibration. To be effective, the frequency and dead weight of the roller must be matched with the type of material being compacted. Heavy-weight, low-frequency vibration rollers are used for gravel and rock-fill materials and clays. Light to medium-weight rollers with high-frequency vibrations are used for sands.

A summary of the various types of compaction equipment and their application to specific soils is given in Table 39.18.

Several key points should be mentioned regarding the above discussion. The pressure applied at the surface decreases with depth in the soil mass. A high pressure applied over a small area decreases more rapidly with depth than a moderate pressure over a larger area. Therefore, the height of the lift should be coordinated to the amount and method of applied pressure.

The amount of pressure that causes shear failure in the soil mass is not always evident from test data. In the field,

rutting of the surface from the drum or rubber tires, or failure of the sheepsfoot to rise out of the ground, indicates excessive compaction pressures for the given moisture content. The unit weight of a soil progressively increases with the number of roller passes up to a point. Beyond this point, the incremental increase in compaction decreases with more roller passes. Typically, 10 to 15 roller passes will provide the maximum dry-unit weight economically attainable.

### California Bearing Ratio Test (ASTM D-1883)

The California Bearing Ratio (CBR) test, or some variation of it, is one of the most widely-used tests to determine the thickness of base, subbase, and subgrade materials used in the design of pavements. The CBR is a measure of the shear resistance of a soil.

The test consists of measuring the load required to cause a standard-size plunger to penetrate a properly-prepared soil sample at a specified rate. This value is expressed as a percentage of the load required to cause the same plunger to penetrate the same depth into a standard sample of crushed stone. The CBR of a soil depends on the type of soil, its density, molding moisture content, and moisture content after soaking. Therefore, it is important to keep these parameters in the field consistent with the values as they are tested in the laboratory. A generalized range of CBR values for various soils is listed in Table 39.16.

### LATERAL-EARTH PRESSURE AND RETAINING WALLS

Lateral-earth pressure is a significant design parameter associated with land development. Retaining walls, foundation walls, braced and unbraced excavations, bridge abutments, underground tunnel structures and their head-wall and wingwalls are all subject to lateral-earth pressures and thus require a quantitative estimate of the forces acting on the structure (Bowles, 1982).

Earth-retaining structures facilitate an abrupt change in grade where two neighboring masses of earth must be



**FIGURE 39.17** Vibratory and sheepsfoot rollers. (Photo Credit: Frost)

TABLE 39.18 Compaction Equipment and Methods

**REQUIREMENTS FOR COMPACTION OF 95 TO 100% STANDARD PROCTOR  
MAXIMUM DENSITY**

<b>EQUIPMENT TYPE</b>	<b>APPLICABILITY</b>	<b>COMPACTED LIFT THICKNESS, IN.</b>	<b>PASSES OR COVERAGES</b>	<b>DIMENSIONS AND WEIGHT OF EQUIPMENT</b>		<b>POSSIBLE VARIATIONS IN EQUIPMENT</b>
Sheepsfoot rollers	For fine-grained soils or dirty coarse-grained soils with more than 20% passing the no. 200 sieve. Not suitable for clean coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important	6	4-6 passes for fine-grained soil; 6-8 passes for coarse-grained soil	Soil type	Foot contact area, in <sup>2</sup>	Foot contact pressures, psi
				Fine-grained soil $PI > 30$	5-12	250-500
				Fine-grained soil $PI < 30$	7-14	200-400
				Coarse-grained soil	10-14	150-250
						For earth dam, highway and airfield work, drum of 60-in. diameter, dloaded to 1.5-3 ton per lineal foot of drum, is generally utilized. For smaller projects 40-in. diameter drum, loaded to 0.75-1.75 tons per lineal foot of drum is used. Foot contact pressure should be regulated so as to avoid shearing the soil on the third or fourth pass.
Rubber tire rollers	For clean, coarse-grained soils with 4-8% passing the no. 200 sieve. For fine-grained soils or well-graded, dirty, coarse-grained soils with more than 8% passing the no. 200 sieve.	10  6-8	3-5 coverages  4-6 coverages	Efficient compaction of soils wet of optimum requires less contact pressures than the same soils at lower moisture contents.		Wide variety of rubber-tire compaction equipment is available. For cohesive soils, light wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel load if lift thickness is decreased. For cohesionless soils, large-size tires are desirable to avoid shear and rutting.

Smooth wheel rollers	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures. May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.	8-12  6-8	4 coverages  6 coverages	Tandem type rollers for base course or subgrade compaction, 10-15 ton weight, 300-500 lb per lineal inch of width of rear roller  3-wheel roller for compaction of fine-grained soil; weights from 5-6 tons for materials of low plasticity to 10 tons for materials of high plasticity	3-wheel rollers obtainable in wide range of sizes. 2-wheel tandem rollers are available in the range of 1-20-ton weight. 3-axle tandem rollers are generally used in the range of 10- to 20-ton weight. Very heavy rollers are used for proof rolling of subgrade or base course.
Vibrating baseplate compactors	For coarse-grained soils with less than about 12% passing the no. 200 sieve. Best suited for materials with 4-8% passing no. 200, placed thoroughly wet.	8-10	3 coverages	Single pads or plates should weigh no less than 200 lb. May be used in tandem where working space is available. For clean coarse-grained soil, vibration frequency should be no less than 1600 cycles/min.	Vibrating pads or plates are available, hand-propelled or self-propelled, single or in gangs, with width of coverage from 1½ to 15 ft. Various types of vibrating-drum equipment should be considered for compaction in large areas.
Crawler tractor	Best suited for coarse-grained soils with less than 4-8% passing no. 200 sieve, placed thoroughly wet.	10-12	3-4 coverages	No smaller than D8 tractor with blade, 34,500-lb weight, for high compaction	Tractor weights up to 60,000 lb.
Power tamper or rammer	For difficult access, trench backfill. Suitable for all inorganic soils	4-6 in. for silt or clay, 6 in for coarse-grained soils	2 coverages	30-lb minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weights up to 250 lb; for diameter 4-10 in.

maintained at differing elevations (Rico, 1988). The urban development setting has promulgated the use of retaining structures to allow greater unit density. When designed properly, retaining structures, although not the most economical solution when compared to large-scale grade changes, may be the only feasible engineering solution in many cases.

Preliminary cost estimates and comparisons to alternative (non-wall) solutions often reveal retaining-wall solutions that appear economical. "However, on detailed analyses of the design drawings and cost estimates it is often observed that the construction of the retaining wall has been given only trivial consideration, without paying attention to the often costly details" (Rico 1988). Many wall designs are engineered without proper consideration of substantial and effective wall drains, control of backfill material and proper evaluation of special foundation conditions. Improperly-designed walls are pre-disposed to failure and a greatly-reduced lifespan. Retaining walls are often associated with roadways and parking lots, which receive a great deal of visual traffic. Failed retaining walls can generate a poor perception of the land development engineer and their company.

### Earth Pressures Against Walls

Earth pressure is the force per unit area exerted by the soil on the retaining structure. "Lateral," meaning "sideways," combined with "earth pressure" results in the term "Lateral-Earth Pressure," which is the force or pressure exerted in the principally horizontal direction against the retaining structure. Retaining-structure design considers the analysis and determination of these lateral-earth forces, their magnitude, and their applied direction (Liu, 1937).

The magnitude of the earth pressure depends on the physical properties of the soil, the soil-structure interaction, which occurs at the interface of the soil mass and the adjoining retaining structure, and the deformation behavior of the soil-wall system. The directions of the applied forces are influenced by the geometry of the retaining wall and the retained earth within a given distance to the wall. Since movement of the retaining wall is a primary factor in developing the earth-pressure forces acting against the wall, and since the characteristics of the soil influence these earth-pressure forces, the design of the wall system becomes indeterminate. (USS Steel Sheet Piling Design Manual, 1975).

Design engineers typically calculate two categories of earth pressures—active-earth pressure and passive-earth pressure—when designing retaining structures subject to minor movements inward or outward. A third category—earth pressure at rest—reflects a condition where absolutely no movement of the wall system occurs.

Active-earth pressures are developed when the soil mass behind the wall system moves outward as a result of some horizontal movement of the wall. Movement of the wall in the opposite direction (i.e., towards the soil) will in turn

mobilize the shear strength of the soil and is referred to as passive-earth pressure.

Two of the earliest earth-pressure solutions used to satisfy static equilibrium ( $\sum F_h = \sum F_v = 0$ ) of the retaining-wall system were the Coulomb Theory (1776) and later the Rankine Theory (circa 1857). Because of its simplicity, only the Rankine Theory is considered here.

Rankine Theory is based on several assumptions:

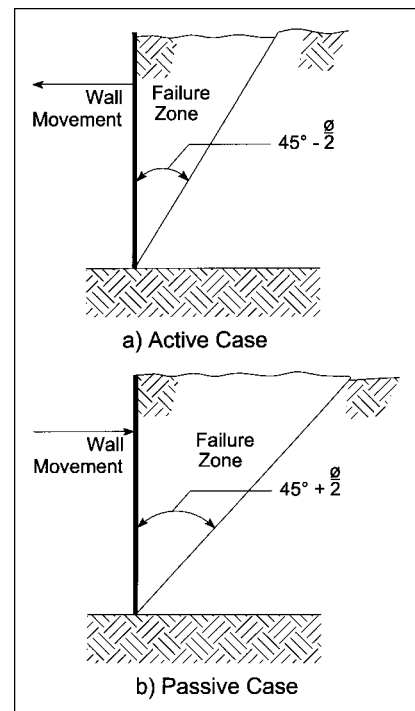
- No adhesion or friction exists between the wall and the backfill.
- The ground surface behind the wall is either flat or sloping.
- A sliding-wedge-type failure occurs along an assumed failure plane (i.e., shear plane), which is represented as a function of the soil's angle of internal friction ( $\phi$ ).
- The back of the wall is vertical.

Figure 39.18 represents the active and passive conditions associated with Rankine analysis of wall design.

The equations for computing lateral-earth pressure based on the Rankine Theory are as follows (LIU, 1937):

$$P_a = \frac{1}{2} \gamma H^2 K_a \quad (39.20)$$

$$K_a = \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}} \quad (39.21)$$



**FIGURE 39.18** Active and passive failure zones for Rankine Theory.

$$P_p = \frac{1}{2} \gamma H^2 K_p$$

$$K_a = \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}} \quad (39.22)$$

where

- $P_a$  = active-earth pressure
- $\gamma$  = unit weight of backfill soil
- $H$  = height of the wall
- $K_a$  = coefficient of active-earth pressure
- $\beta$  = angle between backfill surface line and a horizontal line
- $\phi$  = angle of internal friction of the backfill material
- $P_p$  = passive-earth pressure
- $K_p$  = coefficient of passive-earth pressure

When the backfill behind the wall is horizontal (i.e., level)

$\beta = 0^\circ$ , the coefficients of earth pressure become:

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} \quad (39.23)$$

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \quad (39.24)$$

For convenience, Table 39.19 provides Rankine earth-pressure coefficients for level backfill ( $\beta = 0^\circ$  for various  $\phi$  values).

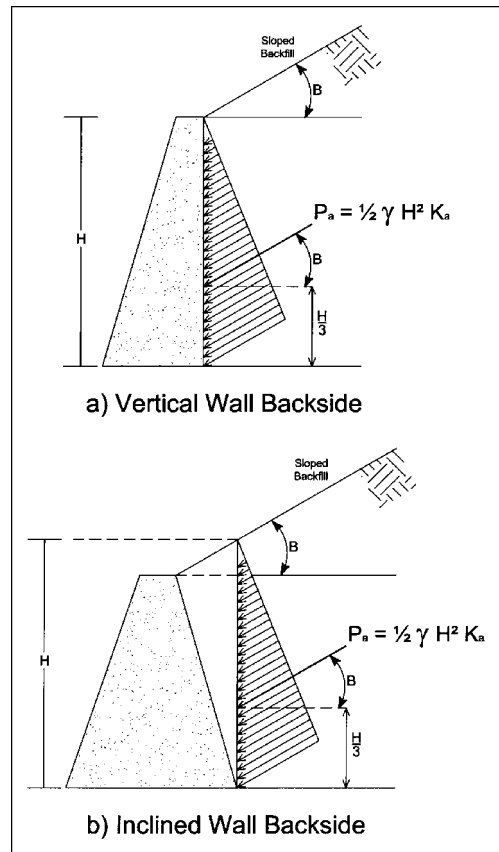
The resultant lateral-earth-pressure forces acting on a simplified wall section are depicted in Figure 39.19. Special attention is given to walls with inclined backsides as depicted in Figure 39.19(b).

### Design Considerations for Retaining Walls

Retaining-wall performance has a checkered past, and for many designers the visual intensity of a failed or failing wall system can mar their reputation in the industry.

Statistical studies of retaining-wall failures are provided in Figure 39.20 and can be summarized as follows (*Retain-ins & Flood Walls*, 1994):

- Improper design of the drainage system and/or the walls foundation base is the main cause of retaining-wall failure.



**FIGURE 39.19** Rankine Lateral Earth Pressure Theory for vertical and inclined wall geometry.

- Clay soils as backfill or as foundation-bearing material are involved in most retaining-wall failures.

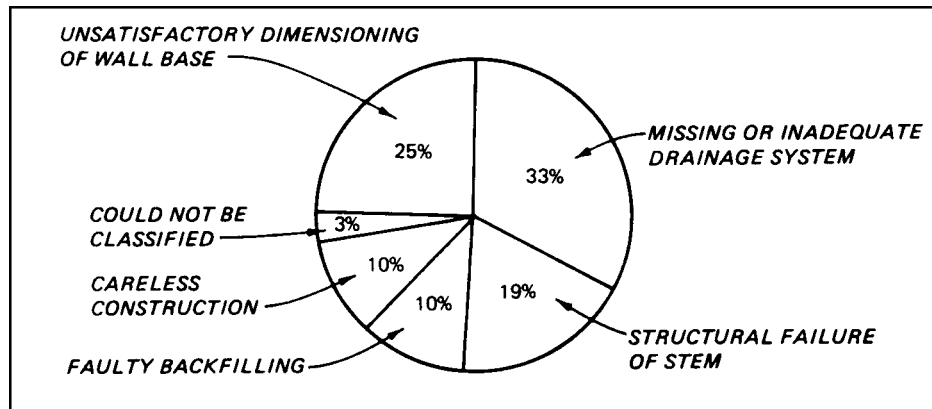
Structural failures of the wall system could be a result of structural integrity but are most probably caused by the inability of the designer and/or geotechnical engineer to recognize one or more of the unique soil or loading conditions that could dramatically impact the forces on the wall. Improper construction practices also lead to premature wall failures. The practical designer should consider:

Proper drainage through redundant methods, including proper surface drainage, weep holes, longitudinal drain pipes, and/or commercial drainage products

Proper free-draining backfill material and filter layers to protect the drainage system from becoming clogged

**TABLE 39.19 Rankine Earth Pressures for Level Backfill**

$\phi$	10°	12.5°	15°	17.5°	20°	22.5°	25°	27.5°	30°	32.5°	35°	37.5°	40°
$K_a$	0.70	0.64	0.59	0.49	0.45	0.45	0.41	0.37	0.33	0.30	0.27	0.24	0.22
$K_p$	1.42	1.55	1.70	1.86	2.04	2.24	2.46	2.72	3.00	3.32	3.69	4.11	4.60



**FIGURE 39.20** Causes of failure of rigid-concrete retaining walls. (Tcheng and Iseux, 1972. From *Retaining and Floodwalls*, ASCE 1994. Reprinted with permission from American Society of Engineers.)

Detailed plans and specifications and adequate field-compaction testing and inspection oversight during construction (inspection of retaining-wall drainage and backfill operations are often omitted)

Insufficient geotechnical exploration and laboratory testing prior to the design stage resulting in the use of “assumed soil parameters” with inadequate safety factors

Excessive backfill compaction and/or use of heavy equipment adjacent to the top of the wall can induce very high residual pressures against the wall

### Types of Retaining Walls

Retaining walls come in many different styles, shapes, materials and design complexity. However, most retaining walls can be classified into one of the following types: gravity, cantilever, anchored, or mechanically-stabilized earth (MSE).

The type of wall selected for a particular project is often a function of constructability, aesthetics, and of course economy (Winterkorn, 1975). Gravity retaining walls have been used for centuries and can be found in all walks of life. The majority of walls constructed in the last hundred years or so have been cantilever walls constructed of reinforced concrete.

### MSE Walls

Over the last 25 years, mechanically-stabilized earth walls (MSE) have grown in popularity, especially in highway construction. MSE walls utilize the natural forces of an engineered fill with small quantities of materials in a composite arrangement of equilibrium.

The basic components of MSE walls consist of soil, reinforcement and facing elements. A variety of facing elements and reinforcement are being used in MSE walls. Metal strips, steel rods, woven geotextile fabrics, and high-strength-plastic geogrids have all been used successfully as reinforcement materials (see Figure 39.21).

Likewise, facing materials have evolved over recent years and come in a variety of shapes, architectural finishes, and colors. Facing elements constructed of precast concrete panels, pre-fabricated metal sheets, welded wire mesh, and in some cases plastic geogrids have all been used successfully. The popularity of MSE walls is related to many factors, the most significant of which is cost. MSE walls have proven to be very cost-efficient compared to their traditional counterparts. This is especially true when the wall is in a fill condition with a surface area greater than 2000 sq ft and an average wall height of 10 ft. The advantage and disadvantages of MSE systems are outlined below (Mitchell and Villet, 1986 extracted from *Retaining and Floodwalls*, ASLE, 1994).

### Advantages

- Mechanically-stabilized backfill systems are economical when compared to conventional retaining walls.
- Construction of mechanically-stabilized backfill systems usually is easy and rapid. It does not require skilled labor or specialized equipment. Many of the components are pre-fabricated, allowing relatively quick construction.
- Regardless of the height or length of the wall, the structure remains stable during construction.
- When compared to conventional retaining walls, mechanically-stabilized backfill systems are relatively flexible and can tolerate large lateral deformations and large differential vertical settlements (when this is anticipated, vertical-sliding joints can be installed at intervals to compensate for movement). The flexibility of mechanically-stabilized backfill systems allows the use of the lower factor of safety for bearing-capacity design than conventional, more rigid structures.
- Mechanically-stabilized backfill systems potentially are better-suited for earthquake loading than conven-



**FIGURE 39.21** Photo of reinforced earth wall showing pre-fabricated concrete facing panels, galvanized metal reinforcing strips, and granular backfill.

tional retaining walls because of the flexibility and inherent energy-absorption capacity of the coherent earth mass. In designing mechanically-stabilized backfill systems for earthquake regions, provision should be made for slippage of reinforcement elements rather than tension failure of the elements, resulting in a ductile structure.

- Mechanically-stabilized backfill systems, because of their flexibility and mass, are capable of withstanding dynamic loads imposed by wheel loads.
- Geosynthetic reinforcements are stable under chemical and biological conditions normally occurring in soils. Since facing elements play only a secondary structural role, a greater flexibility is available to meet aesthetic requirements than for conventional retaining walls. Facing arrangements range from concrete panels of various shapes, textures, and colors to provision of vegetation at the exposed face of the soil.

#### Disadvantages of MSE Walls

- Corrosion of metallic reinforcement occurs and must be assessed on a project basis by determining the potential aggressiveness of the soil. Special coatings such as galvanized zinc and resin-bonded epoxy are used with a sacrificial thickness of steel added in the design to give the required service life.

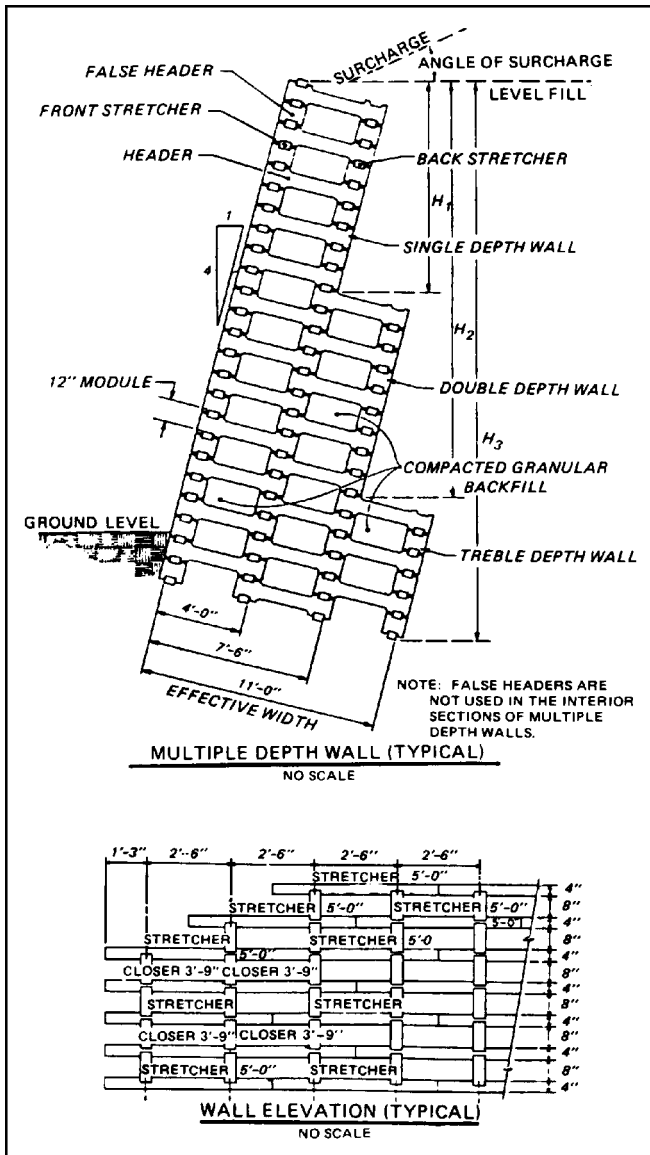
- Although geotextile and geosynthetic reinforcement is a robust material, some allowance must be made for decrease in strength because of abrasion during construction. This will vary with the type of reinforcement material.
- Different polymers and fabrics have different creep characteristics. Allowable loads in the grid should be selected based on allowable deformations, as well as the results of creep tests (10,000 hour).
- The construction of mechanically-stabilized embankments in cut regions requires a wider excavation than conventional retaining walls.
- Post-excavation behind the mechanically-stabilized wall is restricted.

#### Modular Wall Systems

Modular wall systems like MSE walls have grown in popularity in recent years. Modular components come in a variety of shapes and designs and are usually fitted with a type of interlocking mechanism that forms a composite wall system.

Most modular wall systems are designed as gravity walls and are especially popular where aesthetics are an important factor. One of the more well-known modular systems is the Criblock system (see Figures 39.22 and 39.23).





**FIGURE 39.22** Schematic diagram of cribblock retaining wall. From *Retaining and Floodwalls*, ASCE 1994. Reprinted with permission from American Society of Engineers.

Crib walls are constructed with some type of exoskeleton elements made of wood, precast concrete or steel, and filled with compacted fill.

The advantages and disadvantages of the precast concrete modular systems are provided below (Mitchell and Villet, 1986, extracted from *Retaining and Flood Walls*, ASLE, 1994).

### Advantages of Modular System

- Modular systems are economical when compared to conventional retaining walls in cut situations, particularly where the retaining wall has a total surface area greater than 500 ft<sup>2</sup> and average wall heights greater than 8 ft.



**FIGURE 39.23** Photo of a cribblock wall.

- Assembly of the wall components requires no fasteners, and the modules may be reused easily and economically.
- The precast-concrete, modular retaining wall does not utilize reinforcing elements and, therefore, is not subject to corrosion damage.
- Excavation behind the precast-concrete, modular retaining wall does not adversely influence the stability of the system, as might occur for the mechanically-stabilized wall.

### Disadvantage of Modular System

- The precast-concrete modular retaining wall could sustain cracking of interior connecting members because of relatively small (0.5 ft per 100 ft of wall length) longitudinal differential settlement.

Various engineering companies involved will provide site-specific plans and limited designs for their proprietary system. Stability is evaluated in a manner similar to a conventional gravity retaining wall. For stability calculations, the interlocking precast-concrete, modular system is assumed to behave as a single coherent block. The system must be stable against sliding along the base of the structure, overturning about the toe of the wall, bearing-capacity failure of the foundation soil, differential settlement, and overall slope stability.

### SLOPE STABILITY

A slope refers to an earth mass, natural or man-made, the surface of which forms an angle with the horizontal. Land development practices usually include engineering design and construction of man-made slopes such as are found in embankments, earth dams, levees, foundation excavations and trenches, and natural slopes found in hills, mountains, river banks, and coastal boundaries.

No slope, natural or man-made, is completely stable. Gravitational forces combined with climatic changes, especially changes involving water, are forever altering slopes. Occasionally slopes are altered or influenced by earthquakes, glaciers, and wind.

Slope stability is measured by its factor of safety, the ratio of the resisting forces (or moments) to the driving forces (or moments). The factor of safety is associated with several different soil parameters. For example, the factor of safety with respect to strength ( $F_s$ ) is the ratio of available (i.e., maximum) strength to the required strength to sustain a load. In terms of the Coulomb equation, this is written as

$$(FS)_s = \frac{(c + \sigma \tan \phi)_{avail}}{(c + \sigma \tan \phi)_{req}} \quad (39.25)$$

The factor of safety with respect to cohesion ( $F_c$ ) is the ratio between actual cohesion and the cohesion required for stability. Similarly, the factors of safety with respect to friction and critical height<sup>2</sup> are

$$(FS)_\phi = \frac{(\tan \phi)_{avail}}{(\tan \phi)_{req}} \quad (39.26)$$

$$(FS)_H = \frac{H_{avail}}{H_{req}}$$

respectively. For dry, granular soils with  $c = 0$ , the factor of safety is  $(\tan \phi) / (\tan \beta)$  where  $\beta$  is the angle of the slope with respect to the horizontal. The factor of safety must be greater than 1 for a slope to be considered stable. In practice, a factor of safety of about 1.5 is desirable.

From this discussion, the following general statements can be inferred regarding slope stability:

1. Since the shear strength of granular soils increases with increased normal stress, the stability of a granular slope does not decrease as the height increases.
2. The stability of a clay slope decreases as the height increases since the shearing strength of a clay is independent of the normal stress.

There are numerous methods available to perform slope-stability analysis. Most are described in soil mechanics and geotechnical engineering texts. The conditions, parameters and assumptions, such as soil-moisture conditions, kinematics of slope failure, shear strength, density and slope geometry vary considerably for each method. The level of reliability of these factors dictates the acceptable factor of safety and the method used for the analysis. Most cases require the analysis to account for seepage, various moisture conditions, and slopes that contain several types of soils. The geotechnical report typically provides the results from

the analysis and recommendations for slope angles. Despite the apparent similarity of the engineering properties of the soils on a site, the recommended slope angles may vary depending on the function of the slopes. Soils reports often recommend different slope angles for road embankments, cut slopes, slopes around storm-water-management ponds, etc.

Analysis and construction of an embankment that is strong, incompressible and stable requires specifying a soil with certain engineering properties. Ideally, the specified soil is within haul distance of the site, or even better, available as excess at the site. Of course, the embankment is only as stable as the underlying foundation soil.

The mode of failure of an embankment depends on the strength of the fill material, the underlying soil, as well as the drainage characteristics of both. Fills on deep soils with low strength fail because of inadequate bearing capacity. Reducing failure potential in such cases includes the use of lightweight fill material, using flat slopes for the embankment, or the use of a counterweight at the toe. If the weak underlying soil stratum is not very thick, it may be more economical to remove and replace it with an adequate foundation soil. In the worst case, where neither of the two preceding remedies is feasible, some structural foundation support will be needed.

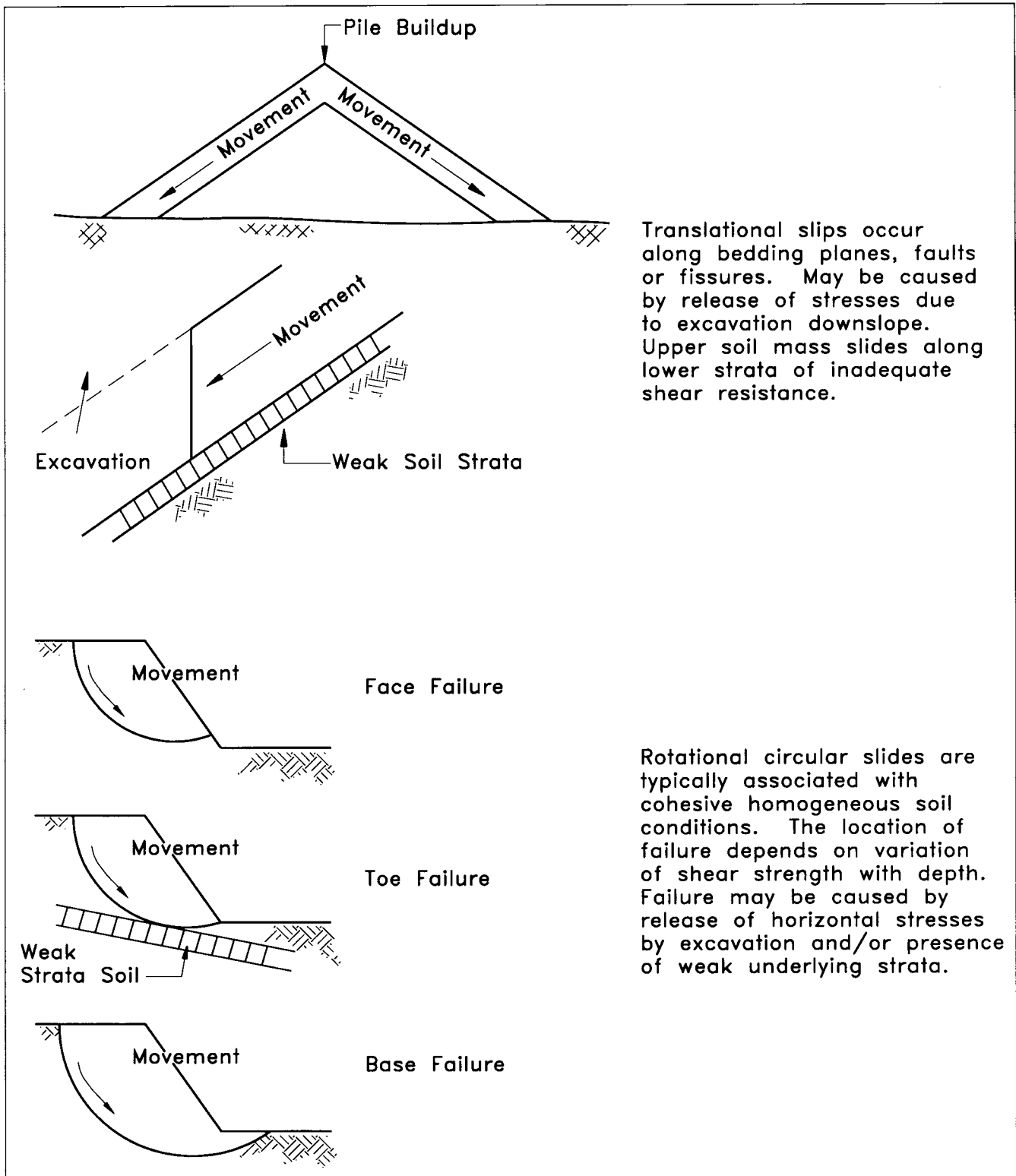
Organic silts, organic clays, and peat are highly compressible. Although these soils may have sufficient bearing capacity to support the embankment without failing, excessive settlements are the concern. Remedies for neutralizing excessive settlement include preconsolidating soft clays by careful overloading, slow construction, or removal of the inadequate material. Alternatively, many ground-improvement techniques are available today which can compensate for poor soils. Analyses for preconsolidation measures require calculations for primary and secondary consolidation rates. The cost for delaying construction until secondary consolidation is achieved must be evaluated against the cost for removal and replacement.

The potential for failure exists when embankments are placed on thin layers of soft clays. The failure occurs by horizontal sliding within or along the clay layer. The clay layer gains strength as it consolidates under construction and embankment loadings. If the imposed stresses increase faster than the consolidation, failure results. This type of failure occurs during or soon after construction for this reason. In this situation, construction and fill placement must proceed at a slower pace or the use of band/wick drains employed to allow the clay to consolidate. Also, lighter embankment material may be used or the embankment slopes could be flattened. Again, removal and replacement of the clay layer can also be a consideration.

Several types of slope failure are illustrated in Figure 39.24.

Slope failure occurs whenever the available resisting forces are less than the activating forces causing instability. In effect, when the strength of the soil is less than the

<sup>2</sup> Critical height is defined as the maximum height for which the slope remains stable.



**FIGURE 39.24** Types of slope failure. From Schroeder/Dickenson, *Soils in Construction*, 4th edition, copyright 1984. Reprinted by permission of Pearson Education, Inc., Upper Saddle River, NJ.

downslope-force component of the soil weight and other applied loads, the result is slope movement.

An imbalance of the two countering forces results in a number of ways; an increase in loading at the top of slope. This can be through the addition of a structure or by steepening the slope. Weakening of the slope also results from undercutting the toe of the slope.

Soil strength in cohesionless soils is reduced by an increase in ground-water (i.e., pore-water) pressure. Ground-water pressures are increased by raising the water table through excessive rainfall and snowmelt or seepage from underground springs and lakes.

Piping and drawdown often cause slope failure on dam embankments. Rapid drawdown can cause an increase in the effective weight of the embankment, analogous to adding weight at the top of the slope. "Piping" is a term used to describe the progressive removal of soil particles caused by percolating water.

Vibrations from earthquakes, blasting and pile driving can cause densification of loose sands, silts and some other highly-compressible soils located below the ground-water level, and this increases the pore-water pressure and correspondingly reduces the available soil strength. Cyclic stresses can also cause liquefaction of loose, uniform-saturated sand layers.

Natural processes such as weathering, leaching, creep, and mineralogical changes in soil and rock masses can reduce shear strength of the soil and result in slope movement.

Examples of slope (shear) failure are shown in Figure 39.25.

## PROBLEM SOILS

Certain types of soils require special attention because of their irregular performance characteristics. The areal extent of such soils varies and specific problem soils are indigenous to selected areas of the country. The site may contain several isolated pockets of such soils or may be a major deposit area. Identification and location of problem soils early in the design process alleviates costly delays for redesign or unanticipated costs for additional excavation and reinforced foundation design. Additionally, provisions in local ordinances may require additional explorations, testing, and special designs. In extreme cases, the provisions may even preclude certain design elements of the project. Some of the common types of problem soils are described herein.

### Expansive Clays

Certain clays undergo severe volume changes without any removal or application of external loads. Such shrink and swell phenomena are the result of changing moisture conditions. Soils that are most susceptible to such moisture-induced volume changes contain significant amounts of smectite, a mineral predominant in montmorillonitic clays. Typically, these soils have a high plasticity index and are predominant in the southern United States. Local names

for such soils are gumbo, adobe, and black cotton. Swelling of these soils can create high stresses on foundations, walls, and slabs, even to the extent of breaking basement walls and lifting light structures. Environmental factors greatly affect the behavior of expansive soils. Such factors as surrounding and underlying soils, hydrology, slope angle and length, and vegetation can influence the behavior of these expansive soils.

Damage to structures on expansive soils may not become evident for years after completed construction. Shrinking of the soil during dry periods causes gaps to form under footings and foundations, thereby reducing the bearing support of the soil. This results in uneven structure settlement causing cracks in the structure. When the soil becomes moist again, the expanding soil causes the foundation to rebound to near original position. After several cycles of shrink and swell, the structure does not return to its full original position. The building deteriorates further and damage becomes apparent.

Buildings do not have to be directly situated on expansive soils to develop problems. Experience in Fairfax County, Virginia shows that some houses have settlement problems with footings located on soils 3 ft above the expansive soil strata (see Figure 39.26).

Using expansive soils as backfill around basement walls is not a recommended practice. Buildup of excessive pressures against the wall is caused after the soil settles and moisture is added (see Figure 39.27). During dry periods, additional soil particles fall into the cracks of the soil. When the soil expands again, the added soil coupled with the volume increase pushes on the wall. The cyclic stress loadings may eventually cause the wall to fail. Not only is this damage a headache for the homeowner, the result can cause damage to nearby sewer, gas, and water pipes.

As illustrated in Figure 39.28, large trees in expansive soils near buildings can exacerbate the damaging effects of shrink and swell phenomena. During a dry period, a tree's demand for water can cause localized shrinking of the soil. The decrease in volume in a small area coupled with increased volume elsewhere near the house creates a wavy ground surface around the structure. Large elm, poplar, and willow trees can contribute to such failure; however, small-diameter trees (e.g., less than 12 in.) and shallow roots result in significantly fewer failures. A recommendation is that trees be planted more than one-half their expected mature height away from the foundation.

When construction cannot be avoided on expansive soils there are remedies that can mitigate damage potential, some of which can be costly. Such remedies include (Brown 1988):

- Excavating the expansive soils and replacement with granular soil or other suitable material
- Construction of the foundation below the shrink-swell zone or near the water table (which then requires

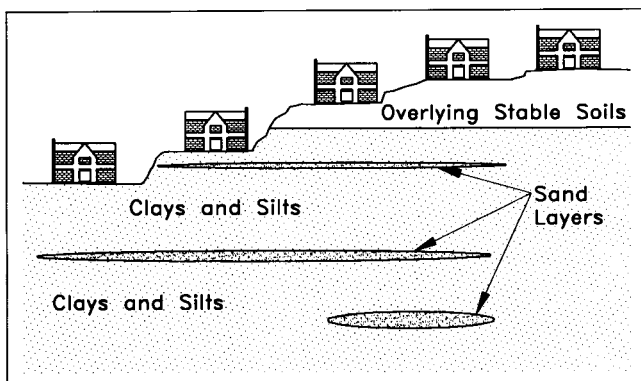


a)



b)

**FIGURE 39.25** Examples of slope (shear) (a) A slope failure in front of historic Mount Vernon caused by increased groundwater seepage with resulted in a reduction of the soils shear strength. (Photo courtesy of Paul G. Swanson.) (b) A typical highway cut-slope failure caused by over steepening.



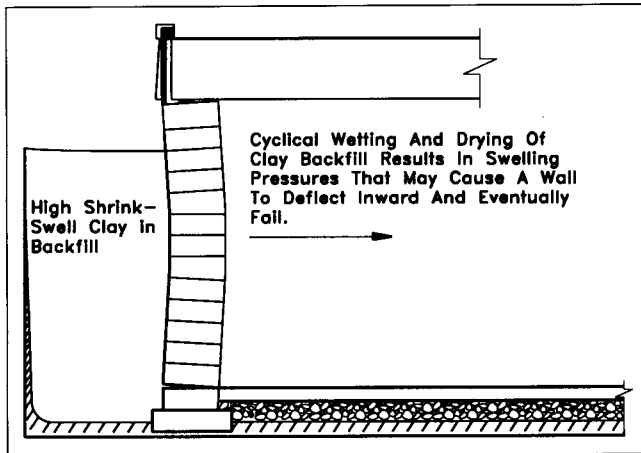
**FIGURE 39.26** Expansive soils underlying stable soil. Reprinted with permission from Johnson, L. K. *Homeowners Guide to Overcoming Problems with Marine Clay*. Fairfax County, VA.

other special design considerations for foundation design)

- Designing the foundation so that the dead loads totally counteract the heave
- Using blankets of impervious soil adjacent to foundations and grading the areas away from the foundations to prevent surface-water infiltration
- Locating water and drainage lines to direct water around and away from foundations.

### Frost Susceptible Soils

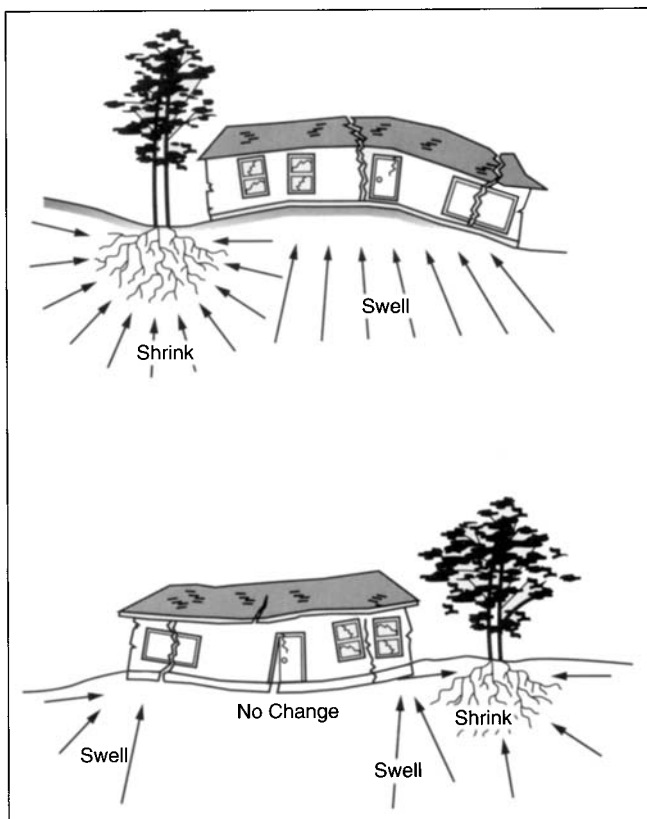
Frost heave is a natural phenomenon and will occur wherever water in soil is exposed to sustained freezing temper-



**FIGURE 39.27** Foundation wall damage from expansive soils. Reprinted with permission from Johnson, L. K. *Homeowners Guide to Overcoming Problems with Marine Clay*. Fairfax County, VA.

atures. When the water in the voids freezes, it expands by about 10%, but this in itself does not create frost-heave problems. Soil heaving because of freezing occurs when saturated fine-grained soils within the capillary zone above the water table form ice lenses or layers parallel to the ground surface. Ground heaving increases as the ice lenses accumulate and grow to thickness of several inches.

Frost heave is particularly damaging to pavement and slabs because of the erratic distribution of the ice lenses,



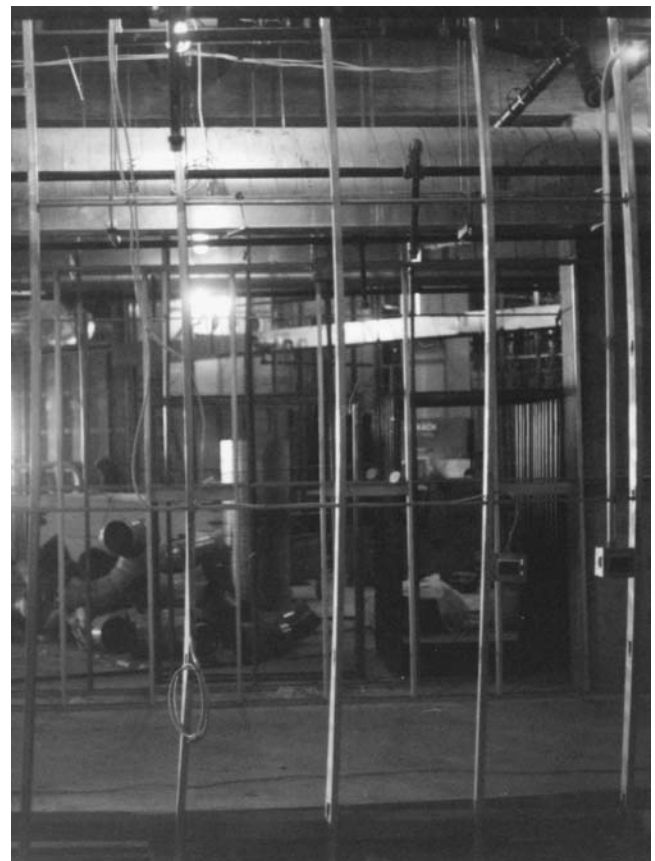
**FIGURE 39.28** Tree influence on expansive soils.

which results in differential heaving and cracking. Figure 39.29 shows the damage caused by frost heave of a ground-level floor slab. Notice the deformed, metal wall studs buckling from the upward displacement of the ground-floor slab. The formation of ice lenses is illustrated in Figure 39.30.

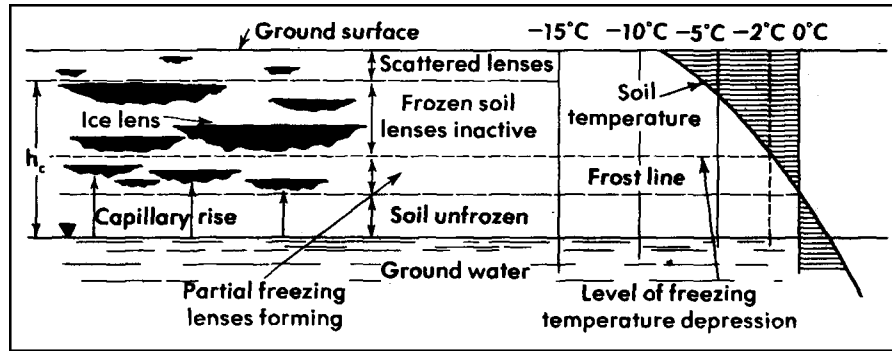
Non-frost-susceptible soils are those that cannot readily support the capillary rise of water and, therefore, prevent the formation of ice-lens growth. These soils typically are “free-draining soils” like clean coarse sands, gravel, and crushed stone.

Typically, silty soils and very fine sands are prone to objectionable frost heave because they facilitate capillary rise. Frost-susceptible soils typically are non-uniform soils containing more than 3% grains finer than 0.02 mm and uniform soils where more than 10% is finer than 0.02 mm.

Prevention of frost heave in footings is achieved most easily by following the recommendations of the local building codes and founding footings at depths below the local frost line. Controlling heave in slabs, walls, pavements, utilities and other construction components can be achieved by removing frost-susceptible soils throughout the depth of frost penetration and replacing them with non-frost-susceptible soils. Alternatively, for pavements and slabs, a horizontal blanket of coarse-grained, free-draining sand, gravel or crushed stone placed above the water level will



**FIGURE 39.29** Photo of frost-heave under floor slab causing metal studs to buckle.



**FIGURE 39.30** Formation of ice lenses. (*Introductory Soil Mechanics and Foundations* by George F. Sowers, copyright 1979. Reprinted by permission of Pearson Education, Inc., Upper Saddle River, NJ.)

break the capillary-tension forces. Care should be taken when designing these blankets; improper drainage or inadequate thickness could aggravate rather than prevent frost heave (Sowers, 1979). Figure 39.31 shows common frost-penetration depths in the United States.

### Metastable Soils

Collapsing soils are those that decrease in volume when they become saturated or when subjected to vibration after saturation. These are called metastable soils and are associated with fine, sand-silt-clay materials deposited along the base of mountains and along alluvial fans in arid and semi-arid regions. Similar to expansive soils, collapsing soils are generally stable until the moisture content changes. Additional moisture disrupts the clay and water bonding that maintains the soil structure in its metastable condition, collapsing the soil into a more compact and stable condition and often resulting in rapid and severe settlement.

Loess, a soil found in sections of the midwest and western United States, is the most common type of metastable soil. Compacted loess is a satisfactory foundation material for spread footings or mats as long as the dry-unit weight is around  $99\text{#/ft}^3$  or more. For shallow deposits (3 to 6 ft), compaction is necessary for the full depth of the deposit or above the permanent water table to insure adequate performance. For loess with dry-unit weights less than  $99\text{#/ft}^3$ , pile foundations driven to underlying acceptable soil strata should be considered.

Collapsing soils typically are found in arid and semi-arid environments, alluvial valleys, and playas. Gypsum and anhydrite are often present in or around such soils. In general, these soils are very moisture-sensitive and exhibit greatly reduced strength when wetted. Differential settlement occurs if the soils are spread over a wide area or if a structure is partially founded on pockets of metastable soils.

### Organic Soils

Those soils that contain significant amounts of decayed and decaying vegetation matter are organic soils. These soils have very little cohesive or friction strength and are highly

compressible, even under light loads. Organic soils are characteristic of estuarine, lacustrine, and floodplain areas.

Peat, a common organic soil, is fibrous, partially-decomposed organic matter or a soil containing 80% or more fibrous organic matter. Peat soils are dark brown or black, loose and extremely compressible. Muskeg is a type of peat indigenous to northwest Canada and Alaska.

### Dispersive Clays

Certain types of soils will go into suspension and become highly erodible in the presence of water (flowing or static) despite being compacted. Such soils are known as dispersive clays and often contain montmorillonite or illite minerals. These dispersive clays are not uncommon and typically are found in floodplain deposits, slope wash, lake beds, and loess.

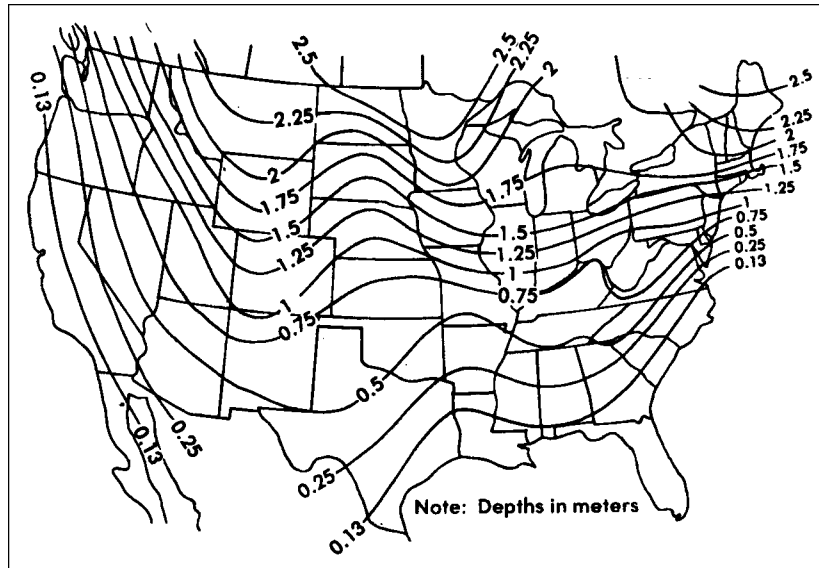
### Normally-Consolidated Clays

A soil is normally consolidated if the maximum pressure on an element of soil mass is equal to the present soil-column pressure. That is, the compression state of the soil is attributable to the existing overburden weight. Otherwise, if the soil mass compression behavior is caused by pressures larger than the present overburden pressure, the soil is overconsolidated.

Normally-consolidated clays are subject to additional long-term settlements under loads greater than the existing overburden pressure. Such long-term settlements can be reasonably estimated from tests performed on undisturbed samples.

### Limestone

Limestone and dolomite contain substantial amounts of carbonate minerals ( $\text{CO}_3$ ). Gypsum and anhydrite are salts composed of calcium sulfate ( $\text{CaSO}_4$ ). All of these sedimentary minerals are highly soluble. Consequently, the presence and development of subsurface cavities is a consideration. Other similar types of materials found near shorelines are marl and chalk.



**FIGURE 39.31** Max frost penetration depth. (*Introductory Soil Mechanics and Foundations* by George F. Sowers, copyright 1979. Reprinted by permission of Pearson Education, Inc., Upper Saddle River, NJ.)

The surface collapse of soil overlying a subsurface cavity is usually referred to as a sinkhole, particularly in limestone and dolomite terrain. Open-solution voids in the bedrock allow the erosion of overlying soil into the void by groundwater movement. This action, in turn, forms a soil cavity at the bedrock surface, and this will continue to enlarge until it can no longer support the overlying soil and a sinkhole occurs at the ground surface. These conditions emphasize the importance of positive-surface and ground-water control. An example of surface collapse and preparation of the bedrock surface for remedial measures is shown in Figure 39.32.

### GROUND-IMPROVEMENT METHODS

Numerous ground-improvement techniques have been developed and/or perfected in the last half-century. Geotechnical engineers have found new ways to compensate for Mother Nature's less than desirable subsurface conditions and continued redevelopment of urban areas. This rapid growth and use of ground-improvement techniques has been fueled by the increasing need to develop marginal land and develop cost-effective solutions to land development.

Some of the ground-improvement techniques available today include:

- Vibro compaction/replacement
- Dynamic compaction
- Wick/band drains
- Lime and cement stabilization
- Geosynthetics

Ground-improvement techniques described herein have been employed successfully and globally to facilitate con-

struction on otherwise undesirable properties. These techniques are briefly discussed so that land development engineers can offer potential solutions to clients who would like to develop problematic building sites.

### Vibro Compaction/Replacement

Vibro compaction/replacement has been used for over 50 years to reduce settlement and improve bearing capacity on non-cohesive soils. The addition of stone through special vibro-replacement equipment allows improvement of weak ground and poorly controlled man-made fills (see Figure 39.33). Vibro-techniques have been used successfully on housing projects, warehouses, and light industrial structures (Moseley and Priebe, from *ground improvement*, Moseley 1993).

### Dynamic Compaction

Dynamic compaction has increased considerably over the last 25 years and is a widely-accepted technique for improving weak soils, rubble fills, solid waste, collapsible soils, and liquefiable soils (see Figure 39.34). The simplicity and economic benefits of dynamic compaction have made it an attractive alternative to pre-loading, vibro techniques, or deep-foundation systems.

Dynamic compaction involves the systematic dropping of large weights onto the ground surface to compress and consolidate weak or soft soil and rubble material. Heavy weights of 5 to 20 tons falling from heights of 30 to 85 ft typically are used to improve the bearing capacity and reduce differential settlement of the underlying ground to depths of up to 30 ft or more. (Mayne et al., 1984).

Dynamic compaction is most widely used to improve primarily sandy materials and granular fills; however it has been used to improve sites containing fine-grained soils.





a)



b)

**FIGURE 39.32** (a) Surface collapse (b) Preparing bedrock for remedial measures.

The degree of improvement varies with soil type and ground-water depth.

Poor subsurface soil conditions at sites proposed for office buildings, warehouses, townhouses, highways, airports, and industrial sites have been improved successfully using dynamic compaction.

### Wick/Band Drains

Wick/band drains are used to accelerate the consolidation of low-permeability, fine-grained soils. Discussion of time-rate consolidation earlier in this chapter indicated that settlements of clayey soils were dependent upon the dissipation of pore water out of the void spaces. The faster the water could be squeezed out of the soil layer, the faster the settlement would occur. The installation of wick or band drains vertically into the compressible layer on a predetermined spacing reduces the flowpath distance of the water and speeds up consolidation.

Wick/band-drain installation is usually associated with pre-loading or large fill projects where settlement times must be shortened to accommodate future construction or



**FIGURE 39.33** Vibro-replacement ground-improvement method using stone columns.

civil projects such as highway approaches to bridge structures and airport runways (see Figure 39.35).

### Lime and Cement Stabilization

Lime and cement stabilization are used where poor near-surface soil conditions can be altered economically through the introduction of lime or cement. Soil stabilization with admixtures has been used extensively in the highway construction field for many years.

**Soil-cement.** The term commonly adopted from cement stabilization research is a mixture Type I Portland cement and soil. The use of soil-cement has proven to be very successful for improving sandy and silty subgrade soils for support of airfields, roadways, and large parking lots. The addition of between 3 and 8% cement by weight is typical depending on the strength characteristics desired.

**Lime Stabilization.** This is also referred to as lime modification when smaller amounts of lime are utilized in an effective method to alter the undesirable characteristics of highly-plastic, clayey soils when used for subgrade support. Generally, stabilization techniques with hydrated lime are likened to those with cement. The difference, however, is that lime is more effective with fine-grained soils having plasticity. Admixtures containing 4 to 6% of hydrated lime



**FIGURE 39.34** Dynamic compaction. (Courtesy of Terasystems, Inc.)

by weight are common for subgrade improvement (see Figure 39.36).

### Geosynthetics

Around since the 1960s, geosynthetics have exploded onto the engineering/construction market over the last 15 years. This is a nearly billion-dollar industry that includes product categories such as geogrids, geotextiles, geocomposites, geonets, and geomembranes. (See Figure 39.37.)

**Reinforcement.** By adding tensile strength to the soil, geogrids and geotextiles can support horizontal loads like those found in MSE walls and vertical loads like those experienced beneath wheeled-traffic loads (Figure 39.38(a) and (b)).

**Separation.** Keeping two unlike or different materials apart is accomplished through the use of geotextiles (Figure 39.39).

**Cushioning.** This provides separation and absorbs impact or abrasive action that could damage critical membranes. It is common in ponds that are lined but require ballast to hold down the liner (Figure 39.40).

**Filtration.** This allows water to pass unrestricted while retaining soil or other materials. It is common for trench drains and erosion-control measures (Figure 39.41).

**Transmission.** Drainage allows water or other fluids to move along or within the geosynthetics. Geocomposites/geonets are used for a variety of applications. Common uses include drainage behind walls and wick drains (Figure 39.42).

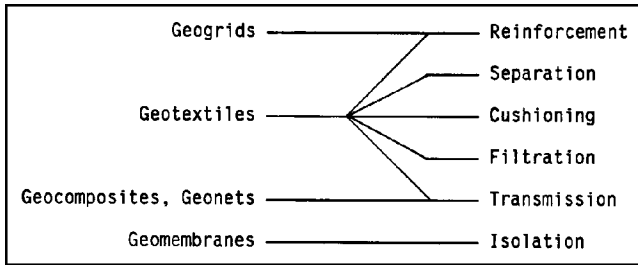
**Isolation.** This can be the retention of one fluid or the separation of two fluids. Geomembranes in landfills and pond liners are examples of isolation products (Figure 39.43).



**FIGURE 39.35** Wick-drain installation.



**FIGURE 39.36** Lime stabilization of clayey subgrade soils in a proposed parking lot.

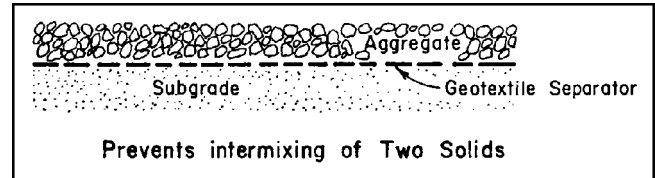


**FIGURE 39.37** Functions of major types of geosynthetics. (Fluet 1988.)

Applications for geosynthetics are rapidly expanding and will likely continue to gain popularity as cost-effective alternatives to traditional construction methods.

Common uses include:

- Roadway subgrade stabilization
- Asphalt overlay support
- MSE wall reinforcement
- Filters and drains for roadways, walls, landfills, and ponds
- Erosion control and silt fences

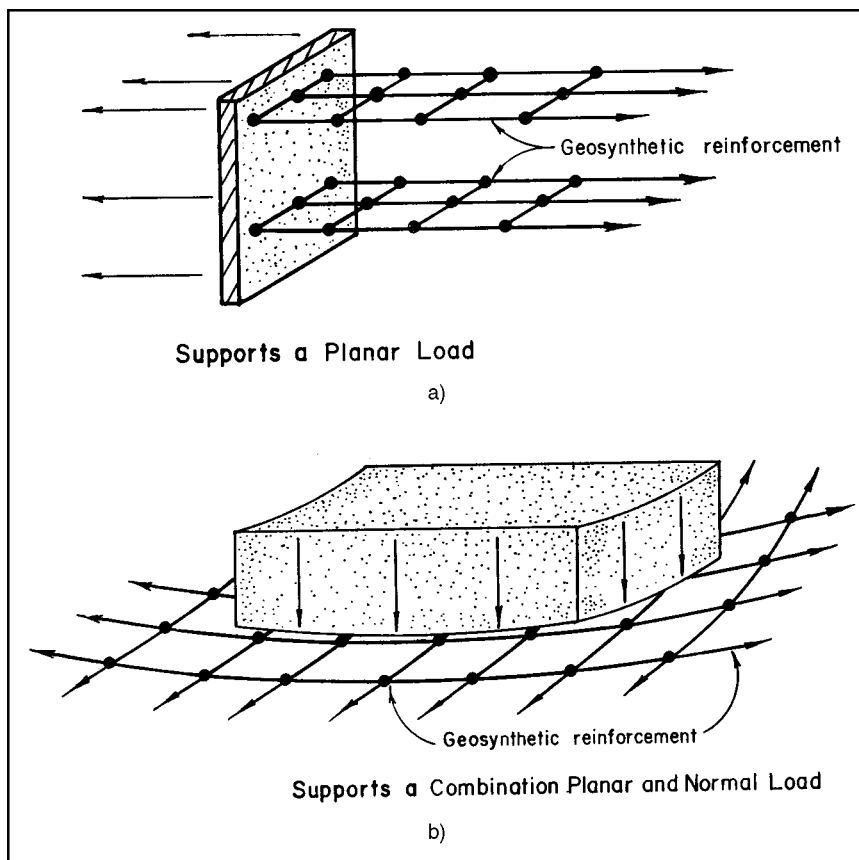


**FIGURE 39.39** Separation function (Fluet, 1988.) From *Geosynthetics for Soil Improvement*, by Holtz, R. D., ASCE 1988. Reprinted with permission from American Society of Engineers.

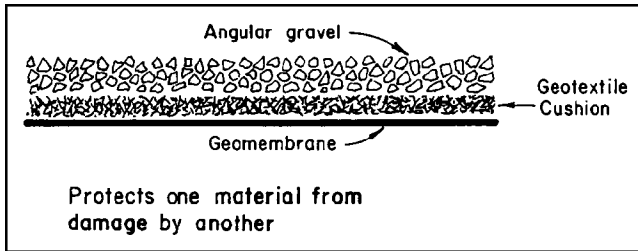
- Consolidation drains
- Impermeable liners

### SUBSURFACE INVESTIGATION

Decisions for the planning and design of a project depend highly on subsurface conditions at the site. Not only is geologic and soil information necessary for design, but information on what is contained below the surface affects various project decisions as well. The subsurface investigation aids the foundation designer and helps the owner in making economic and financial decisions for the project. Knowledge of subsurface conditions, such as contaminated ground water, presence of hazardous materials (natural or manufactured) and extremely poor soil and rock condi-



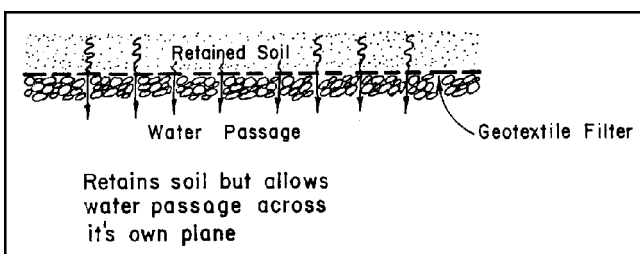
**FIGURE 39.38** Reinforcement function: (a) tensile member; (b) tensioned membrane. (Fluet, 1988.) From *Geosynthetics for Soil Improvement*, by Holtz, R. D., ASCE 1988. Reprinted with permission from American Society of Engineers.



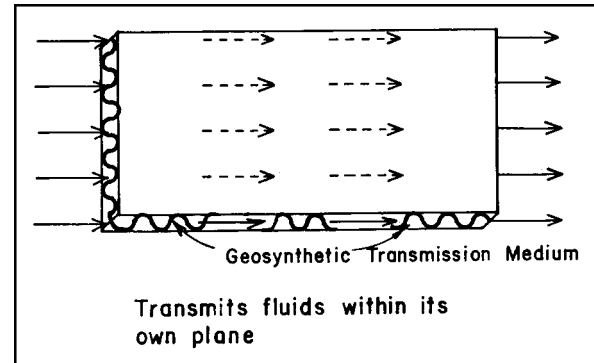
**FIGURE 39.40** Cushioning function (Fluet 1988.) From *Geosynthetics for Soil Improvement*, by Holtz, R. D., ASCE 1988. Reprinted with permission from American Society of Engineers.

tions, at the early planning stages can save the owner money. For example, conditions may be so bad that the owner may abandon the project totally and thereby cut his losses before he is committed to purchasing the site or incurring further design costs. Although performing the subsurface exploration may be somewhat costly up front (subsurface exploration costs can be around 0.5% to 1.0% of the total construction costs), the long-run savings in most cases warrant such an expenditure. Additionally, many public agencies require subsurface information before the project is approved. The extent of this information depends on the nature of the subsurface conditions and the project. For projects where foundation and subsurface conditions are relatively straightforward and not complex, the information may be covered adequately by a short soils report. For complex projects with unusual or difficult conditions, the required information may be quite extensive. Local ordinances may also dictate the type of information needed.

A subsurface investigation is performed by geotechnical engineers. It is recommended that the owner contract separately with the geotechnical engineer to perform the work. Depending on the contract arrangements between the owner and the land design consultant, provisions may be made so that the land design consultant can contract directly with the geotechnical engineer. On the other hand, the land design consultant may have geotechnical staff and resources capable of doing the subsurface analysis. Additionally, quite often the geotechnical engineer sub-contracts with another company to do the actual drilling, digging test holes, and collecting samples, under the engineer's direction.



**FIGURE 39.41** Filtration function (Fluet 1988.) From *Geosynthetics for Soil Improvement*, by Holtz, R. D., ASCE 1988. Reprinted with permission from American Society of Engineers.

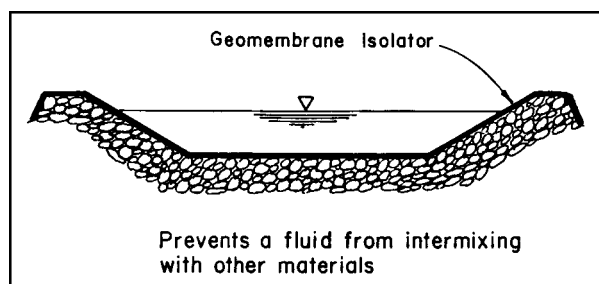


**FIGURE 39.42** Transmission function (Fluet 1988.) From *Geosynthetics for Soil Improvement*, by Holtz, R. D., ASCE 1988. Reprinted with permission from American Society of Engineers.

All investigations should begin by researching available information, followed by a thorough surface reconnaissance. Available information that may be helpful includes published geologic reports, SCS county soil reports, stereo air photos used for photogrammetric mapping, information and experience at nearby sites, etc. Based on an assessment of this information, a subsurface investigation program can be planned to obtain adequate data for analysis and design of the project. Specific objectives of the investigation normally include:

- Location and depth of soil layers, bedrock and ground water, as well as other features that may affect the project, such as subsurface cavities, hazardous waste and contaminated soil
- Samples of all significant materials for visual examination and classification or laboratory testing if necessary
- Field data to determine the density or compactness of all materials
- Any other special requirements based on project plans or subsurface conditions

Because of the inherent uncertainties of soil and rock behavior, the detail of the subsurface investigation is carried



**FIGURE 39.43** Isolation function (Fluet 1988.) From *Geosynthetics for Soil Improvement*, by Holtz, R. D., ASCE 1988. Reprinted with permission from American Society of Engineers.

out to a level that balances design requirements and cost with risk. For certain types of development projects, such as commercial and retail projects, the subsurface investigation is begun early in the planning phase. For initial residential development (i.e., lot and street layout), Soil Conservation Service Soil surveys may provide sufficient preliminary information to begin the project.

### Investigation Program

Soil survey reports are available from the Soil Conservation Service (SCS). These reports delineate and describe the soils of a locality. The very early SCS soil reports were compiled mainly for agricultural purposes. Recently, updated reports now contain additional information usable in land planning and engineering design. The reports describe the pedological soil series found in the locality, provide a map showing their location, and have tables that equate the soil series to engineering index properties. Other tables provide data relating the soil series to wildlife habitat, building site development, sanitary facilities, construction materials, water management, physical and chemical properties, and hydrologic characteristics.

The SCS soil survey report is an invaluable document for any development project. The information can be used throughout various stages of a development's design process. As part of the preliminary investigation, the soils should be mapped onto a scaled work drawing of the site. Soils that are anticipated to present problems for the project (e.g., high water table, poor bearing support, highly erodible) are identified on the map. These questionable areas need to be flagged in the field. The soils and subsurface conditions in the questionable areas can then be confirmed with supplemental data such as existing geotechnical reports for nearby projects or geotechnical investigations at the site. Although the SCS soils report is not the sole source of subsurface conditions for the project, it may allow the design of the project to proceed expediently.

A preliminary, subsurface exploration program is sometimes advantageous in gathering sufficient data to perform preliminary calculations for size and cost for foundations of major structures and to determine if there are any subsurface conditions that will impact the project significantly. The amount of data gathered in the field during a preliminary program depends on the extent of existing data and the need for additional information for preliminary design purposes. Unanticipated field conditions may warrant extra test pits, bore holes, and additional soil samples. As details of the proposed project develop during design, additional subsurface exploration may be required to supplement the previously-obtained data.

Field exploration programs should be carefully planned and laid out to obtain adequate data for design of the project. No set rules apply to laying out methodically exploration points that cover all conceivable conditions. Sound judgment supersedes any generalized rule-of-thumb statements. The number and location of the exploration points depends on the prevailing soil conditions, the variability of

the soils, and proposed project details. Highly-variable soil conditions warrant an increase in the number of exploration points to determine the areal extent of the soil. The increased number of exploration points, however, does not necessarily require an increase in the number of tests for determination of the soil's physical properties. The following guidelines may be helpful in establishing acceptable boring locations:

1. For office buildings or building complexes, a boring at each corner and one in the middle would be the minimum. For larger building groups, the objective might be to locate and space the borings to establish subsurface cross sections in perpendicular directions.
2. For dams, the objective is to establish geologic profiles across the valley at the longitudinal axis, at the downstream and upstream toes, and at all major hydraulic structures such as spillways, outlet works and large culverts/conduits. Borings within the impoundment area itself may help identify the suitability of excavated soils for the dam embankment and the seepage/infiltration characteristics of the area.
3. Borings in road areas are located to obtain data for four different criteria:
  - (a) Shallow borings along the alignment are spaced to identify and verify areas of similar soils as anticipated from other data sources such as soil maps and aerial photographs.
  - (b) Borings are taken in the vicinity of major structures such as bridge abutments, piers, and retaining walls. The number of borings for abutments or piers typically is one or two. Borings for retaining walls are placed to give longitudinal and transverse subsurface profiles.
  - (c) Borings are taken in cut areas to determine the difficulty of excavation and slope stability. These borings are more closely spaced and deeper than the roadway alignment borings.
  - (d) Borings for borrow areas provide data on the suitability of the soil to be used as fill material. Depending on the size of the projected borrow area, a grid pattern with borings at 200- to 400-ft intervals may suffice.

Again it is emphasized that the actual location of the borings should depend on project details, anticipated site conditions, and what is actually encountered during the drilling. The preceding generalizations are only to provide initial guidance. Other guidelines are presented in Table 39.20.

### Depth of Exploration

The depth of test borings depends on two factors: first, the magnitude and distribution of the imposed loading; and

**TABLE 39.20 Guidelines for Boring Layout**

AREAS FOR INVESTIGATION	BORING LAYOUT
New site of wide extent.	Space preliminary borings 200–500 ft apart so that area between any four borings includes approximately 10% of total area. In detailed exploration, add borings to establish geological sections at the most useful orientations.
Development of site on soft compressible strata.	Space borings 100–200 ft at possible building locations. Add intermediate borings when building sites are determined.
Large structure with separate closely spaced footings.	Space borings approximately 50 ft in both directions, including borings at possible exterior foundation walls at machinery or elevator pits, and to establish geologic sections at the most useful orientations.
Low-load warehouse building and large area.	Minimum of four borings at corners plus intermediate borings at interior foundations sufficient to define subsoil profile.
Isolated rigid foundation, 2500–10,000 sq ft in area.	Minimum of three borings around perimeter. Add interior borings depending on initial results.
Isolated rigid foundation, less than 2500 sq ft in area.	Minimum of two borings at opposite corners. Add more for erratic conditions.
Major waterfront structures, such as dry docks.	If definite site is established, space borings generally not farther than 50 ft adding intermediate borings at critical locations, such as deep pump-well, gate seat, tunnel, or culverts.
Long bulkhead or wharf wall.	Preliminary borings on line of wall at 200-ft spacing. Add intermediate borings to decrease spacing to 50 ft. Place certain intermediate borings inboard and outboard of wall line to determine materials in scour zone at toe and in active wedge behind wall.
Slope stability, deep cuts, high embankments.	Provide three–five borings on line in the critical direction to provide geological section for analysis. Number of geological sections depends on extent of stability problem. For an active slide, place at least one boring upslope of sliding area.
Dams and water retention structures.	Space preliminary borings approximately 200 ft over foundation area. Decrease spacing on center line to 100 ft by intermediate borings. Include borings at location of cutoff, critical spots in abutment, spillway and outlet works.

second, the subsurface characteristics. Since the basic objectives of any boring program are to determine a subsurface profile and to identify engineering properties of the materials, borings should extend to a depth that provides a reasonable comfort level for design analysis purposes. As in selecting the location of borings, this is a judgment decision to be made by the geotechnical expert and the person designing the foundation and structure. A suggested rule-of-thumb is “to carry borings to such depth that the net increase in soil stress under the weight of the structure is

less than 10% of the average load of the structure, or less than 5% of the effective stress in the soil at that depth, whichever gives the lesser depth, unless bedrock or dense soils known to lie on rock are encountered first.”<sup>3</sup>

The following is presented as a guide for determining boring depths:

<sup>3</sup> American Society of Civil Engineers (1976); *Subsurface Investigation for Design and Construction of Foundations of Buildings*.

1. Extend borings through any unsuitable and questionable material into firm, stable soils that are capable of sustaining the imposed loads without excessive settlement.
2. Borings should extend a sufficient depth into any apparent rock to insure the existence of bedrock rather than a boulder.
3. Explorations along roads are carried to depths of  $5 \pm$  ft below subgrade in areas of light cut and fill where no adverse subsurface conditions exist. In areas of deep cuts and large embankments, the depth of exploration depends on the topography and nature of the underlying soils.
4. In residential areas where houses have relatively shallow foundations, the depth depends on the nature of the subsurface soils. Typically, the depth of exploration will be 5 to 10 ft below the foundation. In most cases, borings will not be needed at every dwelling location.
5. For other buildings with substantial loadings, the depth of exploration beneath the footing extends 1.5 to 2 times the least dimension of the structure. In the case where the loading is carried on piles or caissons, the exploration must extend a sufficient depth into a competent bearing stratum to carry the imposed loading and to be sure that underlying weaker material is not present.
6. In borrow areas, the depth of exploration extends to the depth required to provide the amount of suitable material needed.

Other guidelines are presented in Table 39.21.

### Methods of Subsurface Exploration

For relatively shallow explorations, test pits and test trenches normally are used. These open excavations, dug by backhoes or other power equipment, are limited to 15 ft or less depending on the type of equipment used for digging. One advantage of test pits and trenches is that they allow inspection of the soil strata and ground water as it exists in place. Since test pits and trenches are used to gather information at relatively shallow depths, they would be applicable to projects with shallow foundations with light loadings, shallow excavations, and borrow areas. Test pits and trenches are not effective for areas that have a high ground water table.

Other techniques for performing exploratory analysis include various methods of drilling and sampling. Auger boring, wash boring, rotary drilling, and rock-core drilling all use drill bits driven by power machines. In each technique, when the required depth is reached, a soil sample is ob-

tained and the drilling continues to the next desired depth. These soil samples are used for visual classification and laboratory tests when in-situ soil properties are required.

A summary of the types of boring methods is given in Table 39.22.

**Cone Penetration Test (CPT).** Now widely accepted as a reliable subsurface investigative tool, the CPT is used in combination with SPT techniques to evaluate the engineering properties of soils. The mechanical CPTs have been replaced with electric friction cone penetrometers that utilize load cells, strain gauges, and special transducers to record even minor changes in the subsurface stratigraphy (see Figure 39.44). Tip resistance, sleeve resistance and pore-pressure measurements can be obtained on a continuing basis through the soil profile. CPT testing is especially useful in obtaining skin resistance for deep-foundation design. (Hunt, 1984.)

**Dilatometer Test (DMT).** Dilatometer testing, like CPT testing, uses a probe-like device that is pushed into the ground and provides rapid, accurate and economical soil parameters of the subsurface. DMT data has proven very reliable for computing strength and consolidation characteristics of soils in-situ.

The test involves thrusting a flat steel blade into the ground and at predetermined depth intervals, then measuring the lift-off and expansion pressure of a flexible membrane located on the face of the blade (Mayne and Frost, TRB 1169). (See Figure 39.45.)

### The Soils Report

The geotechnical consultant provides the land development engineer with the geotechnical information in the form of a report. This information affects not only the preliminary design and layout of the development, but also the final engineering design. Some of this information may be included as part of the specifications, which will reflect on the contractor's approach to construction and his bid estimates. Data included in the report will be of specific use for different aspects of design and construction. For example, the earthwork contractor is concerned about the difficulty of excavation, existence of bedrock, suitability of excavated material for reuse, and the presence of ground water. The foundation contractor is interested in confirming that the soil is capable of bearing support, as well as the difficulty of driving piles or drilling shafts and their expected depths.

The geotechnical report typically is organized as follows, but depending on the nature of the project, some of the listed topics may not be included:

- Scope and purpose of the investigation
- General description of the proposed project

**TABLE 39.21 Guidelines for Boring Depths**

AREAS OF INVESTIGATION	BORING DEPTH
Large structure with separate closely spaced footings.	Extend to depth where increase in vertical stress for combined foundations is less than 10% of effective overburden stress. Generally all borings should extend no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Isolated rigid foundations.	Extend to depth where vertical stress decreases to 10% of bearing pressure. Generally all borings should extend no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Long bulkhead or wharf wall.	Extend to depth below dredge line between $\frac{3}{4}$ and $1\frac{1}{2}$ times unbalanced height of wall. Where stratification indicates possible deep stability problem, selected borings should reach top of hard stratum.
Slope stability.	Extend to an elevation below active or potential failure surface and into hard stratum, or to a depth for which failure is unlikely because of geometry of cross section.
Deep cuts.	Extend to depth between $\frac{3}{4}$ and 1 times base width of narrow cuts. Where cut is above groundwater in stable materials, depth of 4–8 ft below base may suffice. Where base is below groundwater, determine extent of pervious strata below base.
High embankments.	Extend to depth between $\frac{1}{2}$ and $1\frac{1}{4}$ times horizontal length of side slope in relatively homogeneous foundation. Where soft strata are encountered, borings should reach hard materials.
Dams and water retention structures.	Extend to depth of $\frac{1}{2}$ base width of earth dams or $1\text{--}1\frac{1}{2}$ times height of small concrete dams in relatively homogeneous foundations. Borings may terminate after penetration of 10–20 ft in hard and impervious stratum if continuity of this stratum is known from reconnaissance.

- Geologic conditions of the site that describe the physiographic region, the geologic formations, predominant soils, and topographical description.
- Drainage characteristics and facilities
- Methods and details of the exploration program
- Types and results of laboratory tests performed on the samples
- Details of subsurface conditions determined from the exploration and testing program
- Details of the analysis used for design evaluations
- Conclusions and recommendations
- Plan showing the locations of borings

- Logs (results) of the borings (see Figure 39.46), as well as subsurface profiles

### SUMMARY

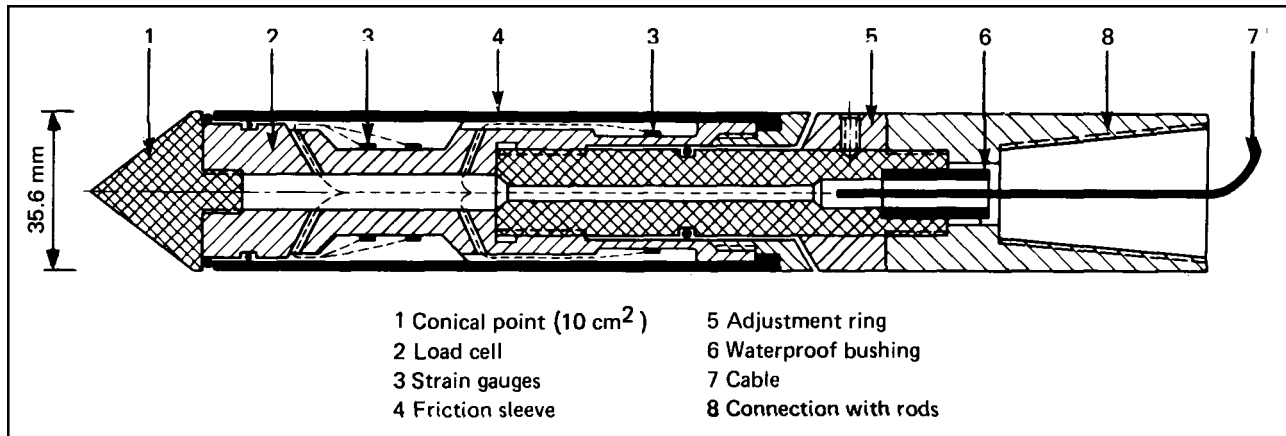
The land design team needs to be aware of the nature of subsurface conditions at the site. These conditions will not only have an impact on how and where specific features are constructed, but also will affect the cost to develop the site.

During the early stages of the design, subsurface information obtained from existing data is often sufficient to make some preliminary decisions about merging the development program to the site. This information is also useful in the planning of the preliminary subsurface investigation. Results from the preliminary investigation produce the in-



TABLE 39.22 Methods for Subsurface Exploration

BORING METHOD	PROCEDURE UTILIZED	APPLICABILITY
Auger boring	Hand- or power-operated augering with periodic removal of material. In some cases continuous auger may be used requiring only one withdrawal. Changes indicated by examination of material removed. Casing generally not used.	Ordinarily used for shallow explorations above water table in partly saturated sands and silts and soft to stiff cohesive soils. May be used to clean out hole between drive samples. Very fast when power-driven. Large-diameter bucket auger permits examination of hole. Hole collapses in soft soils and soils below groundwater table.
Hollow-stem flight auger	Power operated, hollow stem serves as a casing.	Access for sampling (disturbed or undisturbed) or coring through hollow stem. Should not be used with plug in granular soil. Not suitable for undisturbed sampling in sand and silt.
Wash-type boring for undisturbed or dry sample	Chopping, twisting, and jetting action of a light bit as circulating drilling fluid removes cuttings from holes. Changes indicated by rate of progress, action of rods, and examination of cuttings in drilling fluid. Casing used as required to prevent caving.	Used in sands, sand and gravel without boulders, and soft to hard cohesive soils. Most common method of subsoil exploration. Usually can be adapted for inaccessible locations, such as on water, in swamps, on slopes, or within buildings. Difficult to obtain undisturbed samples.
Rotary drilling	Power rotation of drilling bit as circulating fluid removes cutting from hole. Changes indicated by rate of progress, action of drilling tools, and examination of cutting in drilling fluid. Casing usually not required except near surface.	Applicable to all soils except those containing such large gravel, cobbles, and boulders. Difficult to determine changes accurately in some soils. Not practical in inaccessible locations because of heavy truck-mounted equipment, but applications are increasing since it is usually most rapid method of advancing bore hole. Soil samples and rock cores usually limited to 6 inches.
Percussion drilling (Churn drilling)	Power chopping with limited amount of water at bottom of hole. Water becomes a slurry that is periodically removed with bailer or sand pump. Changes indicated by rate of progress, action of drilling tools, and composition of slurry removed. Casing required except in stable rock.	Not preferred for ordinary exploration or where undisturbed samples are required because of difficulty in determining strata changes, disturbance caused below chopping bit, difficulty of access, and usually higher cost. Sometimes used in combination with auger or wash borings for penetration of coarse gravel, boulders, and rock formations. Could be useful to probe cavities and weakness in rock by changes in drill rate.
Rock core drilling	Power rotation of a core barrel as circulating water removes ground-up material from hole. Water also acts as coolant for core barrel bit. Generally hole is cased to rock.	Used alone and in combination with boring types to drill weathered rocks, bedrock, and boulder formations.
Wire-line drilling	Rotary type drilling method where the coring device is an integral part of the drill rod string which also serves as a casing. Core samples obtained by removing inner barrel assembly from the core barrel portion of the drill rod. The inner barrel is released by a retriever lowered by a wire-line through drilling rod.	Efficient for deep hole coring over 100 feet on land and offshore coring and sampling.



**FIGURE 39.44** Fugro Electric CPT probe. (From Hunt, 1984, courtesy of Fugro).

formation needed to make decisions on whether to proceed with the project or abandon it altogether. If the project proceeds, additional subsurface exploration is likely to be required.

The land design engineer must rely on the expertise of the geotechnical engineer for such things as required slopes in cut and fill areas, acceptability of soils for dams and lakes, bearing capacity for foundations, and the presence of high ground water or shallow bedrock.

#### LIST OF SELECTED ASTM TESTING METHODS

##### ASTM D-422 Particle-Size Analysis of Soils

This method covers the quantitative determination of the distribution of particle sizes. The distribution of particle sizes larger than 75  $\mu\text{m}$  (retained on the No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 75  $\mu\text{m}$  is determined by a sedimentation process using a hydrometer to secure the necessary data.



**FIGURE 39.45** Dilatometer testing equipment.

##### ASTM D-423 Standard Test for Liquid Limit of Soils

The water content, expressed as a percentage of the weight of the oven-dried sample, at the boundary between the liquid and plastic states is the liquid limit of the soil. This method covers the determination of the liquid limit of soils.

##### ASTM D-424 Standard Test Method for Plastic Limit and Plasticity Index of Soils

The plastic limit of a soil is the water content expressed as a percentage of the mass of oven-dried soil, at the boundary between the plastic and semi-solid states. The water content at this boundary is defined arbitrarily as the lowest water content at which the soil can be rolled into threads  $\frac{1}{8}$  in in diameter without the threads breaking into pieces.

##### ASTM D-698 Standard Test for Moisture-Density Relations of Soils and Soil Aggregate Mixtures Using 5.5-lb Rammer and 12-in Drop

The laboratory compaction methods cover the determination of the relationship between the moisture content and density of soils and soil-aggregate mixtures when compacted in a mold of a given size with a 5.5-lb (.249-kg) rammer dropped from a height of 12 in. (305-mm).

##### ASTM D-1140 Standard Test Method for Amount of Material in Soils Finer Than The No. 200 (75 $\mu\text{m}$ ) Sieve

This method covers determination of the total amount of material in soils finer than the No. 200 (75  $\mu\text{m}$ ) sieve.

##### ASTM D-1556 Standard Test Method for Density of Soil in Place by the Sand-Cone Method

This method covers the determination of the in-place density of soils. The apparatus described herein is restricted to tests in soil particles not larger than 2 in. (50.8 mm) in diameter.

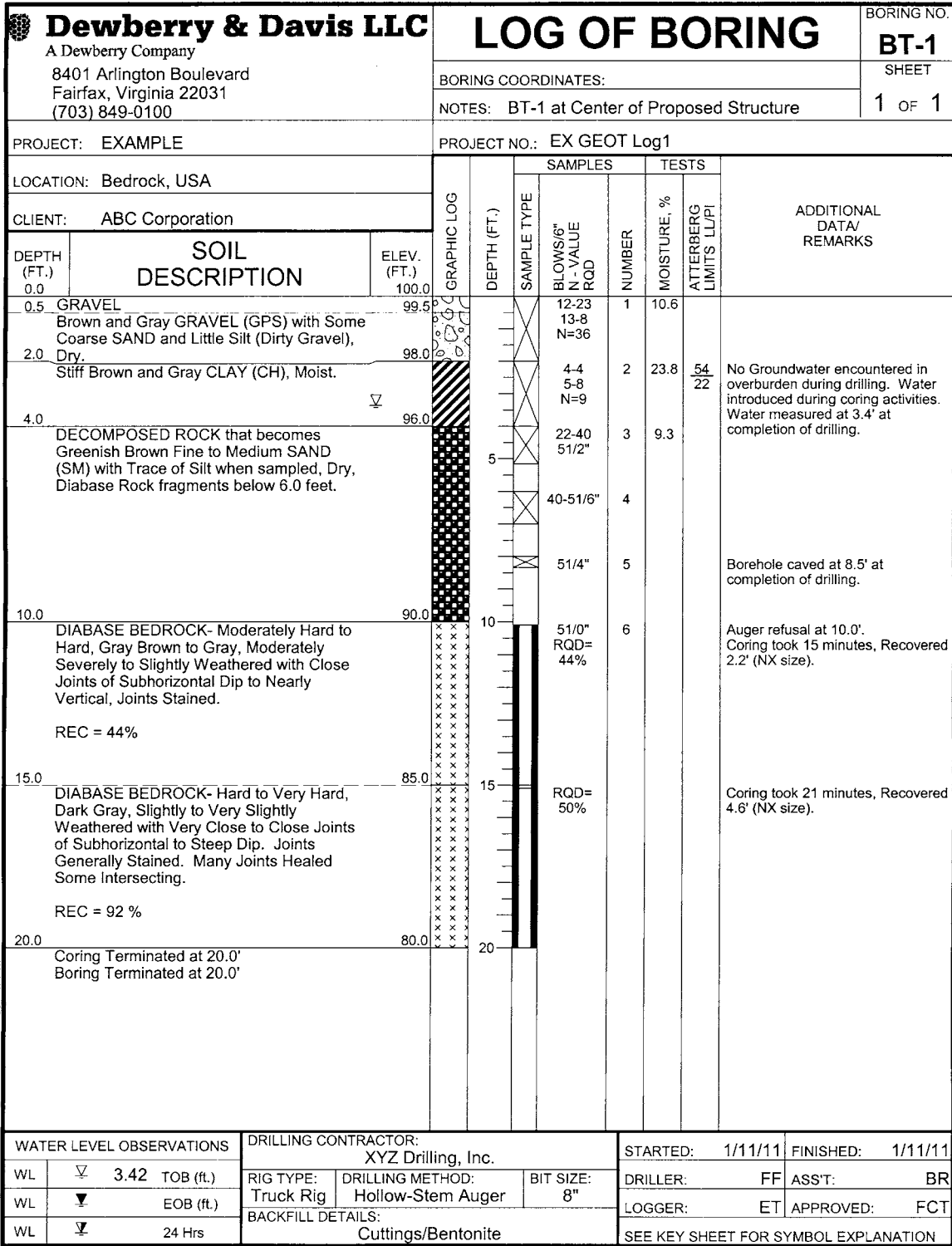


FIGURE 39.46 Boring log.

### **ASTM D-1557 Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb Rammer and 18-in. Drop**

These laboratory compaction methods cover the determination of the relationship between the moisture content and density of soils and soil-aggregate mixtures when compacted in a mold of a given size with a 10-lb (4.54-kg) rammer dropped from a height of 18 in. (457 mm).

### **ASTM D-1883 Bearing Ratio of Laboratory-Compacted Soils**

This method covers the determination of the bearing ratio of soil when compacted and tested in the laboratory by comparing the penetration load of the soil to that of a standard material. The method covers the evaluation of the relative quality of subgrade soils but is applicable to subbase and some base course materials.

### **ASTM D-2166 Unconfined Compressive Strength of Cohesive Soil**

These methods cover the determination of the unconfined compressive strength of cohesive soil in the undisturbed and remolded conditions, using either strain-controlled or stress-controlled application of the test load. The primary purpose of the unconfined compression test is to obtain quickly approximate quantitative values of the compressive strength of soil possessing sufficient coherence to permit testing in an unconfined state.

### **ASTM D-2167 Density of Soil in Place by the Rubber-Balloon Method**

This method covers determination of the density in place of compacted or firmly-bonded soil using a rubber-balloon apparatus. The apparatus is not suitable for very soft soil that will deform under slight pressure or in which the volume of the hole cannot be maintained at a constant value.

### **ASTM D-2487 Classification of Soils For Engineering Purposes**

This method describes a system for classifying mineral and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index, and shall be used when precise classification is required.

### **ASTM D-2850 Unconsolidated, Undrained Strength of Cohesive Soils in Triaxial Compression**

This method covers the determination of the unconsolidated, undrained strength of cylindrical specimens of cohesive soils in the undisturbed and remolded conditions, using either strain-controlled or stress-controlled application of the axial compression-test load where the specimen is an all-around pressure in a triaxial chamber. This method provides for the measurement of the total stresses applied

to the specimen, that is, not corrected for pore-fluid pressure.

### **ASTM D-3080 Direct-Shear Test of Soils Under Consolidated Drained Conditions**

This method describes the procedures for determining the consolidated drained shear strength of a soil material in direct shear.

### **ASTM D-3282 Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes**

This recommended practice describes a procedure for classifying mineral and organo-mineral soils into seven groups based on laboratory determination of particle-size distribution, liquid limit and plasticity index. It may be used when a precise engineering classification is required, especially for highway construction purposes. Evaluation of soils within each group is made by a means of a group index, which is a value calculated from an empirical formula.

### **REFERENCES**

- American Society of Civil Engineers, *Subsurface Investigation for Design and Construction of Foundations and Buildings*, Manual No. 56, 1976.
- American Society for Testing and Materials, *Volume 19: Soils and Rocks*, American Society for Testing and Materials, West Conshohocken, Pa., 1985.
- American Association of State Highway and Transportation Officials, *Manual on Foundation Investigations*, American Association of State Highway and Transportation Officials, Washington, D.C., 1978.
- Asphalt Institute, *Soils Manual*, Manual Series No. 10, 2nd ed., 1978.
- Bowles, Joseph E., *Foundation Analysis and Design*, 3rd ed., McGraw-Hill, New York, 1982.
- Brown, Thomas L., *Site Engineering for Developers and Builders*, National Association of Home Builders, Washington, D.C., 1988.
- Buol, S.W., F.D. Hole, and R.J. McCracken, *Soil Genesis and Classification*, 2nd ed., Iowa State University Press, Ames, Iowa, 1980.
- Clemence, Samuel P., and Albert O. Finbarr, "Design Considerations for Collapsible Soils," *Proceedings of American Society of Civil Engineers, Journal of the Geotechnical Engineering Division*, Vol. 107, No. GT3, March, 1981.
- Das, Braja M., *Principles of Geotechnical Engineering*, PWS Boston, Mass., 1985.
- Dudley, John H., "Review of Collapsing Soils," *Proceedings of American Society of Civil Engineers, Journal of Soil Mechanics and Foundations*, Vol. 96, No. SM3, May, 1970.
- Fang, Hsai-Yang, ed., *Foundation Engineering Handbook*, 2nd ed., Van Nostrand Reinhold, New York, 1991.
- Fitzpatrick, E. A., *Soils: Their Formation, Classification, and Distribution*, Longman, London, 1980.
- Fluet, J.E., Jr., Keynote Address for: "Geosynthetics for Soil Improvement," *Proceedings of the symposium sponsored by the*

- Geotechnical Engineering Division of the American Society of Civil Engineers, Geotechnical Special Publication No. 18, Edited by R.D. Holtz, May 9, 1988.
- Huang, Yang H., *Stability Analysis of Earth Slopes*, Van Nostrand Reinhold, New York, 1983.
- Hunt, Roy E., *Geotechnical Engineering Investigation Manual*, McGraw-Hill, New York, 1984.
- Johnson, L.K., *Homeowners Guide to Overcoming Problems with Marine Clay*, Fairfax County Soil Science Office, Fairfax, Va.
- Liu, Cheng, *Soils and Foundations*, 3rd ed., Prentice-Hall, Inc., Englewood Cliffs, N.J., 1937.
- Mathewson, Christopher C., *Engineering Geology*, Bell & Howell, Columbus, Ohio, 1981.
- Mayne et al., *Journal of Geotechnical Engineering*, Vol. 11, No. 6, June, 1984.
- Mayne, Paul W., and Douglas D. Frost, "Dilatometer Experience in Washington D.C., and Vicinity," *Transportation Research Record 1169*, Geotechnical Instrumentation, 1988.
- Mitchell, J.K., W.C.B. Villet, and A.F. DiMillo, "Soil Reinforcement for Stabilization of Earth Slopes and Embankments," *Public Roads*, Vol. 48, No. 3, pp 88–95, December 1984. U.S. Department of Transportation,
- Moseley, M.P., *Ground Improvement*, CRC Press Inc., Boca Raton, FL., 1993.
- Retaining & Flood Walls*, Technical Engineering and Design Guides, as adapted from the U.S. Army Corps of Engineers, No. 4, 1994.
- Rico Rodriguez, Alfonso, et al., *Soil Mechanics in Highway Engineering*, Tran Tech Publications, 1988.
- Schroeder, W.L., *Soils in Construction*, 3rd ed., John Wiley & Sons, New York, 1984.
- Sowers, George F., *Introductory Soil Mechanics and Foundations: Geotechnical Engineering*, 4th ed., Macmillan, New York, 1979.
- USS Steel Sheet Piling Design Manual*, 1975.
- Winterkorn, Hans Friedrich, and Hsai-Yang Fang, *Foundation Engineering Handbook*, Van Nostrand Reinhold Co., New York, 1975.

## CHAPTER 40

# CASE STUDY: UNION MILLS

*Susan K. Yantis / Lawrence A. McDermott*

*Updated/Revised: Philip C. Champagne P.E., and Jason Graser*

### INTRODUCTION TO THE UNION MILLS PROJECT

The previous chapters have addressed the design and approval process for residential and nonresidential projects. The fact that there are a series of stages and many participants involved in the planning process was also discussed. The federal, state, and local controls, increased citizen participation, and political awareness have transformed a once simple process into an extremely complex one. In fact, there are so many influences and participants that impact and shape the outcome of a project that it is likely that the completed project will have little resemblance to the one originally envisioned by the developer.

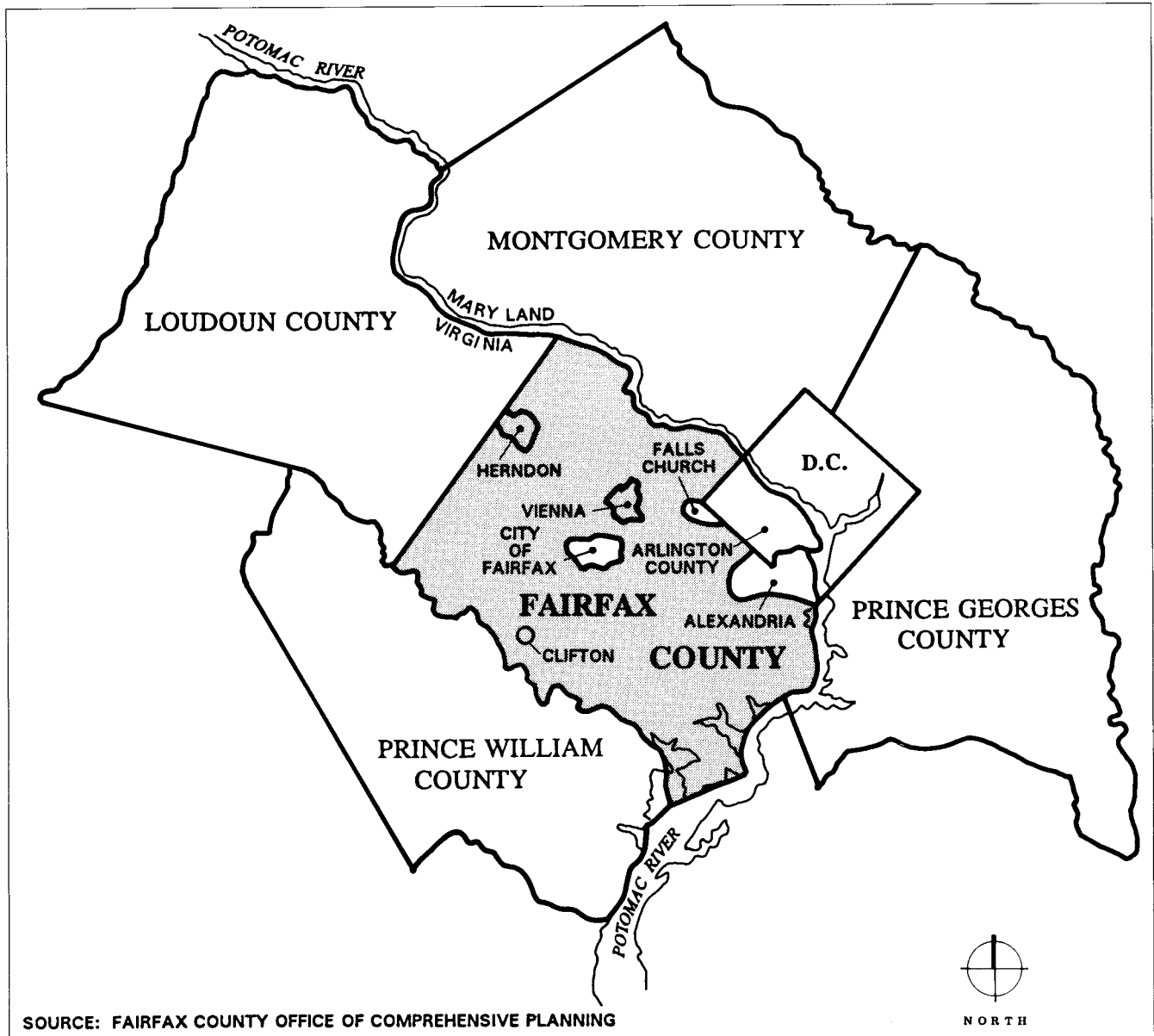
This chapter will review the design process of an actual project known as Union Mills, located in Fairfax County, Virginia. From the conceptual planning stage through the local regulatory process and citizen input to the final approved plans, we will document the evolution of this particular project. Hopefully, this chapter will demonstrate how the efforts of a land development team, in concert with the local public officials, County planning staff, and the surrounding community, resulted in a successful project. Additionally, the review of the planning process specific to a project will illustrate how many of the objectives and goals of all the participants can conflict and how through negotiation and compromise they can be resolved. It must be stressed that this chapter is not a “how to” account of conflict resolution, but rather a discussion of how, in this particular case, issues were identified by a myriad of interest groups and ultimately resolved. The variables in both the process itself and issues raised by interest groups during the development process are too numerous and diverse to lend themselves to a textbook qualitative or quantitative

analysis. Economics, creative engineering/design solutions, compromise, communication skills, genuine citizen concern, and raw politics are all elements of a successful planning/rezoning effort.

### PART I: FEASIBILITY AND SITE ANALYSIS

The Union Mills project is located in Fairfax County, Virginia, approximately 25 miles outside of Washington, D.C. With a population of 827,100 in 1991, Fairfax County has become the largest jurisdiction in the Washington metropolitan area (Figure 40.1) (Fairfax County, 1991b). It has been transformed from a suburban bedroom satellite into a complex commercial, residential, office and research community. Fairfax County experienced a rapid rate of growth during the past three decades. Much of the growth during the past decade has occurred in the western portion of Fairfax County, where the Union Mills project is located. This area's potential for development was enhanced by its proximity to Washington-Dulles International Airport and ease of access to several major roadways. These features have attracted numerous employment centers and new residential developments to this portion of the county.

The western portion of the County was an ideal location for the development of the Union Mills site. As shown in Figure 40.2, the property is located between two areas identified in the County's Comprehensive Plan as suburban centers. These areas are targeted for a complementary mixture of office, retail, and residential uses in a cohesive low to moderate intensity level. The Centreville Area, which is located to the west of the Union Mills project, consists of commercial mixed use and high-density residential uses in the core area with a transition to lower residential densities on the outer edges. Located east of the Union Mills site is



**FIGURE 40.1** Washington metropolitan area.

the Fairfax Center Area: “The Fairfax Center Area is characterized by a mixture of uses including a substantial amount of office space, housing of various types, public facilities and regional- community- neighborhood serving retail uses” (Fairfax County, 1991a). These development nodes are served by Interstate 66, which provides access to Washington, D.C. and Lee Highway (Route 29), a principal arterial roadway. All of these factors supported the development of residential and commercial uses on the site.

The property that became the Union Mills project originally consisted of two parcels containing a total of approximately 108 acres and identified on the Fairfax County zoning maps as 55-3 ((1)) 43 and 46. A copy of the zoning map is shown as Figure 40.3. The property is bordered by a four-lane divided primary highway known as Lee High-

way (Route 29) to the north and a two lane minor arterial road identified as Braddock Road to the south. The surrounding area generally consisted of older single-family detached development on large parcels with a few spot commercial uses along Lee Highway.

After several years in the planning and zoning process, the project was developed by Hazel/Peterson Companies as a mixed use community consisting of a total of 368 residential units with a mixture of single-family detached homes and townhouses at a density of 3.99 dwelling units per acre, supporting recreational amenities, and a child care center. In addition, a neighborhood shopping center including a service station, fast food restaurant, and drive-in bank was developed to serve the residents of the community and the surrounding area.

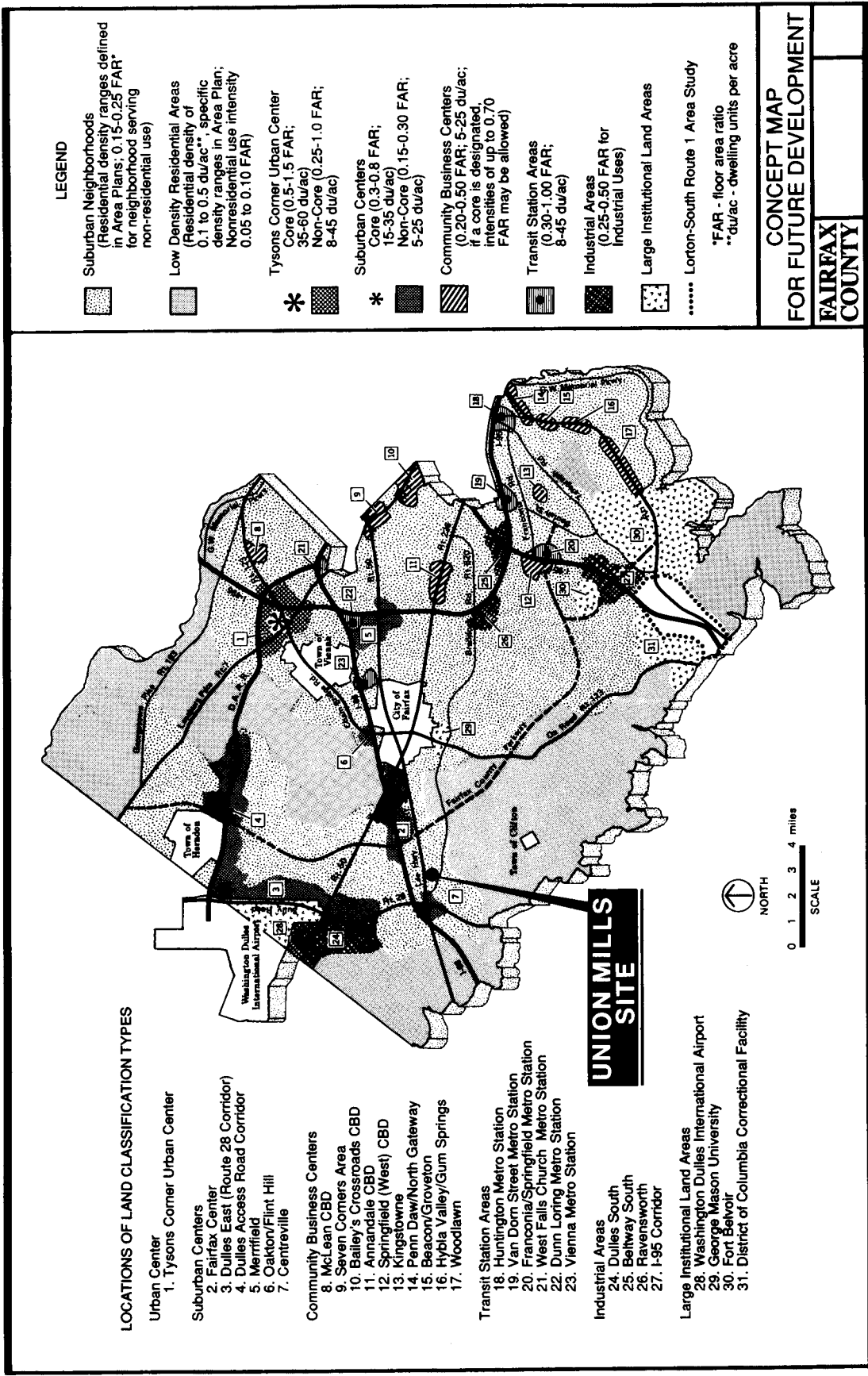


FIGURE 4.0.2 Concept map for future development.



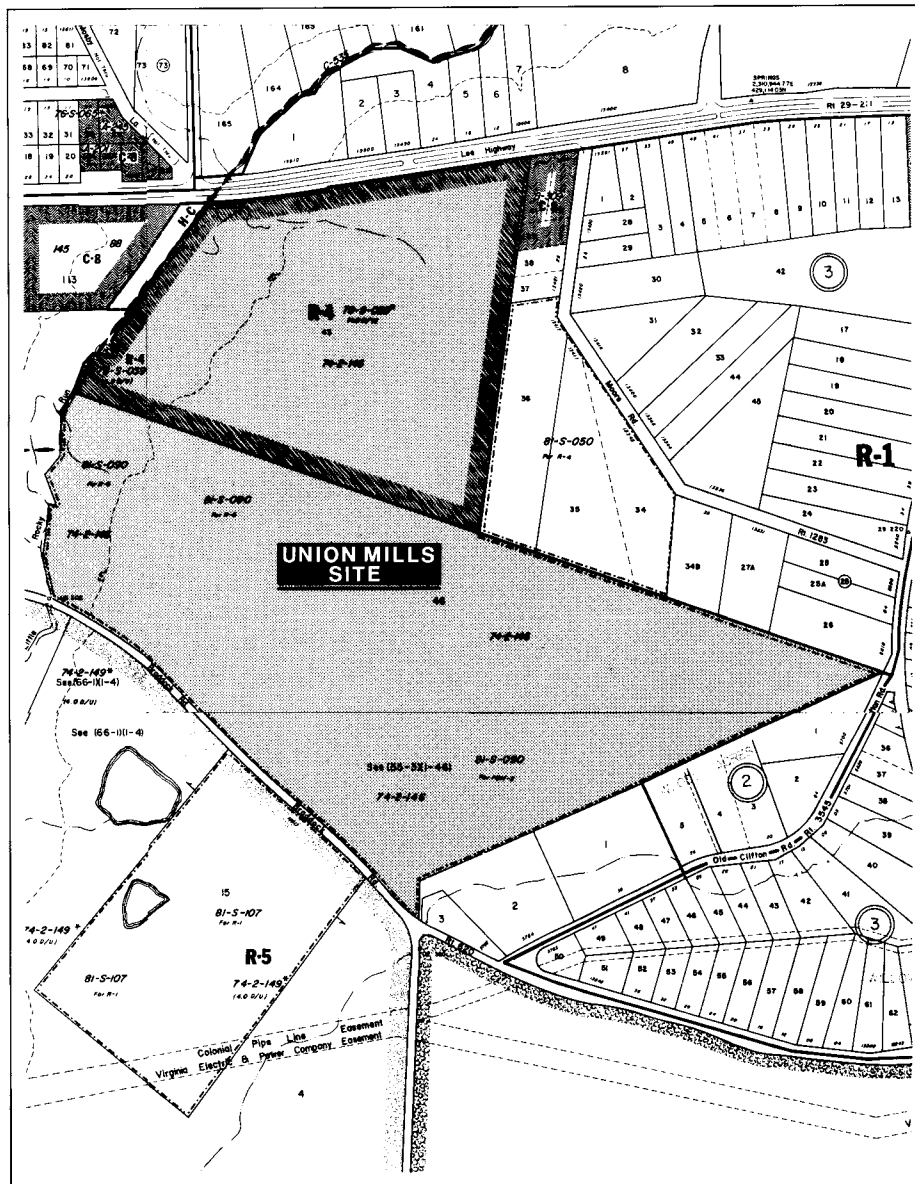


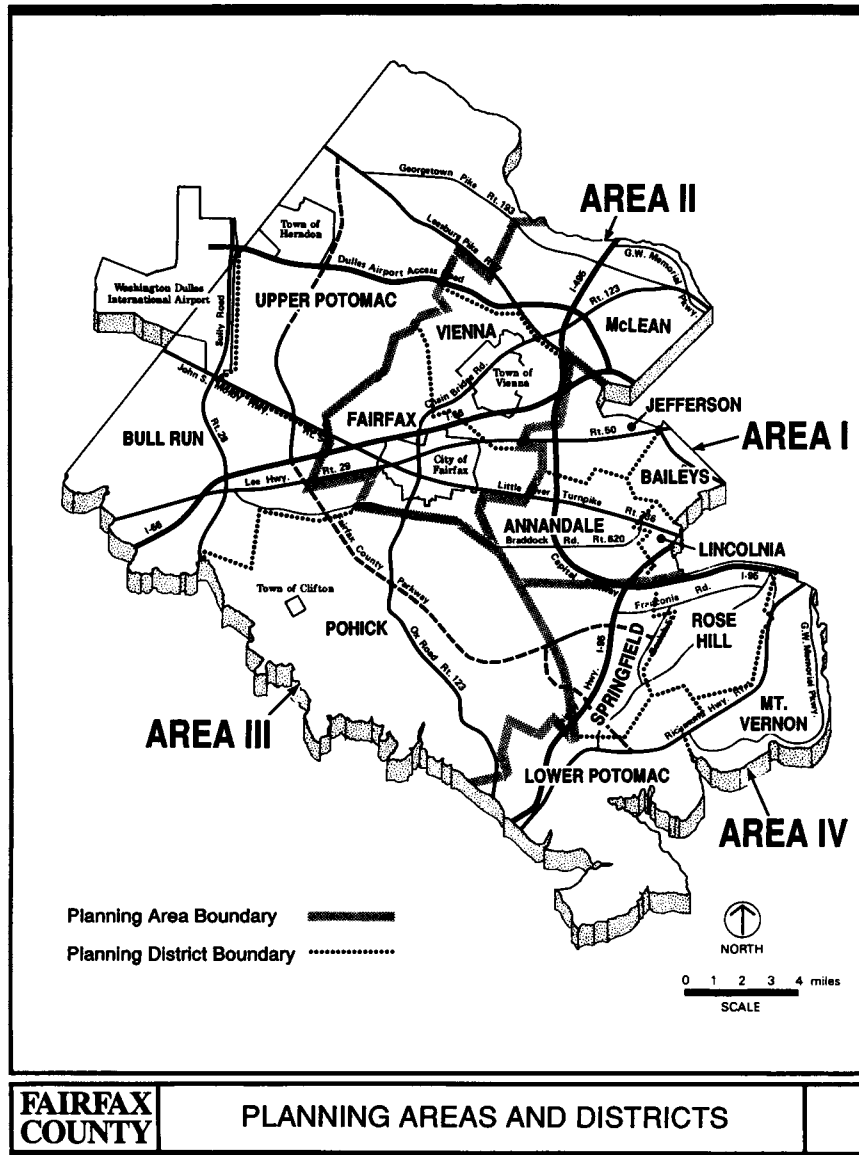
FIGURE 40.3 Zoning map.

### Comprehensive Planning and Zoning in Fairfax County

As discussed earlier, one of the first steps for the land development team is to evaluate the County's Comprehensive Plan. This step is crucial in Fairfax County since the Comprehensive Plan is used as the basis for land use decisions. Although a comprehensive plan is only a guide and not a legal document per se, practically speaking, development in Fairfax County must be in accordance with the Comprehensive Plan. It is extremely rare for a development proposal to be approved by the Planning Commission or Board of Supervisors that is not in strict conformance with the Comprehensive Plan.

The importance and emphasis on the reliance of the County's Comprehensive Plan resulted from the Planning and Land Use System (PLUS) Program in the early 1970s. This Plan evolved from the commitment of the Board of

Supervisors to achieve improved planning and growth control. The PLUS Program included widespread and effective citizen participation through the creation of task forces, surveys, and countless public meetings. The County residents played a major role in the preparation of the Comprehensive Plan and continue their influence on the land development process today. The PLUS Program was implemented through the updating of the countywide Plan and 14 planning district plans in the Comprehensive Plan in 1975. The original 14 planning district plans were grouped in four logical areas of the County, Areas I, II, III, and IV, as shown in Figure 40.4. Each planning district is further broken down into community planning sectors. Land use recommendations are presented for each planning sector. In addition to the parcel-specific land use recommendations, the County's Comprehensive Plan includes jurisdiction-wide



**FIGURE 40.4** Planning areas and districts.

policies relating to land use, transportation, environment, public facilities, and housing.

One significant element of the County's Comprehensive Plan that impacts land development is the designation of an environmental quality corridor (EQC) system. The EQC system contains environmentally sensitive areas that consist of streams, their associated floodplains, poor soils, wetlands, and steep slopes, which are to be protected as part of an open space system. Since the preservation of EQCs is only a recommendation of the County's Comprehensive Plan, dedication of these areas by a private land owner is only obtained through the rezoning, special exception, or special permit process.

As is typical of other jurisdictions, the County's Comprehensive Plan contains a map known as the Land Use Plan. This is a graphic representation of the land uses as recommended in the Comprehensive Plan for the entire

County. Residential development is categorized in the Plan and Land Use Map with density ranges, that is, 1–2 du/ac, 2–3 du/ac, etc. Areas proposed for commercial uses, open space and recreation, and public facilities are also identified on the Land Use Plan.

The review of the Comprehensive Plan by Dewberry & Davis LLC as related to the Union Mills project brought to the client's attention several considerations for the development of the subject property. The property was located in Area III of the Bull Run Planning District and the Centreville Community Planning Sector, as shown in Figure 40.5.

The land use recommendation applicable to the property at that time was: "Land generally located: (1) between Interstate 66, Old Mill Road, Newgate Forest and Route 28, and (2) between Little Rocky Run, Clifton Road, and the Twin Lakes Golf Course facility should be residential use in

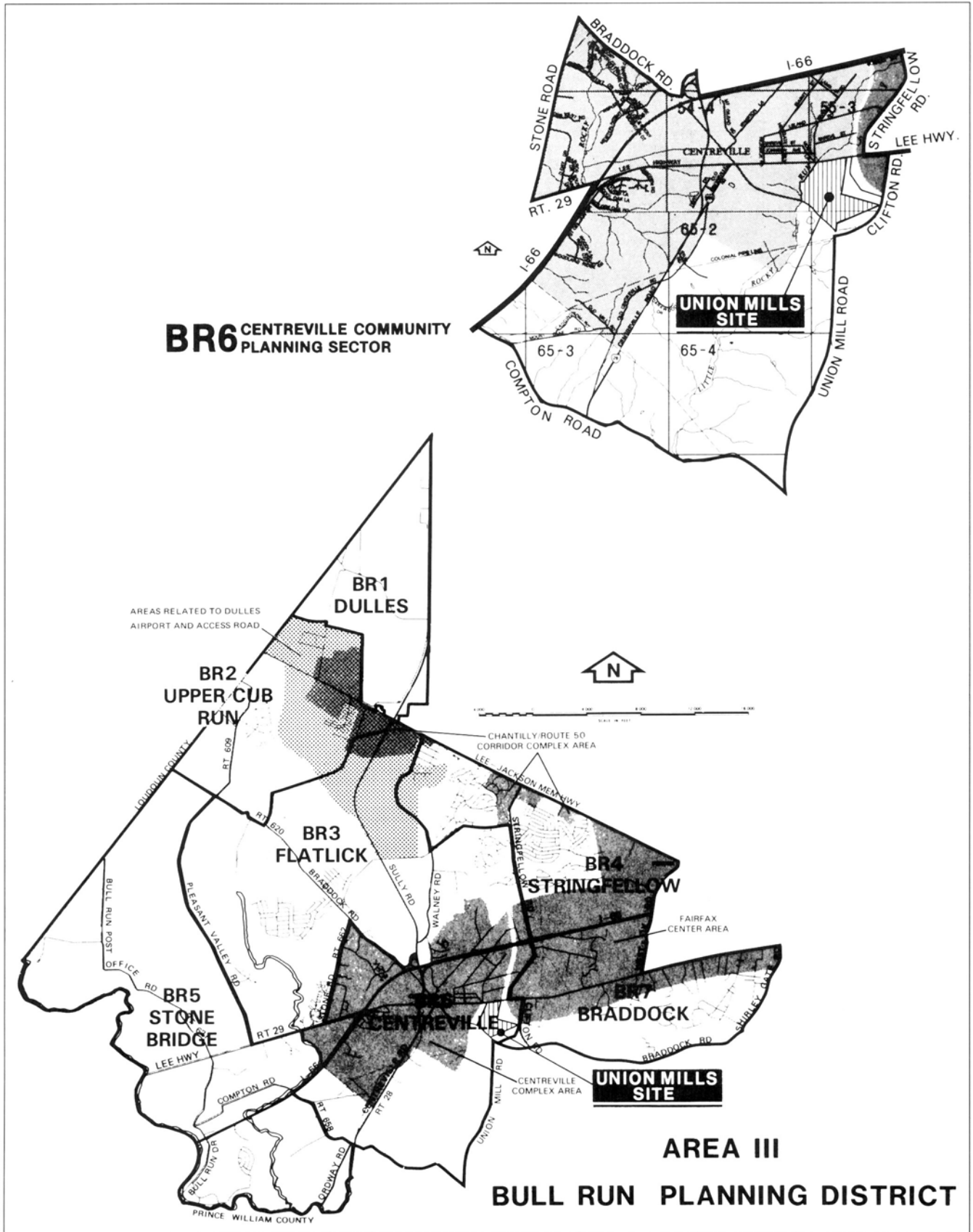


FIGURE 40.5 Bull Run Planning District/Centreville Community Planning Sector.

the 2–3 dwelling units per acre range as a transition away from higher density uses in the Centreville mixed land use areas” (Fairfax County, 1975).

Other applicable recommendations in the Comprehensive Plan that would have an affect on the development of the subject property included the preservation of the Little Rocky Run Stream Valley and its tributaries and the widening of Lee Highway (Route 29) to six lanes.

The implementation element of the Comprehensive Plan contains development criteria for residential density ranges, which were applicable to the development of the Union Mills project. Development Criteria are criteria by which all proposed rezoning applications are evaluated in terms of impacts on public facilities, transportation, environment, and so on. To obtain a favorable credit for the criteria, the applicable impacts must be mitigated.

Another important aspect of the comprehensive planning and zoning process is to be aware of how local governments operate in accordance with their jurisdiction size, structure of government, legislative authority granted by the State, and amount of participation by the residents of the community. In order to understand the land development process as it relates to Union Mills, it is important to become familiar with the basic structure of the government of Fairfax County and the participants in the process.

The governing body in Fairfax County is the urban County Board of Supervisors. The citizens elect one supervisor from each of the nine supervisor districts and an at large chairman. The entire board is elected every four years. The Board of Supervisors plays a major role in the land development process. They formulate and adopt ordinances and policies relating to the development process as well as review and approve rezoning/special exception applications and amendments to the Comprehensive Plan. Although the Board of Supervisors delegates the preponderance of the coordination and negotiations of issues relating to land use to the Planning Commission, the Board of Supervisors makes the final decision on the majority of land use matters. The Planning Commission is a 12-member body appointed by the Board of Supervisors. Each Board of Supervisor member appoints a Planning Commission member to represent his or her district for a four-year term, and the entire Board approves three at-large members. Generally, the Planning Commission holds public hearings twice a week. Historically, each Board member relies heavily on the Planning Commission recommendation on land use matters. Although there have been rezoning cases in which the Board of Supervisors has overruled a Planning Commission recommendation, this is atypical of the process.

As a result of the increased sophistication of the citizens and their influence in the land development process in Fairfax County, organizations made up of representatives of various homeowners associations have been formed within the individual Supervisor Districts to advise the Planning Commission and Board of Supervisors on land use matters. For example, the Western Fairfax County Citizens Association

(WFCCA), a bipartisan volunteer group established in 1980, advised the Planning Commission and Board of Supervisors during the planning process of Union Mills. Referred to in a local newspaper as the “Watchdogs of the Community,” the WFCCA is composed of 11 members, consisting of 6 voting members and 5 alternate members (Barne, 1987c, p. 3). These 11 members are elected annually by the President or other appointed representative of each of the 32 homeowner associations in the western portion of Fairfax County included in WFCCA. These types of citizen organizations have become so influential in the land development process that many Supervisors and Planning Commission members strongly suggest that the applicants present their proposals to these groups for their recommendation prior to the Planning Commission public hearing. It is difficult to receive a favorable recommendation from the Planning Commission or Board of Supervisors without the approval of these citizen groups. These groups typically meet at least once a month to discuss land use matters and listen to presentations from individual applicants for rezoning, special exception, or plan amendment proposals. It is not uncommon to have to meet with these groups several times prior to the Planning Commission public hearing in order to adequately address their concerns.

In addition to the political decision-makers, there are usually homeowners in proximity to the proposed development who have concerns or self-serving interest regarding the proposal. Sometimes these citizens are not part of a homeowners association and lack representation within one of the larger community citizen organizations. The members of the Planning Commission and Board of Supervisors rely heavily on input from the adjacent homeowners, and their concerns must also be addressed along with those of the larger citizen organizations.

### Site Analysis of Union Mills

As stated previously, the Comprehensive Plan recommended the site for residential use at two to three dwelling units per acre. The base level or low end of the density range is two dwelling units per acre. In order to gain a rezoning to allow development at this base level, the project must satisfy all applicable County ordinances and policies. To achieve a density above the low end of the density range, a detailed evaluation of the development criteria set forth in the Comprehensive Plan is used to determine an appropriate intensity of development. These development criteria include development issues such as proffering to a development plan meriting good design, committing to phasing of the development where appropriate, and providing transportation, affordable housing, open space, etc. above the minimum ordinance requirements.

The fulfillment of the development criteria is accomplished through the proffer system, introduced to the land development process in July 1975 as a result of State legislation that enabled Fairfax County to accept commitments

from developers during the rezoning process to make a development proposal more acceptable. Proffers, by state statute, are voluntary commitments made by a landowner/developer and can range from cash contribution for transportation improvements to dedication of land for schools or fire stations to a commitment to provide a specific number of affordable dwelling units (ADUs). Since the introduction of the proffer system in Fairfax County, the applicant proffers all development plans that accompany rezoning applications. The proffering of the plan assures those interest groups, including concerned citizens, that the commitments made by the applicant will become reality. The proffers become part of the rezoning and must be honored by whomever eventually develops the property. A change in a proffered plan requires a new zoning application and the same process as with the original zoning of the property.

Another important early step in the land development process is to research the zoning history of the property. As mentioned previously, the subject property consisted of two large parcels containing approximately 108 acres. During the background investigation of this site it was learned that a rezoning application was submitted in 1974 for the entirety of the subject property, as well as additional properties, by a previous owner. The application was to permit a mixture of townhouses and multifamily dwelling units. This application was withdrawn due to problems in assembling the land. In 1978, the applicant for the parcel located adjacent to Lee Highway (Route 29), which consisted of approximately 39 acres, sought to rezone the property to the R-5 District<sup>1</sup> at a density of 4.7 dwelling units per acre. However, the Board of Supervisors denied the request to the R-5 District and rezoned the property to the R-4 District<sup>2</sup> instead. The approved rezoning was for 156 single family detached residential units at a density of 4.0 dwelling units per acre. Although at that time the comprehensive plan recommended the property for residential use at four to five dwelling units per acre, the Board of Supervisors did not believe that the application warranted a density of 4.7 dwelling units per acre. The approved proffers contained a commitment to develop in general concept with the Generalized Development Plan (GDP) shown in Figure 40.6 but with a lesser number of units as approved by the Board of Supervisors. In addition, the proffers included a commitment to dedicate right of way along Lee Highway, construction of turn lanes into the site, and dedication of 9.7 acres of open space to the future homeowners' association with development of passive recreation facilities.

Since the background investigation on this previous rezoning indicated that the site was at one point recommended in the Comprehensive Plan for residential use at

four to five dwelling units per acre, Dewberry & Davis researched the background and timing of an amendment that changed the recommendation to two to three dwelling units per acre for this property. It was determined that subsequent to the approval of the 1978 rezoning, the Board of Supervisors initiated a study to evaluate the relationship of land use planning and water quality in the Occoquan Reservoir, a source of drinking water supply for over 800,000 people. In an effort to protect the public water supply, the Board of Supervisors implemented the recommendation of the study by adopting a new land use plan for the watershed and a comprehensive rezoning for a portion of the watershed in June 1982. The Union Mills site is located in this watershed and as a result of the new adopted land use plan was downplanned from residential use at four to five dwelling units per acre to two to three dwelling units per acre.

Although a site analysis is typically performed prior to the actual purchasing of the property, Hazel/Peterson did this evaluation subsequent to land acquisition. Hazel/Peterson was very familiar with the development process and expectations in Fairfax County due to their experience on other large-scale projects in the area. Their knowledge and expertise allowed them to purchase land with confidence that they had made a sound investment, and they assembled the two parcels in 1981 with the intent to develop residential uses as recommended in the Comprehensive Plan.

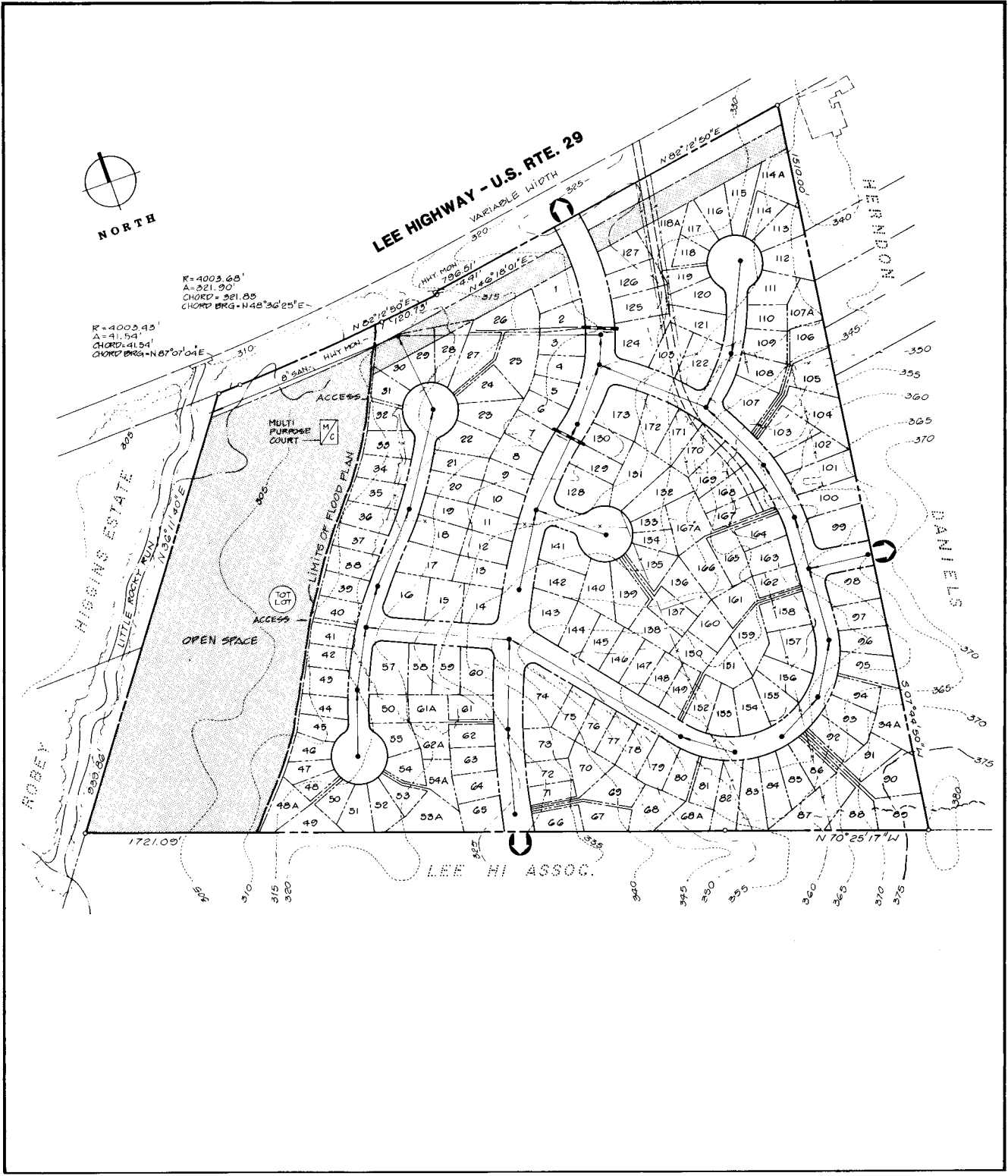
As described previously, Dewberry & Davis performed a "quick and dirty" site analysis, which entailed a review of the essential factors related to development of the property and identified potential constraints. Topography with a contour interval of 2 ft was compiled for the site based on an aerial photographic survey and the 100-year floodplain was identified to be located on the western portion of the property. The presence of a floodplain necessitated further evaluation to determine the limits of the EQC, as defined in the Comprehensive Plan. That analysis indicated that approximately 14 acres (of the 108-acre site) were included in the EQC. During the rezoning process, the County requested that the applicant preserve this area and dedicate it to the Park Authority, and the applicant agreed. However, it should be noted that Fairfax County allows the density for the property to be calculated based on the entire land area, including the EQC, in order to provide the incentive for private land owners to dedicate these areas viewed to be environmentally sensitive.

Given the existence of a stream on the property, a wetland analysis would typically have been performed. However, at the time this project was being developed, the federal, state, and local controls relating to wetland protection, investigation, and mitigation were not developed as they are today.

As part of the feasibility study, the location of utilities was evaluated and determined to be adequate. The access

<sup>1</sup> Residential use at 5 dwelling units per acre.

<sup>2</sup> Residential use at 4 dwelling units per acre.



<p><b>Dewberry &amp; Davis</b> Architects Engineers Planners Surveyors</p>	<p><b>UNION MILLS</b> Generalized Development Plan for Single Family Detached Units September 1978</p>
--	--

FIGURE 40.6 Generalized development plan for single-family detached units.

to the property and planned improvements to the existing public roads along the frontages of the site was evaluated to determine probable right of way dedication.

Subsequent to the site analysis, a yield study was performed in order to determine the maximum number of residential units that could be developed on the site. Although a portion of the property was already zoned R-4 for single-family detached units, the applicant wanted to evaluate the possibility of the development of townhouses on the entire site. The view was that townhouses would be consistent with the planned development to the south of the property. Using the PDH (Planned Development Housing) District provisions of the Zoning Ordinance to allow for greater flexibility in the design, the yield study was performed. (The benefits and opportunities inherent in the PDH district are discussed in Chapter 6). As a result of the site analysis and yield study, Hazel/Peterson Co. decided to pursue the necessary rezoning of the property to develop townhouses on the entire site.

### The Rezoning Process

The development process in Fairfax County is complex and involves many submissions, review, and approval steps before a development project can be completed. The first step is to secure the necessary zoning approvals to permit the proposed land uses. Since the property was zoned R-1 and R-4 with proffers, a rezoning application was required to rezone the entire property to permit the development of townhouses. A rezoning application was filed in September 1981 to rezone the entire 108.5 acres from the R-1 and R-4 Districts to the PDH-5 District. The application was submitted along with a conceptual development plan (CDP) to permit the development of 542 townhouse units for an overall density of five dwelling units per acre.

Once the application for rezoning (or special exceptions) was filed with Fairfax County, a staff coordinator was assigned to the case. The staff coordinator is a professional planner employed by the County who manages the staff evaluation of the application and prepares a staff report. The staff coordinator distributes copies of the development plan, which is submitted by the applicant showing the layout of the proposed development to various County agencies for review and comment. These agencies comment in terms of the project's impact on public facilities and utilities and conformity with applicable regulations and policies. Given that these various agencies have differing objectives, it is not uncommon to have conflicting goals, objectives, and comments. It is the staff coordinator's responsibility to disseminate these comments and negotiate with the applicant on proffers and revisions to the development plan.

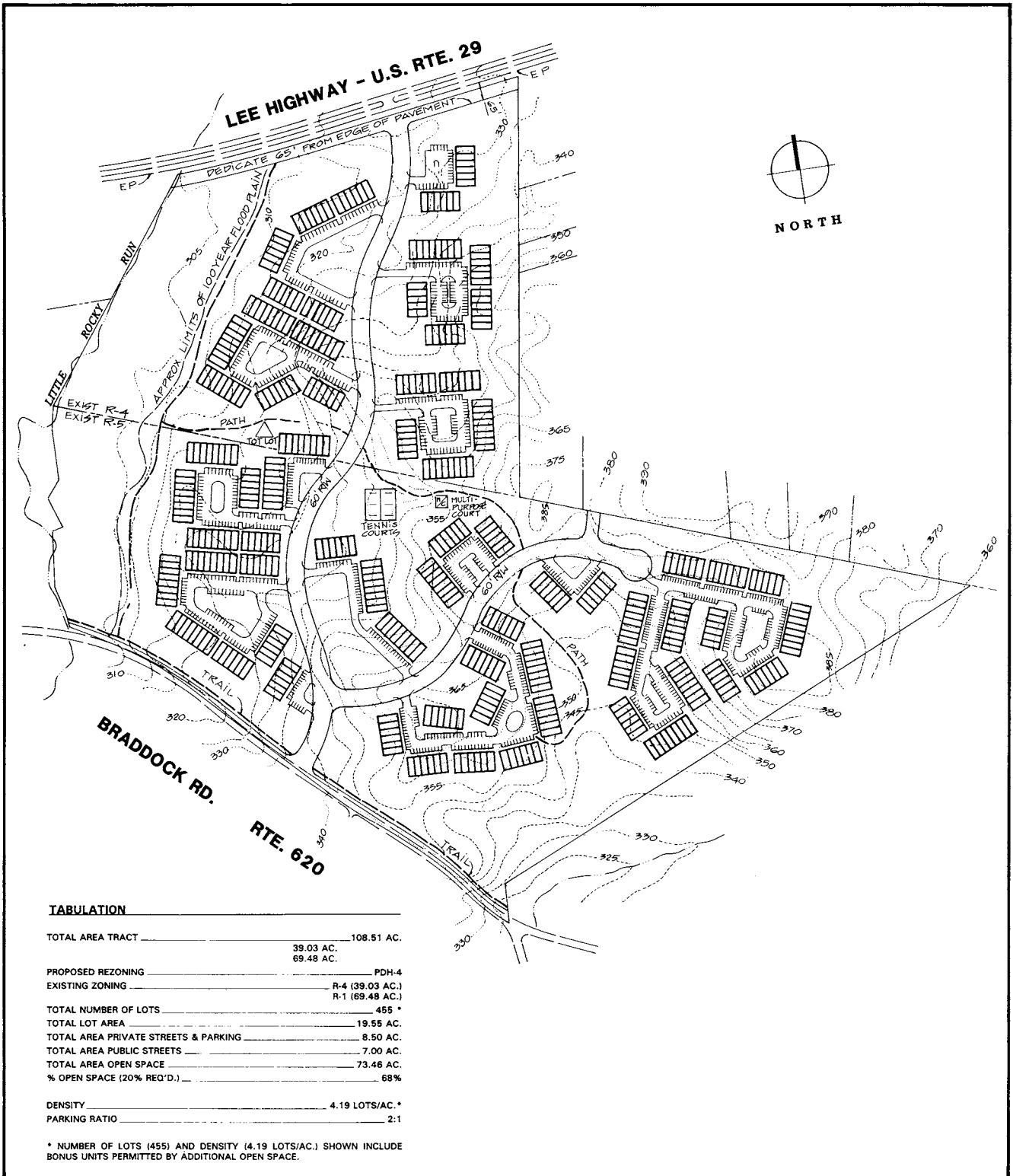
Typically, the applicant for a rezoning or special exception application is the landowner or a contract purchaser of a property. As mentioned previously, Hazel/Peterson Companies is the developer for the Union Mills project. During the planning/zoning process the project was iden-

tified as Centreville Green. Subsequently, the project name was changed to Union Mills for marketing reasons. The applicant for the rezoning of the property was actually Centreville Partnership, whose general partners are John T. Hazel, Jr. and Milton V. Peterson of Hazel/Peterson Companies. It is also typical in Fairfax County for the applicant to retain a development team consisting of a land use attorney and engineering consultant to represent their interest in the rezoning process. The engineering consultant should have strong professional planning capabilities to adequately represent the applicant or alternatively a professional planner is brought in to work with the engineering consultant.

A request for a rezoning to the PDH District is evaluated by two approval processes, a CDP approved by the Board of Supervisors and a Final Development Plan (FDP) approved by the Planning Commission. At the time of the rezoning of the property, a Conceptual Development Plan (CDP) must be submitted to illustrate conceptually how the property is intended to be developed. Public hearings before the Planning Commission and Board of Supervisors are required. At that point in time, the CDP was typically a blob plan, which identified the general location of proposed land uses. A CDP differs from a GDP, which is prepared for rezoning applications to conventional zoning districts in that the GDP shows the locations of the proposed buildings/structures. In this case, however, the CDP, which was ultimately approved, did reflect the location of the proposed townhouses as shown in Figure 40.7, but the development of the property was not proffered to be in conformance with this plan.

Nonetheless, certain elements of the plan were proffered, including the type and number of units, density, and amount of open space to be provided. After discussions with the staff, the initial CDP submitted with the rezoning application was then revised to reduce the number of townhouses from 542 to 455 for a density of 4.19 dwelling units per acre. The reduction in the requested density from five dwelling units per acre was based on several factors. First, the Comprehensive Plan recommendation for the property was for residential use at two to three dwelling units per acre. In recognition of the fact that a portion of the site was already zoned R-4 and several surrounding properties were zoned R-5 but also proffered at four dwelling units per acre, a compromise between the Comprehensive Plan recommendation and the existing zoning was developed. The density was calculated using the existing R-4 zoning at four dwelling units per acre and three dwelling units per acre for the remaining portion of the site in accordance with the high end of the Comprehensive Plan, plus using the bonus provisions set forth in the Zoning Ordinance for the PDH District. Bonus density was merited due to the excessive amount of open space to be provided by the development.

The use of townhouses was accepted by the board of supervisors along with a list of required proffers:




 <b>Dewberry &amp; Davis</b> Architects Engineers Planners Surveyors	<b>UNION MILLS</b> Conceptual Development Plan for Single Family Attached Units February 1982
--	--

FIGURE 40.7 Conceptual development plan for single-family attached units.



- A 100-ft undisturbed buffer was proffered to be provided around the entire perimeter of the property.
- Recreational facilities, consisting of a tot lot, a multipurpose court, and two tennis courts, were proffered to be constructed to serve the future residents of the community.
- Transportation improvements included right-of-way dedication along the public street frontages of the property with provisions of turn lanes into the entrances.
- A roadway was proposed to link from Lee Highway on the north to Braddock Road to the south.
- A \$100,000 contribution towards the future improvement to Braddock Road was also proffered.

The property was rezoned to the PDH-4 District by the Board of Supervisors in September 1982.

The second approval process for the PDH District is the submission of a final development plan (FDP), which is a more detailed plan showing the layout for the subject property. The submission and review process of the CDP and FDP are basically the same, with the exception that only the Planning Commission holds a public hearing and approves the FDP. The Planning Commission considers the FDP in accordance with the approved CDP and determines if it complies with the applicable zoning district regulations. The Planning Commission may establish conditions with the FDP to ensure compliance with the applicable regulations and CDP. A FDP can be filed and processed concurrently with the rezoning application and CDP.

Subsequent to the approval of the rezoning, Hazel/Peterson Co. had Dewberry & Davis prepare a FDP for 18 acres located on the northeastern portion of the property. The FDP shown in Figure 40.8 reflected 83 townhouses and was approved by the Planning Commission in January 1985. The development conditions imposed by the Planning Commission required that a left turn lane from Lee Highway (Route 29) into the site be constructed and the main roadway within the site be a four-lane, 48-ft roadway within a 60-foot right of way. The approval of this FDP would allow the developer to submit a site plan for the first section. Subsequent FDPs would have been required for the remaining 90 acres, but a complete reevaluation of land use was undertaken.

### **Amendments to the Comprehensive Plan Changing Market/Reevaluation of Land Use.**

Since the Union Mills project was rezoned in 1981, a significant amount of development had occurred in the Centreville Area. In 1985, Hazel/Peterson and other developers came to the realization that the amount of existing and planned commercial shopping centers in the western portion of Fairfax County was woefully inadequate to serve the built and planned residential growth.

During this same time frame, Fairfax County conducted a full review of the County Comprehensive Plan. The State enabling legislation requires that "at least once every five years the Comprehensive Plan shall be reviewed by the local commission to determine whether it is advisable to amend the Plan."<sup>3</sup> In furtherance of this requirement, Fairfax County established an Annual Plan Review Process (APR) for the Comprehensive Plan. Every third year the Planning Commission conducts a full review of the Plan, in which nominations can be submitted by citizens, developers, and landowners for Plan Amendments to be considered at public hearings. Nominations must satisfy at least one of the following criteria in order to be scheduled for a public hearing:

1. Significant changes have occurred in the area of concern since the adoption of the Plan.
2. The Plan contains provisions that unreasonably limit the ability of the County to achieve objectives of the Plan
3. Oversights, inconsistencies, or land use-related inequities are contained in the adopted Plan as they affect the area of concern (Fairfax County, 1983, p. 32).

In the intervening years, a limited review of the Plan is conducted. However, an emergency (or urgency) must exist to justify consideration by the Planning Commission. An emergency or urgency is a situation that would jeopardize the health, safety, and/or welfare of the County, or where the existing land use recommendation is not in the best interests of sound land use planning.

During the 1985 APR process and the 1986 limited review process, many nominations were submitted for additional retail commercial facilities in the western portion of the County. One of these nominations was a request filed in December 1985 on behalf of Hazel/Peterson. The nomination was submitted based on the criterion that the adopted Plan contained oversights, inconsistencies, and inequities for the area, as well as provisions that unreasonably limited the ability of the County to achieve the objectives of the Plan. The applicant argued that the existing Comprehensive Plan recommendation of two to three dwelling units per acre for the subject property was not compatible with the four to five dwelling units per acre and five to eight dwelling units per acre on adjacent parcels to the west. Therefore, it was inconsistent with the Plan's objective of promoting compatible development and providing effective transitional densities. They further argued that the Comprehensive Plan acknowledged that no local serving commercial uses were located in the Planning Sector and that additional local serving centers would be required to satisfy growth projections. Therefore, it was requested that

<sup>3</sup> Virginia *Local Planning Legislation, Title 15.1, Chapter 11, Code of Virginia, 1950* as amended, p. 144.

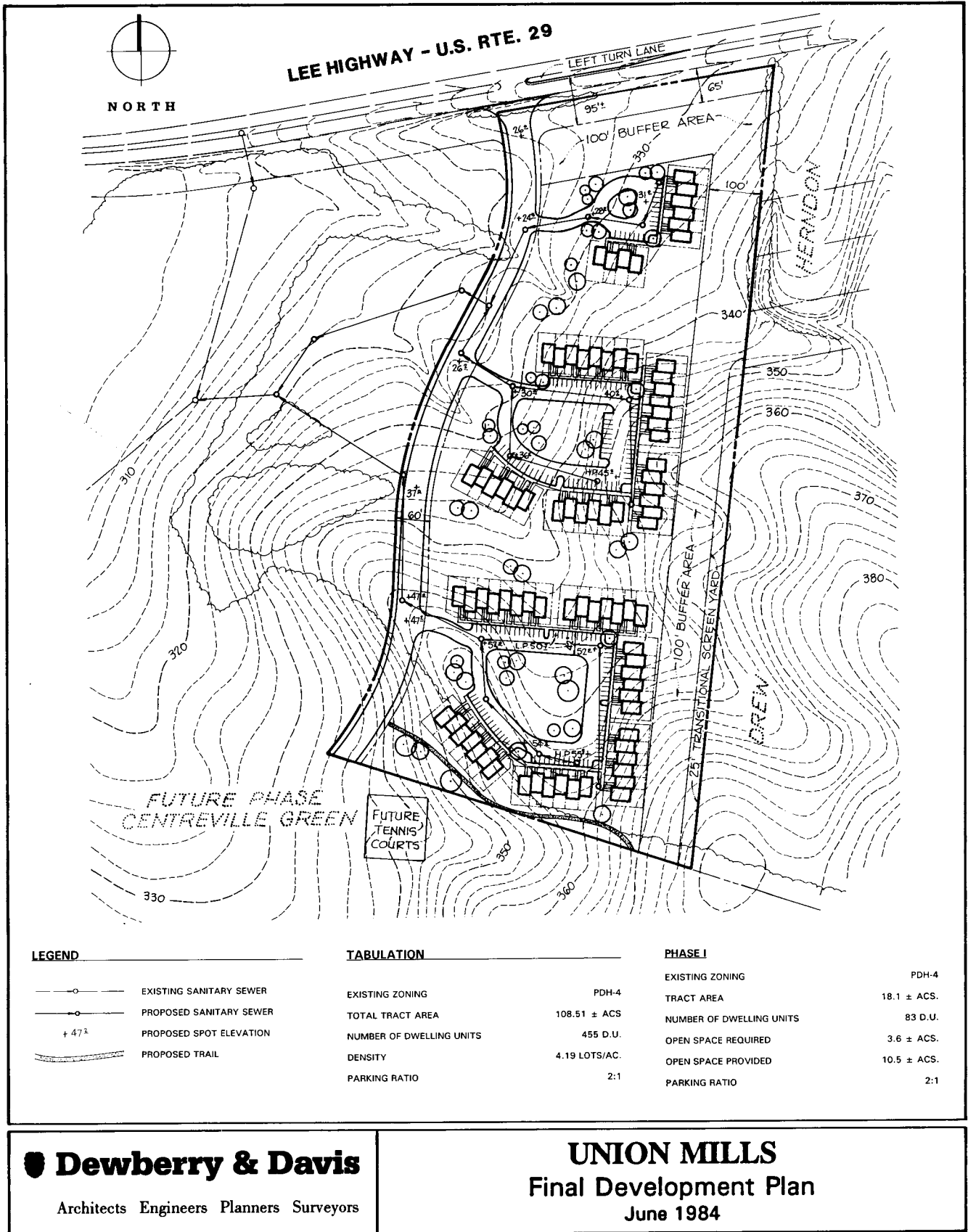


FIGURE 40.8 Final development plan.

the Comprehensive Plan be amended to permit residential development at four to five dwelling units per acre on the site as well as the development of a neighborhood retail center.

In a letter to the Executive Director of the Planning Commission dated February 19, 1986 regarding the Annual Plan Review Nomination, Mr. John "Til" Hazel, Jr. stated that a convenient neighborhood shopping center was necessary to serve the 3000+ plus residential units under development in the vicinity of the subject property. In the event that the retail center was accepted and the approved residential density was maintained, Hazel/Peterson proposed to extend Union Mill Road as a four-lane divided facility north to Route 29 (Lee Highway) and relocate Braddock Road as shown in Figure 40.9 as part of the shopping center development. The applicant argued that these road extensions/improvements were important for Fairfax

County since they were commencing construction of an elementary school and high school that would be accessed by existing Union Mill Road and Braddock Road, both of which were inadequate two-lane roads.

Since the nomination was submitted in a Limited Plan Review Year, an urgency or emergency must exist for the Planning Commission and Board of Supervisors to consider the amendment. The urgency of the Plan Amendment was based on the fact that the plans for the development of the property in accord with the existing PDH-4 zoning were well on their way to approval, which approval would negate the opportunity for Fairfax County to satisfy transportation and retail needs in the area.

**Comprehensive Plan Amendment: Negotiations with Staff.** As a result of the many nominations for retail uses in the western portion of the County, the County staff wanted to evaluate the retail demand for this area and de-

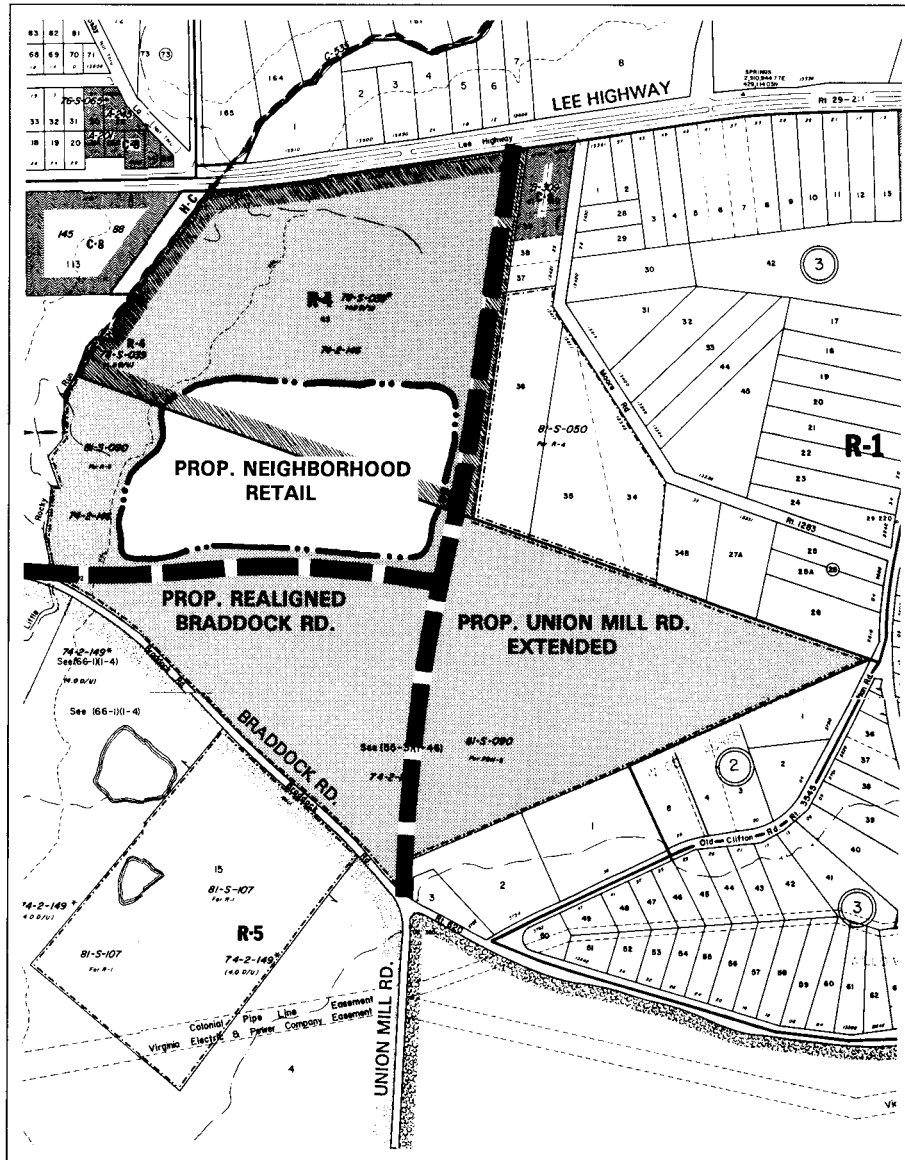


FIGURE 40.9 Proposed Union Mill road extended.

velop guidelines for locating retail commercial facilities. All of the APR nominations that were submitted for retail use in the western portion of the County were deferred pending completion of the Retail Study by County staff. Prior to completion of this study, the County staff initially recommended that the APR nomination for the Union Mills project be screened and not go forward to public hearing because staff believed that the nomination did not demonstrate any urgency or emergency. It was staff's opinion, as stated in the staff report, "that neither the current planned densities nor the densities permitted in the 1982 rezoning of the subject property will unreasonably limit the ability of the County to achieve the objectives of the Comprehensive Plan" (Fairfax County, 1986a, p. 306).

The staff indicated that the results of the pending Retail Study would not recommend the site for a shopping center. Further, it was staff's judgment that the proposed two-lane spine road, proffered with the original rezoning application, connecting Braddock Road west of Union Mill Road to Lee Highway (Route 29) would help convey north-south traffic in the Centreville area. Consequently, extension of Union Mill Road as currently proposed by Hazel/Peterson would not be needed.

The development team continued to argue that the proposed Union Mill Road extension would provide a direct connection to Lee Highway (Route 29) for the 3000+ residential units and future elementary and high school traffic traveling on Union Mill Road. The Planning Commission listened to and was influenced by this argument, and overruling staff recommended that the APR nomination go forward to public hearing as soon as the Retail Study was completed.

Guidelines for locating retail commercial facilities in the County, which were to be included in the Retail Study, were adopted in July 1986. The study "Shopping Centers—Meeting the Demand for Retail Commercial Facilities in Western Fairfax County" (Fairfax County, 1986b) was published in August 1986. The Study identified sites for future retail commercial locations, which were considered in conjunction with those sites nominated during the 1985 and 1986 Annual Plan Review Process. The Study recommended a neighborhood shopping center on a 10–12-acre site located at the intersection of Braddock Road and Clifton Road southeast of the Union Mills property as shown in Figure 40.9.

The Hazel/Peterson team argued that the site recommended by staff consisted of 34 separate parcels owned by 30 separate owners with 12 existing dwellings and would be extremely difficult to assemble. In addition, Hazel/Peterson noted that a shopping center on that site might impact adjacent Residential-Conservation District (R-C) zoned land (one unit per five acres) that was downzoned as a result of the Occoquan Basin Study. In an effort to negotiate with the staff, the development team refined its proposal to reduce the total number of units from 455 townhouse units as permitted under the existing approved PDH-4 zoning to 340 townhouse units and 60 single family detached units,

for a total of 400 units. Hazel/Peterson proposed single-family detached units on the east side of Union Mill Road extended, which was consistent with the Comprehensive Plan recommendation north and south of this portion of the site.

Following extensive negotiations and discussions with the staff, it was determined that the Hazel/Peterson proposal was, indeed, a better location for the neighborhood shopping center. Staff accepted the argument that the competing site was adjacent to low-density residential development and thus did not satisfy the guideline for shopping center locations. Staff further determined that the other site was on the outer edge of an area of very low-density development rather than in an area of suburban or urban density, as was the Union Mills property. The staff ultimately recommended that a neighborhood shopping center be developed on the Hazel/Peterson site, consisting of 10 to 12 acres to serve surrounding residential development. However, contrary to Hazel/Peterson's nomination, staff recommended that the residential development for the remaining portion of the property be maintained at the existing planned density of two to three dwelling units per acre.

**Comprehensive Plan Amendment: Negotiations with Citizens.** As is typical of the Comprehensive Plan amendment process, a public meeting was held for interested citizens regarding all of the plan nominations for retail use in the western portion of Fairfax County, as well as public hearings before the Planning Commission and Board of Supervisors. In addition to this public meeting scheduled by staff, the development team met with the neighboring citizen groups individually to present their proposal. The issues raised at those meetings were related to specific uses within the shopping center, hours of operation of the uses, and the proposed transportation improvements. Initially, there was not much opposition to the proposal.

In March 1986, a Comprehensive Plan Amendment for the Centreville area was adopted by the Board of Supervisors after several years of extensive study and evaluation by a Citizens Task Force. The Centreville Area Plan included a Transportation Plan, as shown in Figure 40.10.

This Plan proposed that Clifton Road, located to the east of the site, be improved to complete the planned loop road system around the Centreville Area. One of the citizens who participated on the Citizen Task Force was strongly opposed to the Hazel/Peterson proposed Plan Amendment since the extension of Union Mill Road conflicted with this recommendation in the Transportation Plan. This citizen argued that the extension of Union Mill Road might entice commuters from adjacent Prince William County to use the Compton and Union Mill Roads, which served the residential areas as shortcuts to and from the workplace. He further argued that the extension of Union Mill Road might defer the funding and construction of another planned vital east-west arterial road in Centreville known as Braddock Road extended. Braddock Road was proposed to be extended from existing Union Mill Road to the west to Route 28,



which would be part of the four-lane divided loop road system around Centreville. The Braddock Road extension was proffered in conjunction with another planned development by Hazel/Peterson known as Centre Ridge, which is located in the core of Centreville. Hazel/Peterson was committed to construct two lanes of the four-lane roadway extension prior to the issuance of a residential use permit for the 2400th dwelling unit in Centre Ridge. Thirty-three hundred and fifty dwelling units were approved to be built in this planned development. The remaining two lanes of Braddock Road extended were to be funded by Fairfax County.

This citizen solicited and gained the support of the influential Western Fairfax County Civic Association (WFCCA), which took the position that they opposed the Union Mills proposal if it would result in increased density and delay the construction of the Braddock Road extension. The WFCCA recognized that the roadway would not be constructed for many years based on the existing proffer. The WFCCA further realized that they could influence the process and, therefore, recommended that construction of the retail center should not occur until such time as Braddock Road extended was constructed. Neighboring subdivisions, including the Board of Trustees for the Little Rocky Run subdivision, a 3000+ unit planned community located to the south of the property, supported the proposal. The testimony presented during the public hearing before the Planning Commission was split by strong opposition of the WFCCA and considerable support by the neighboring citizens for the Plan Amendment.

Following the public hearing, the Planning Commission, on a motion by the District Commissioner, deferred the decision on the Plan nomination for approximately one month. Such a deferral is typical when WFCCA is opposed to a particular development proposal. However, the Planning Commission did allow the record to remain open during the deferral period to receive additional written testimony from the public. The citizen activist opposing the Plan Amendment continued to encourage the community to join WFCCA in the fight against the proposal. Consequently, a petition was filed by approximately 50 homeowners in the Little Rocky Run subdivision opposing the Plan Amendment. The petition stated that the testimony at the Planning Commission public hearing, by the Little Rocky Run homeowners association supporting the proposal, was only the opinion of Board members and not a consensus of the individual homeowners in the area. The other neighboring subdivisions to the east continued to support the proposal.

In an effort to address concerns expressed by the Planning Commission members at the public hearing, a traffic analysis was performed by a transportation consultant retained by Hazel/Peterson. The goal of the study was to quantify and confirm staff's opinion that a neighborhood center at Union Mills would lower the traffic demand in

the downtown Centreville area. The study demonstrated that there would be less traffic in the downtown Centreville area with the location of the shopping center at Union Mills than there would be if there were not a shopping center there. The study showed that the shopping center actually removed trips from key primary roads in the Centreville area. The development team continued to argue that the traffic generated by the proposed retail center did not result in the need for Braddock Road extended and, therefore, should not be linked to the development.

The Planning Commission recommended approval of the Plan Amendment to allow a retail center on the Union Mills property to the Board of Supervisors. However, consistent with the recommendation of the WFCCA, the opening of the retail center was contingent upon the opening of both Union Mill Road extended and Braddock Road extended. In addition, specific language recommending screening and buffering in excess of Ordinance requirements in order to protect the adjacent neighborhoods was included in the Plan Amendment. Consistent with the Planning Commission recommendation, the Board of Supervisors approved the Plan Amendment to allow a neighborhood shopping center on the subject property on January 12, 1987, a full year after the nomination was submitted.

### **The Rezoning Process Revisited**

The approval of the Comprehensive Plan Amendment allowed the development team to prepare and file the rezoning applications necessary to develop the property in accordance with the new Comprehensive Plan recommendation. An application to amend the CDP and proffers, which were previously approved for the entire property when it was originally zoned PDH-4, was required to delete the land area necessary for the shopping center and to revise the remaining portion of the residential layout accordingly. Although the PDH District does allow commercial uses, the amount of land area that can be devoted to commercial uses is limited and the commercial uses must be designed to serve primarily the needs of the residents of the planned development. Since the land area necessary for the proposed retail center exceeded this limitation and was intended to serve the other neighboring communities, a separate rezoning application was required to rezone the approximately 16 acres of the property to the C-6 District (Community Retail Commercial District). A successful rezoning to the C-6 District would permit the development of the neighborhood shopping center, service station, fast food restaurant, and drive-in bank. A combined CDP/FDP was prepared for the residential portion of the development to include a child care center, as shown in Figure 40.11.

A CDP and FDP was combined showing the location of the proposed townhouses, single-family detached lots, and child care center in order to expedite the review process. A separate GDP was prepared for the shopping center with companion special exception applications for the service

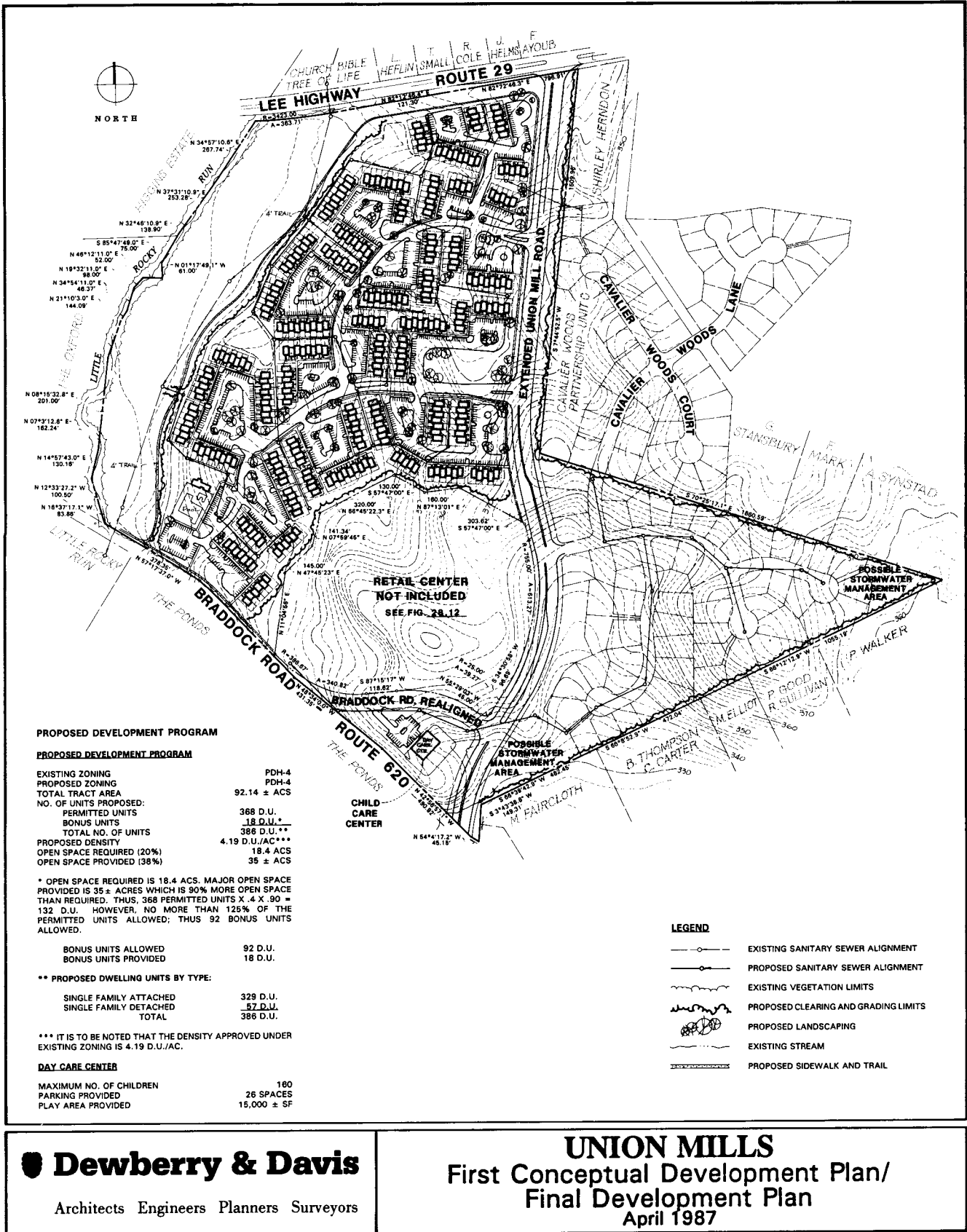


FIGURE 40.11 First Conceptual Development Plan/Final Development Plan—Union Mills (residential areas).

station, fast food restaurant, and drive-in bank, as shown in Figure 40.12. The applications were filed with Fairfax County in April 1987 to be processed concurrently.

As discussed earlier, the site design for the project was a result of several factors. The alignment of Union Mill Road extended required the reconstruction of its intersection with Braddock Road. Knowing that this proposed roadway and intersection would affect the design of the layout, Hazel/Peterson had a preliminary design of the roadways prepared during the rezoning process. This preliminary design was ultimately submitted to the Fairfax County transportation staff and by reference became part of the approved proffers.

The extension of Union Mill Road from Braddock Road to Lee Highway (Route 29) bisected the property, creating two distinct land bays. The eastern side of Union Mill Road extended was proposed for 57 single-family detached homes on lots averaging 12,000 ft<sup>2</sup> to be compatible with the existing and planned development adjacent to the north and south of this land bay.

The property to the west of Union Mill Road extended was proposed for 323 townhouses consistent with the previously approved rezoning. The townhouses provided a transition in land use from the higher-density multiple family units to the west to the single-family detached units to the east of the site across Union Mill Road extended. The proposed density for the residential portion of the layout was 4.12 dwelling units per acre. This included 12 bonus dwelling units resulting from the provision of open space beyond ordinance requirements. The realignment of existing Braddock Road to the north and the creation of its intersection with Union Mill Road extended was an ideal location for the proposed shopping center to consist of 118,000 ft<sup>2</sup> of retail space. This location at the intersection of a minor arterial collector road would provide good visibility for the shopping center and convenient right turn access for vehicles on the trip from work to home. The service station, fast food restaurant, and drive-in bank were located near the entrances to the shopping center to provide for visibility and convenient access. The realignment of Braddock Road to the north created another land bay south of the shopping center that was designed to accommodate a child care center for 160 children to serve the surrounding residential areas.

**Negotiations with Staff.** During the processing of the rezoning application, the County staff presented many issues that had to be addressed. The major issues presented by the staff were related to the proposed density and design/layout of the residential component. The proposed density at 4.12 dwelling units per acre was generally consistent with the original approved rezoning at 4.19 dwelling units per acre. However, it was staff's judgment that the proposed application was not comparable to the original plan and did not justify the high end of the planned density range nor warrant the 12 bonus units proposed through the provision of excess open space. The initial staff report stated that to "allow a 16.37 acre decrease in the size of the subject site

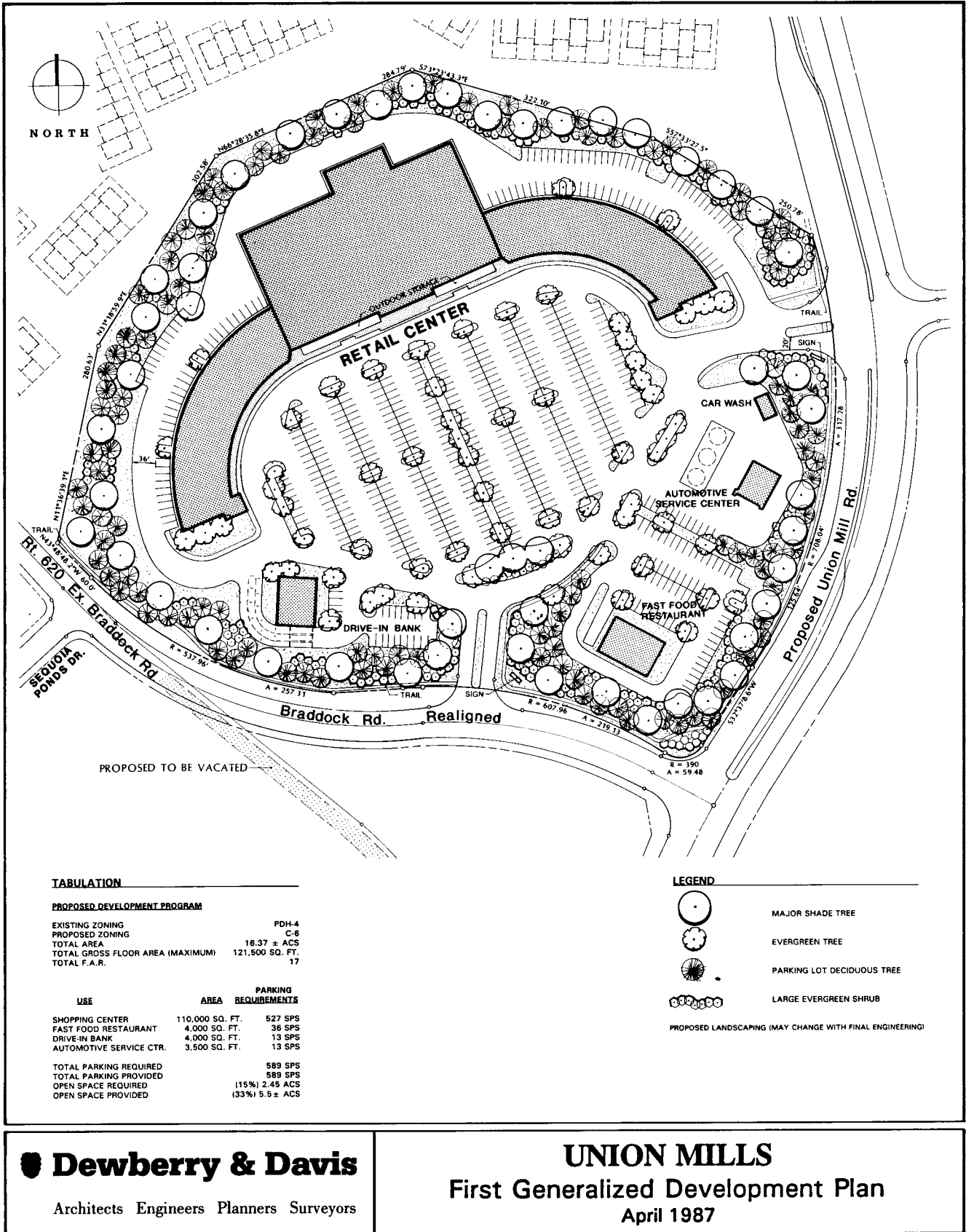
[for the retail center], a change in the unit mix from all townhouses to a combination of 323 townhouses and 57 single family detached dwellings and to add a child care center [would] significantly increase the intensity of the use and development of the application property" (Fairfax County, 1982, p. 22).

The staff report further stated that "the proposed development plan . . . did not exemplify a design layout or features which merit recognition for good design and amenities" (Fairfax County, 1982, p. 13). Staff argued that the proposed single-family attached units were crowded too closely together and that the cluster concept of the single-family attached unit layout did not cluster around any open space areas. Further, it was staff's position that the plan lacked significant areas of landscaped open space that were necessary since the entire site was proposed to be cleared of the existing vegetation. In addition, the staff argued that of the 35 acres or 38% open space being provided, approximately 14 acres were EQC and the applicant would receive density credit for this area. The remaining 21 acres of open space being provided did not justify the 12 proposed bonus units.

In an effort to address the staff concerns on the design, the CDP/FDP was revised and resubmitted to the staff a total of seven times throughout the process. Ultimately, the 12 bonus units were deleted in the single-family attached layout, which satisfied the staff's concern on the density. This resulted in 311 single-family attached units and 57 single-family detached units for a total of 368 units at a density of 3.99 dwelling units per acre. The landscaping was significantly increased along the major roadways and internal streets and within the courts of the townhouse clusters. Further, the applicant proffered landscape details for the townhouse end units, which were proposed to face sides of other end units and at the entrances to the shopping center. A 25-ft undisturbed buffer on the rear yards of the single-family detached lots was provided around the periphery. To protect the adjacent single-family detached units as well as the townhouse units behind the shopping center, landscaped buffers and fencing was provided.

**Negotiations with Citizens: Deadlock Issue.** Once the rezoning applications were filed with Fairfax County, the development team immediately presented the details of their proposal to the neighboring citizen associations. As stated above, the Comprehensive Plan tied the opening of the retail center to the construction of both Union Mill Road extended and Braddock Road extended. It was the Hazel/Peterson team's task to convince the adjacent citizen associations that the shopping center and, more importantly, Union Mill Road extended, would only be delayed if these projects had to wait until Braddock Road extended was in place. Hazel/Peterson knew their only hope to overcome this Comprehensive Plan recommendation was to get these neighboring homeowner associations to support their position. After several meetings, these neighboring homeowner associations voted to support the proposals with the





**Dewberry & Davis**  
Architects Engineers Planners Surveyors

**UNION MILLS**  
First Generalized Development Plan  
April 1987

FIGURE 40.12 First Generalized Development Plan—Union Mills (retail center).

understanding that Union Mill Road would be extended and in service prior to the opening of the shopping center. These citizen groups believed that “they urgently need[ed] more roads in their area and [they] would rather have Union Mill [Road] extended than nothing at all” (Barne, 1987b, 3). The citizens expressed their belief that the construction of Braddock Road extended was important to improving the transportation network in the Centreville Area and that the County should commit to the construction and funding of the additional two lanes concurrent with construction of the two lanes by Hazel/Peterson with the development of Centre Ridge.

The proffers initially submitted by Hazel/Peterson for the rezoning application included a commitment to extend Braddock Road to Route 28 prior to the opening of the retail center. Although Hazel/Peterson continued to argue this requirement, they realized it would be very difficult to obtain approval for a rezoning contrary to the Comprehensive Plan recommendation. However, once the development team received support from the neighboring citizen groups that deleted the linkage of the opening of the retail center to the construction of Braddock Road extended, they realized their argument was strengthened and withdrew the proffer. This revision to the proffers occurred only days before the public hearing before the Planning Commission on the pending rezoning application. As a result, conflicting testimony was presented at the public hearing with the neighboring citizens supporting the rezoning application and the umbrella organization, WFCCA, continuing to strongly oppose the development without the construction of Braddock Road extended. The decision by the Planning Commission was deferred for a week to allow the citizen groups time to resolve the conflict. Prior to the scheduled decision by the Planning Commission, a meeting was held with various representatives of the homeowner associations and WFCCA. The major point of contention continued to be the linkage between the construction of Braddock Road extended and the construction of the shopping center. As stated in a local newspaper, “Although the extension was a hot topic, opponents said it was secondary to what they claimed was a violation of the County master land use plan.” The newspaper article further stated from an interview with one critic, “It wasn’t a question of us being anti-transportation . . . what they were asking for violated the actual planning process by allowing them to manipulate the County’s Comprehensive Plan in terms of zoning” (Moncado, 1987c, p. A8). In an interview with another local newspaper, Jeffrey Saxe, Hazel/Peterson Companies, stated that the amendment to the Comprehensive Plan . . . linked Braddock Road extended to the construction of the shopping center . . . was unfair to the Centreville Partnership. The link is not in there because of the need for Braddock Road extended. This [amendment amounts to] using the planning process for reasons other than sound planning” (Barne, 1987a). The Planning Commission member successfully made a motion to defer the rezoning application

for two months citing deadlock between the citizens and the developer. Three days later, Hazel/Peterson withdrew the rezoning applications since a resolution of the linkage requirement for Braddock Road extended could not be reached. After almost two years of effort pursuing the commercial use, Hazel/Peterson intended to pursue the development under the existing PDH-4 zoning. Dewberry & Davis initiated the preparation of the FDP based on the approved CDP as well as a preliminary plan.

**The Rezoning Revisited Compromise.** The District Supervisor and the neighboring homeowner associations as well as some members of WFCCA were upset that the applications were withdrawn, thereby eliminating the proposed extension of Union Mill Road, which was vital to improving their access to the major roadways. Although Hazel/Peterson was pursuing the existing zoning, these citizens continued to work with the umbrella group of WFCCA in an attempt to reach a compromise. Several months later, a meeting was held by the District Supervisor, representatives from WFCCA, and Hazel/Peterson. At that meeting a consensus was reached that Hazel/Peterson would resubmit the rezoning application with the following commitments:

1. The rezoning application would be considered as it was at the time it was withdrawn. There would be no requests for revisions to the development plans or proffers.
2. There would be no attempt to link the opening of the shopping center with the completion of Braddock Road extended.
3. The rezoning application would be processed on an accelerated basis with completion within four months.
4. Review of the residential preliminary plan, which includes Union Mill Road, extended concurrent with the rezoning. This was to compensate Hazel/Peterson for the six-month delay and additional expenses incurred by Hazel/Peterson as a result of the revision of the development schedule.
5. Hazel/Peterson will accelerate design of Union Mill Road extended and submit engineering plans.

In addition, Hazel/Peterson agreed to accelerate the funding of Braddock Road extended provided that the Board of Supervisors: (1) expedited review and approval of certain plans and plats pending for their Centre Ridge development; (2) approval of a bond issue financing the other two lanes of Braddock Road extended; and (3) initiated condemnation proceedings where appropriate.

Within days of this meeting, the WFCCA submitted a letter to the Supervisor indicating unanimous support for the rezoning application as it was at the time it was withdrawn. The letter dated February 20, 1988, to Supervisor McConnell from Dick Frank, President, WFCCA, stated

that the support was based upon the reality that the building of Braddock Road extended could potentially be put on a fast track as committed by Hazel/Peterson Companies. Further, Hazel/Peterson's willingness to advance a percentage of its share of monies for two of the four lanes of Braddock Road extended and the County's inclusion of the additional funds in the upcoming bond referendum gave the citizens the comfort level they needed on this issue.

The art of compromise was successful in this particular instance in that all parties involved attained all or the majority of what they initially were seeking. Hazel/Peterson received full support of the community, staff, and decision-makers for their rezoning application, which included the commercial element. The citizens were successful in gaining the approval of a much needed community shopping center without jeopardizing or delaying the construction of a vital portion of the overall transportation network for the Centreville area. The local politicians were praised by the community for their "efforts to bring the applicant and community together, to not just resolve differences but to produce solutions" (Frank, 1988).

After one year of negotiations in the rezoning process, the property was rezoned by the Board of Supervisors to the PDH-4 and C-6 Districts to permit the development of the Union Mills project as we know it today. The development of the property was proffered to be in conformance with the approved CDP/FDP and GDP as shown in Figures 40.13 and Figure 40.14 respectively.

The proffers included the following:

- Dedication of the right of way and construction of the Union Mill Road extension
- Dedication of the floodplain/EQC to Fairfax County
- Funding and installation of a traffic signal at Union Mill Road extended and Braddock Road realigned
- Conformance to the limits of clearing and grading as depicted on the CDP/FDP and GDP
- Noise attenuation measures to mitigate noise from Lee Highway, Route 29
- Construction of a trail system along the roadways and the floodplain and within the development
- Recreational amenity package to include a swimming pool, bathhouse, and tot lot
- Hours of operation and the maximum number of students enrolled in the child care center
- Best management practices to be incorporated into the stormwater management plan for the project
- Notification to Fairfax County prior to construction to allow exploration of site for archaeological purposes to remove historic artifacts if found

- Submission of a traffic analysis prior to site plan approval of the entrances to the retail center to determine requirement for turn lanes
- Compatible architecture for the retail center and freestanding uses. The architectural elevations/perspectives of these uses as well as their dumpster facilities were to be submitted to the Planning Commission for review prior to final site plan approval.
- Restriction of various uses within the retail center

Since the freestanding uses were the subject of special exception applications, development conditions were imposed by the Board of Supervisors rather than proffers. The development conditions contained limitations to the hours of operation, number of employees, signs, and landscaping for all the uses.

The entire planning/zoning process of Union Mills approximated seven years from the time the original rezoning application was filed in 1981 to the approval of the final rezoning application in 1988.

### Conclusion to Feasibility and Site Analysis

As stated in the introduction, this case study was not intended to be a "how to" text on resolving conflict associated with the planning/rezoning process. These processes are evolutionary by their very nature and of a complexity that does not lend itself to textbook formula resolution.

Although this case study centered around a project in Fairfax County, Virginia, the process, participants, politics, and problems are similar in jurisdictions throughout the nation. Our world is getting smaller as the population increases and developable land decreases. Consequently, the land development process has gained importance and requires sensitivity to a myriad of interest groups and issues that mandate creativity in design and engineering solutions and, most of all, the ability to communicate effectively.

The landowners, developers, and consultants must be keenly aware of the need for compatibility of differing land uses and design projects accordingly. The development team must be open to negotiations and compromise to resolve competing and/or conflicting interests. Politicians and policy makers must recognize and acknowledge that we live in an imperfect world and it is not possible to please all of the people all of the time. Citizens must continue to be vigilant in protecting their interests while understanding that there are certain property rights inherent in ownership.

The coordination and compromise between these interest groups and the implementation of good planning and engineering principals can result in a quality development. As was the case with Union Mills, following seven years of planning, engineering, negotiations, and compromise, a community was developed that provided quality housing for new residents with easy access to a community shopping center, implemented the County's transportation and

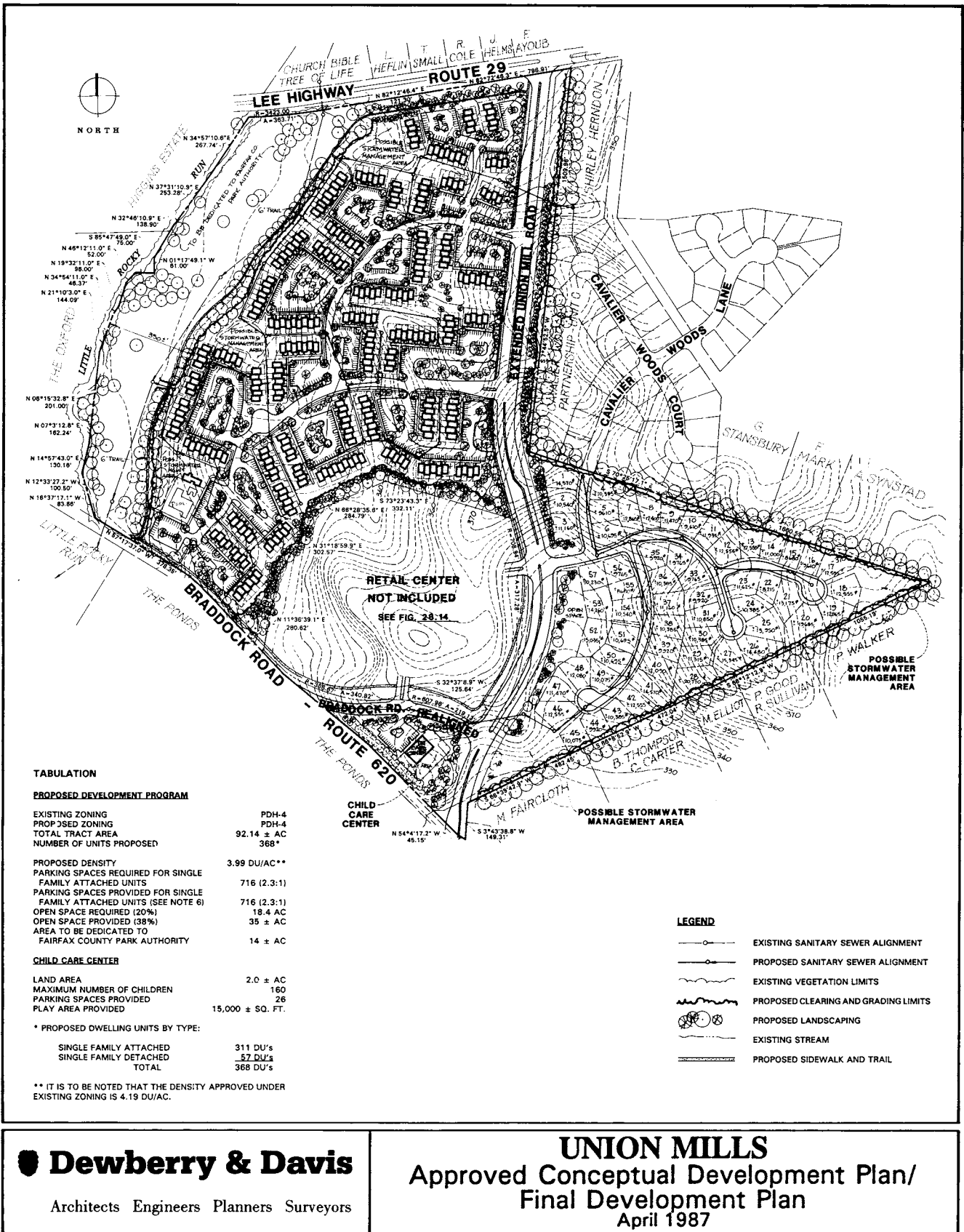
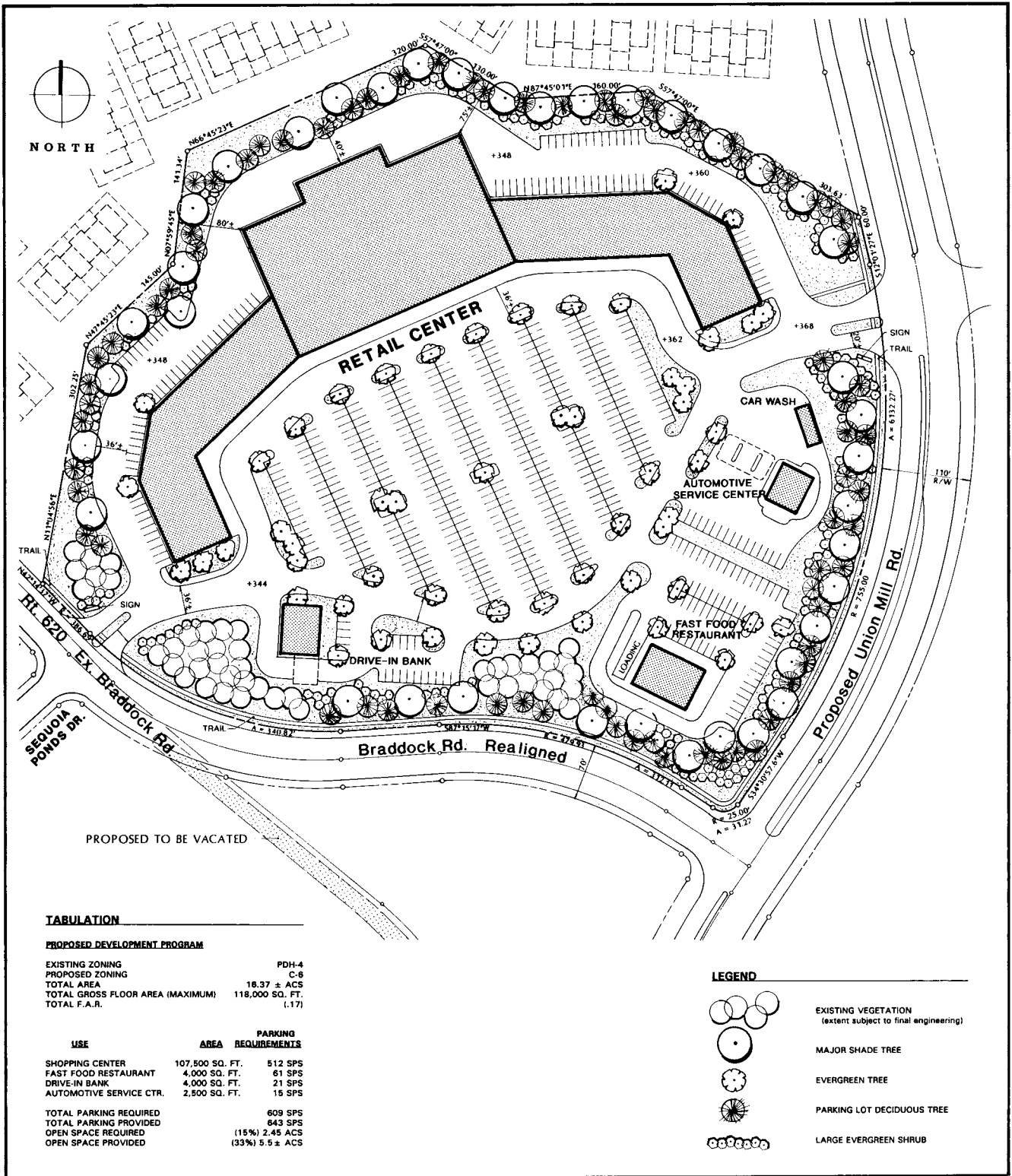


FIGURE 40.13 Approved Conceptual Development Plan/Final Development Plan (residential area).



<p><b>Dewberry &amp; Davis</b> Architects Engineers Planners Surveyors</p>	<p><b>UNION MILLS</b> Approved Generalized Development Plan April 1987</p>
--	--

FIGURE 40.14 Approved Generalized Development Plan (retail center).

land use plans, and met the expectations of adjacent property owners and community activists.

### Epilogue: Political Retrospect

Now that Union Mills is a completed, vibrant community, it is of interest to gain a perspective from both the politicians and developer who were so intimately involved in the planning and zoning process for this project. Both these entities offered their insights and reactions to the politics of land use as it applied to the Union Mills development.<sup>4</sup>

Supervisor Michael R. Frey is currently the elected District Supervisor of the area in which Union Mills is located. Throughout the planning and zoning process of the project, Supervisor Frey was the Senior Administrative Aide to Elaine McConnell, who was, at that time, the District Supervisor. As Administrative Aide to the Supervisor, Mr. Frey was very much involved in all of the land use applications within the District. In this role, he acted on behalf of the Supervisor and had a significant amount of influence on the planning and zoning process. An interview was conducted with Supervisor Frey in an attempt to gain insight into the motivation and political considerations surrounding the Union Mills project.

As discussed throughout this chapter, the most controversial aspects of the rezoning application related to transportation, specifically the extension of Union Mill Road and the timing of the construction of New Braddock Road. Supervisor Frey's understanding of the base of this controversy is as follows (Frey, 1993):

It [the Comprehensive Plan] didn't show Union [Road] Mills as being extended. While it made perfect sense to extend Union Mill Road, the fact that it wasn't on the [Comprehensive] plan raised the suspicions of some of the anti-development activists involved, "By God, if Til Hazel wants it, it's no good. We don't care what it is." If Til Hazel wanted to leave all of western Fairfax undeveloped, those folks would have wanted asphalt!

One of the other pressures and one of the things that created the other base of support for the application was the fact that Centreville High School was coming on-line. It [the high school] had no way to get in or out. You had a horrible intersection of Union Mill [Road] and Braddock [Road]. That was the package they [Hazel-Peterson] proposed. "We'll build the shopping center, staff's identified it as being needed. We'll put it on our property. We'll rezone, we open up Centreville Green to the process again, everybody gets their new shot at it. We'll agree to some phasing and, if you do it quick, we'll build Union Mill Road before Centreville High School opens."

Through the plan process, there was a belief on the part of some of the citizens that was unshakable, that there would be a lot of traffic coming from the west to the shop-

ping center, and somehow they had to have access from the west . . . So they [the citizens] wanted New Braddock Road built before the shopping center opened.

As highlighted previously in this chapter, the citizen groups are sophisticated, very active, and influential in the planning and zoning process in Fairfax County. In discussing the role of the Western Fairfax County Citizens Association in this project, the question was posed as to whether the Supervisor influences or guides the committee in their decisions or whether they function as an independent advisory body. Supervisor Frey responded (Frey, 1993).

I've met with them on a couple of projects. Obviously, you always like to have citizen support. I do my best to explain and to persuade, but I don't ever want to be perceived as controlling . . . I think it works best that way. If they're [WFCCA] independent, if their credibility comes from the belief by the Board or the staff and the other citizens in western Fairfax that they speak for the citizens in the communities here and that's their only agenda. Their credibility stems from the belief on the part of the Board that they are speaking for the citizens and not themselves. Other Districts have various committees where the Supervisor appoints the land use committee or the Planning Commissioner appoints a land use committee. If I appointed members of the land use committee, it is my land use committee.

In response to a question about what he would do if there was an issue or rezoning which his views conflicted with the recommendation of the WFCCA, Supervisor Frey stated (Frey, 1993):

There have been situations where there have been conflicts, for instance between WFCCA and a smaller individual group . . . You have to look at the issues and look at the impact. Are the real larger community issues the precedence and that kind of thing? Or do we need to flip to [the] local group? Or is this something that is really of a benefit for the local community? Sometimes you have to look at both sides and make a choice.

Although, admittedly, the planning and zoning process relating to Union Mills was quite lengthy, Supervisor Frey agreed that the end result was a quality development. "I think it's a beautiful project. There's no question that the road network has been a tremendous benefit to the community" (Frey, 1993).

While the Supervisor's perspective of the project's success is based somewhat on public benefit derived from the project and citizen support, the owner/developer's view is more focused on land rights and overall financial success of the project. Jeff Saxe, Senior Vice President of Planning for the developer, Hazel-Peterson, was directly involved in all aspects of the Union Mills project and was willing to share some of his thoughts and insights relating to the project and process.

<sup>4</sup> Direct quotations from interviews held have been changed for grammatical purposes only.

It is Mr. Saxe's thinking that "In retrospect, the whole zoning and master plan process was very lengthy and adversarial and it shouldn't have been. It was unfortunate that we spent so much time on a project by redoing, I don't know the solution" (Saxe, 1992/93).

Further, when asked whether he thought the citizens in Fairfax County have too much power over the process, he responded, "Yeah, you know I wish they didn't but they do and they probably always would in any kind of jurisdiction like [Fairfax]. I don't have the solution but I don't think the end result in all the adversarial discussions changed anything particularly" (Saxe, 1992/93).

In response to a question as to whether the delays, caused by citizen influence, resulted in a loss of profit, Mr. Saxe stated (Saxe, 1992/93):

Well, I couldn't say if we lost money or not. But I think . . . why it paid for us to go through the zoning process is throughout the [19]80's the value of land seemed the longer you owned it, the more it was worth. So it was sort of paying us to wait. Unfortunately, sometime in that process, the value of a townhouse lot started to go back down. Pending that extra couple of years we actually ended up selling the townhouses, I don't mean that we lost money, but we didn't make as much as we would have two years earlier.

According to Mr. Saxe, "One thing that I wish we could have done was to put the retail center on Lee Highway, but that was contrary to the Comprehensive Plan. Since we had to put units up there, those units are difficult to sell" (Saxe, 1992/93).

Mr. Saxe agrees that the timing of the construction of New Braddock Road was the primary citizen issue and the one that caused significant delays in the project. Eventually, a bond issue was passed that allowed the construction of the four-lane facility in time for the opening of the Centreville High School. The resolution of this issue was key to all parties, and Mr. Saxe made the following observation (Saxe, 1992/93):

This was one of the big successes, and key citizens are to be complimented for that. They put the pressure on and got the bond money and got the entire road built. Both sides probably went at it too hard. We both would have been better off if we could have cooperated a year earlier and not wasted all that time.

## PART II: CONCEPTUAL DESIGN

It should be understood that this case study represents problem-solving approaches used on a specific project. These solutions are a result of a wide variety of influences and variables and should not be viewed as the only appropriate manner in which to address the specific engineering issues discussed herein. Each project, client, or design professional is different, and therefore there are an infinite number of engineering constraints/issues and corresponding solutions. Further, this case study does not discuss design specifics, which are addressed in detail in the previous

chapters of this Handbook. This chapter shares lessons learned from the Union Mills project. Each project represents a learning process for all involved and that certainly was the case with the development of this community.

The Site Analysis and Feasibility section of the Union Mills project illustrated the planning and zoning process from the initial planning/feasibility stages to the approval of the Conceptual Development Plan (CDP), Final Development Plan (FDP), and Generalized Development Plan (GDP) by the Fairfax County Planning Commission and Board of Supervisors. This zoning approval established the overall framework and pattern for development of the project. Further, this approval permitted the owner/developer of the project, Hazel/Peterson Companies, the rights to construct 368 residential units consisting of 311 townhouses and 57 single-family residences, a swimming pool recreational facility, a day care center, and a 122,000 ft<sup>2</sup> neighborhood shopping center with associated uses. The breakdown of the residential component (shown in Figure 40.15) of this project is as follows:

Section 1: 66 townhomes

Section 2: 71 townhomes

Section 3: 71 townhomes

Section 4: 65 townhomes

Section 5: 38 townhomes

Section 6: 57 single-family lots

In conjunction with this rezoning, Fairfax County required certain conditions, including roadway improvements, assurances that water quality would be protected, stormwater management facilities, protection and preservation of the Environment Quality Corridor, specific recreational facilities, and limitation on the amount of retail space. These commitments and restrictions were volunteered by the developer in the form of proffers (see Chapter 2). The FDP for the residential component of the project and the GDP for the commercial/retail component are the plans to provide the graphic layout of streets, lots, uses, parking areas, public facilities, landscape, setbacks from adjoining properties, and other requirements established during the zoning process. Together, these documents outline the general framework for the engineering design of the project.

### Development Patterns and Principles

#### Preliminary Engineering and Design.

Although a considerable amount of preliminary engineering is conducted during the rezoning process, it is unrealistic to think that all engineering constraints and issues can be foreseen and properly addressed during that stage of the development process. During the rezoning of the Union Mills project, preliminary engineering design was undertaken and many potential issues identified and resolved to a comfort level

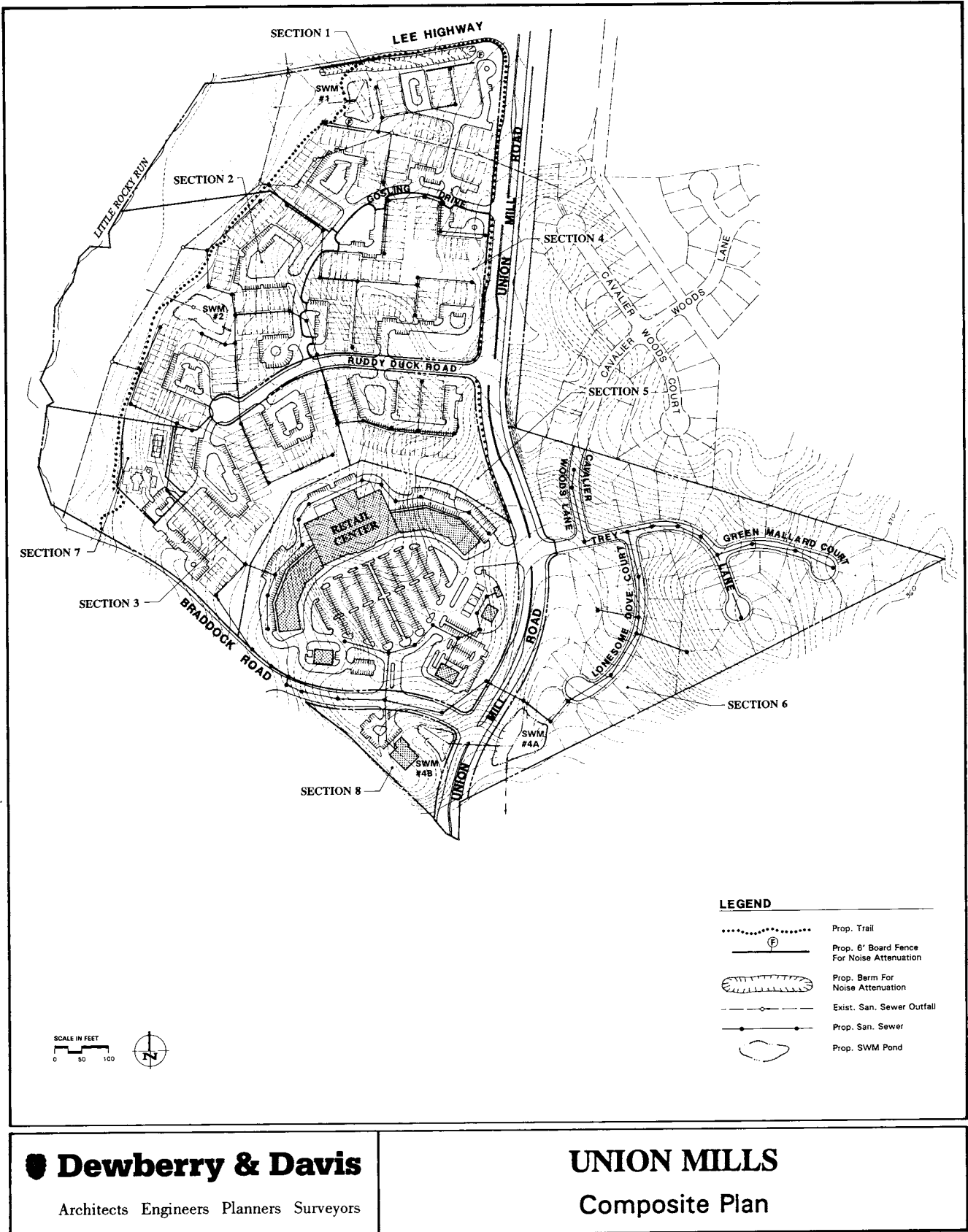


FIGURE 40.15 Composite plan: Union Mills.



sufficient to proceed with the final engineering phase at the appropriate time.

Successful engineering design is dependent upon a number of factors, including a firm understanding of the client's goals and time constraints; effective communication among the client, planning consultants, and the engineering project manager; and comprehensive site data collection and evaluation. A detailed field investigation to further the knowledge of the physical characteristics of the site will help identify the development constraints and opportunities.

*Client Input.* Since the very beginning of the rezoning process, the client had been establishing goals, formulating schedules, and providing overall guidance to the development team. At the preliminary engineering design phase, the client must give additional guidance when more refined data are available and new issues surface. Although many of these "new" issues were preliminarily identified during the rezoning process, the detail of the updated information further refines the framework in which to pursue a viable solution. Any viable solution must be made with the overall timing, phasing, and cost of the project in mind. It is incumbent upon the design engineer not only to identify potential problems, but also to provide the client with all solution options as quickly as possible after they are identified to aid the client in the decision-making process. Although the client is the ultimate decision-maker, the team members provide the data required the client to make a sound business decision.

*Effective Communications—Project Handoff.* It is essential that all team members be aware of the nuances of a given project. In the case of the Union Mills project, a series of handoff meetings was conducted by those participating in the zoning action to brief the engineering element of the team on history and evolution of the project to date as well as the status of the zoning effort. Often, during the rezoning process, there are many issues that cannot be resolved and therefore must be discussed by all members at the time to foster a clear understanding on the background and proposed resolution. It is the planner's role to explain the proffers, graphics, and plans generated during the rezoning process to the engineering professionals. The handoff meeting is a tool to identify potential problems and establish optional approaches to resolving such problems.

During the Union Mills handoff meetings, several issues of importance surfaced. For instance, the developer did not intend to construct a service drive along Lee Highway even though the service drive was required by VDOT. It was determined that a waiver request for processing through the Board of Supervisors should be prepared and submitted as quickly as possible. This was done and ultimately approved by the Board of Supervisors with no delay of the project.

Also as a result of the handoff meetings, the issue of stormwater management and related Best Management Practices (BMPs) surfaced. Specifically, Fairfax County was in the process of establishing revised design criteria for BMP facilities. There was a great deal of concern that the BMP

facilities shown on the Final Development Plan would be insufficient to satisfy the new design criteria. Given this uncertainty, the decision was made to proceed as quickly as possible with the preliminary design of the stormwater management/BMP facilities using the older criteria to assess the potential impact of the site layout and be prepared to upgrade the facilities as per new criteria if necessary.

Another immediate benefit of the handoff meeting was the decision to proceed with detailed field run topography along the floodplain to supplement the aerial topography. During the rezoning process, the preliminary design of the project was based on County-provided topography at 5-ft contour intervals, which is sufficient for the preparation of the zoning document. However, the planner and client were very much aware of the relatively flat topography adjacent to the floodplain and were concerned that inaccuracies in the topography could impact the number of proposed units as well as stormwater/BMP facilities. Consequently, the decision was made to spend the money for the field run topography to avoid potential problems later.

The use of the handoff process to communicate effectively the subtleties of the rezoning documents serves as an early warning system for the engineering component of the team. Ongoing dialogue between all members of the team throughout the development process is essential to the success of a given project. This ongoing dialogue certainly added to the success of the Union Mills development.

*Data Collection/Evolution.* Data collection, updating, and evolution are integral in preparation of the final engineering drawings. The significant amount of time required for the plan amendment and rezoning of the Union Mills site necessitated updating the data collected for that process. The engineering team reviewed all the information that was part of the rezoning and had to determine what was usable and what data required updating.

In the case of the Union Mills project, the previously prepared boundary survey was revised to reflect new easements and encumbrances identified in a new title report. An aerial topographic survey was obtained and all the utility companies were requested to provide the most current information on the location of utilities on and around the Union Mills site. Floodplain information along Little Rocky Run was obtained from the U.S. Geological Survey (USGS).

This technical information, coupled with the zoning documents, including the proffers, becomes the basis used for the preparation of the preliminary plat and final site plan. Additional issues surfaced in compiling the new data. Examples of such issues relating to Union Mills are as follows:

- Analysis of the aerial topography revealed that gaps were present where dense forestation existed. Field run topographic survey was required to fill these gaps.
- Once the topographic survey was completed, the USGS floodplain information was plotted on the topographic map. The results showed that the USGS flood-

plain was 10 to 100 feet wider than the floodplain shown on the Final Development Plan (FDP). This finding would require a revision to the FDP.

- The preliminary engineering studies of the stormwater management/BMP facilities were reviewed for compliance with the new design standards established by the County. As a result, the new, more restrictive, design standards, coupled with more accurate floodplain information, would require a new preliminary engineering design.
- The proffers were the subject of a detailed analysis to identify those that would have an impact on engineering design and phasing implications, including:
  - Dedication and construction of major roadways
  - Incorporation of stormwater management/BMP plans
  - Traffic analysis to determine where turn lanes would be required
  - Noise attenuation measures adjacent to the major thoroughfares
  - Recreational amenity package
  - Timing and construction of the trail system within the project

Following the compilation and analysis of the updated data, a revised layout was produced to accommodate the additional constraints resulting from the newly acquired data. The new layout mandated the relocation of five lots to other areas of the project. In view of the fact that these changes to the FDP were a result of engineering consideration, a request for an administrative approval was submitted rather than a Final Development Plan Amendment being applied for through the formal public process. The FDP/proffer interpretation process is discussed in more detail below.

**The Preliminary Plan.** Generally, Fairfax County requires that the preliminary plan be submitted after approval of the rezoning case by the Board of Supervisors and that the preliminary plan be approved prior to the submittal of site plans and/or subdivision plans. The preliminary plan provides basic information such as road and lot layouts, defines open space areas, identifies preliminary utility layouts for sanitary sewer, water, and major storm sewer systems, locates and sizes stormwater management/BMP ponds, and identifies limits of clearing and grading as well as siltation/erosion control measures and density and open space computations.

The purpose of the preliminary plan is threefold: the plan (1) represents the owner/developer's intent to subdivide; (2) demonstrates how the subdivision will comply with the zoning ordinance as well as the FDP and proffers; and (3) allows the various agencies to review and identify

zoning and engineering design issues that must be addressed and resolved prior to the submission and approval of construction plans. As such, it establishes the framework for the County to recommend changes to the project in order to be sure that the project complies with design requirements prior to final design.

Given the lengthy and complicated zoning process experienced by the Union Mills project, the developer took advantage of an approval process known in Fairfax County as "concurrent processing." Specifically, with approval by the Board of Supervisors, the preliminary plan can be submitted and reviewed concurrent with a rezoning action, which in the case of Union Mills was the FDP. The risk to the developer with this process is that changes resulting in the rezoning document mandate mirror changes in the preliminary plan. The obvious advantage of concurrent processing is immediate approval of the preliminary plan upon approval of the zoning document. These time savings can, in some instances, equate to significant dollar savings because they shorten the time a developer is required to carry interest on the property before selling it to a builder, thus significantly discounting the engineering costs for revising the preliminary plan.

**Project Phasing.** During January 1990, the rezoning process was completed, as were the preparation and submission of the preliminary plan. Upon the completion of the preliminary engineering process, the client was in a position to proceed with the final engineering design phase of the development process. At this point, key issues now driving the project included the selection of the project engineering team, project phasing, and plan submission schedule.

In an effort to maintain efficiency on this project, a project team was selected consisting of the project manager, design engineers, and computer-aided drafting and design (CADD) technicians and draftspersons. Importantly, the persons selected for the team had been associated with the project during the rezoning and preliminary plan stages and also had prior experience with the client. The manner in which this team was selected and the experience of the individuals involved reduced the time normally attributed to becoming knowledgeable about a new project and/or client.

Establishing a plan submission schedule to DEM as part of the client's expediting process required a coordinated effort between the client and DEM. Specifically, the client was able to obtain a commitment on the part of DEM for plan approval in early 1991, provided the plans were submitted by March or early April 1990. Essentially, this agreement reduced the normal processing time for such plans by between 50 and 65%. This short turnover required that the engineering team design, draft, and submit the final engineered plans within a six- to eight-week time frame.

The proposed sequence of construction is known as phasing. In many cases the land development engineer believes that this is the sole responsibility of the client or developer. While it is true that the developer should have

the best knowledge of his economic situation and his marketing plan, the engineer can have a significant impact in the success of the project through input on phasing of construction. For example, it is important to understand the cost involved in moving earth between sections of a given project. When that earth is moved several times due to poor phasing, the resultant cost can be exorbitant, with no resultant benefit to the project or the client. The benefits are obvious when the project phasing is done such that fulfilling earthwork requirements in one area can be accomplished by the sequential excavation of earth at an adjacent area. The engineer can provide invaluable information on phasing by way of earthwork takeoffs.

In addition, as discussed further below, the engineer working with the client can have a positive impact in organizing land parcels into plan sets to reduce bonding costs and/or spread these costs better over the life of the project. It must be understood that typically the income stream does not begin for the client until the first lot or house is sold. Consequently, phasing in terms of early bond release should be a factor in the establishment of the overall project schedule. Several examples of phasing in conjunction with specific plan development are presented below.

*Ruddy Duck Road.* As can be seen in Figure 40.15, Ruddy Duck Road is a primary access road to several townhouse sections of the development. Early in the engineering design process, it was determined that this road should be submitted as an independent plan rather than be incorporated into the townhouse plans. Separating the road from the townhouse plans afforded maximum flexibility to the sequencing of the townhouse plans. In addition, submitting the road as an independent plan allowed road construction to be completed, and the correspondingly early bond release would not encumber the client's cash reserve for subsequent construction in the townhouse sections.

*Sanitary Sewer Outfall.* The sanitary sewer outfall alignment is shown in Figure 40.15. It is noted that in the Union Mills project the 2300-ft sewer line that serves the residential area as well as the retail center was prepared and processed as a separate plan. Again, with phasing as a primary consideration, the decision to file this on a separate plan was made in consideration of the fact that the retail center area would require a significant amount of fill material and the timing of the construction of the retail center was uncertain. Further, it did not make good sense to include the sanitary sewer plans with the design of Braddock Road or Union Mill Drive since inevitable changes to the design of these roads would delay construction of the sanitary sewer outfall. Designing this sanitary sewer line as a separate plan ensured that construction of the townhouses serviced by the line, all of the retail center and the single-family detached lots as well as a proposed child care center in Section 8 of the project would not be delayed. The flexibility here was that construction of any or all of these sites could commence at the discretion of the client.

*Townhouse Sections.* In consultation with the client, the engineer developed five sections of townhouses. The section boundaries were determined based upon utility/infrastructure

availability and open space/density requirements. In Fairfax County, each section must meet open space and density requirements on a cumulative basis. Therefore, the engineer first computed open space and density for the overall site to ensure the project met the requirements determined by the zoning process. Section boundaries were then adjusted to ensure that sections would, on a cumulative basis, meet the requirements. The townhouse sections were phased based upon marketing considerations, the amount of infrastructure needed to support the section, proximity to existing utility tie-ins, and construction costs.

Although five sections of townhouses (instead of only one or two sections) would be more costly to the client from an engineering perspective, the advantage of smaller performance bonds coupled with being able to complete construction sooner enabled the client to market and sell the smaller sections independently, thereby maintaining a positive cash flow.

As discussed later, each builder who bought a townhouse section incorporated his own type of townhouse, which required revisions to the grading plan. Having smaller and separate townhouse sections later proved to be a time- and cost-saving method for each builder. Table 40.1 lists the townhouse sections, summarizing the reasoning used to phase the section into sequence.

Section 6, the single-family section composed of 57 lots, would be the last residential section to be constructed due to its heavy dependence on road, stormwater management, and utility construction extensions.

### PART III: SCHEMATIC DESIGN

#### Preliminary Engineering

The following discussion highlights many of the elements of the schematic design process, identifies a myriad of engineering issues encountered during design of the Union Mills project, begins to explain how they were resolved. This is not an attempt to analyze each plan in terms of preparation and design, but rather a discussion on the major considerations that need to be appropriately addressed during the final engineering design process.

Early in the schematic design process, it was decided that the major infrastructure components of the project would be the subject of separate plans. This approach not only provided the developer with the maximum flexibility with respect to satisfying the needs of yet-to-be identified builders, it permitted the bonding of these improvements only when needed. Consequently, separate plans were prepared for the following infrastructure components of the project:

- Union Mill Road/Lee Highway Improvements
- Ruddy Duck Road
- Braddock Road widening
- Stormwater Management/BMP Ponds 1, 2, 3, and 4

**TABLE 40.1 Phasing Considerations for the Union Mills Townhouse Sections**

SECTION NUMBER	NUMBER OF UNITS	PHASING CONSIDERATIONS
1	66	<ul style="list-style-type: none"> <li>● Visibility to Lee Highway (Marketing)</li> <li>● Proximity to open space along Little Rocky Run (Marketing)</li> <li>● Proximity to existing 12" sanitary sewer</li> <li>● Required the minimum amount of road construction (Union Mill Road)</li> <li>● Only 1 stormwater management pond required</li> </ul>
2	71	<ul style="list-style-type: none"> <li>● Sited along major open space (Little Rocky Run)</li> <li>● Proximity to existing 12" sanitary sewer</li> <li>● Flexibility to access section from either Section 1 or Ruddy Duck Road</li> </ul>
3	71	<ul style="list-style-type: none"> <li>● Access from Ruddy Duck Road</li> <li>● Availability of existing sanitary sewer</li> <li>● Proximity to recreational facility (swimming pool)</li> <li>● Sited along major open space (Little Rocky Run)</li> </ul>
4	65	<ul style="list-style-type: none"> <li>● Access from Section 1 and Ruddy Duck Road</li> <li>● Visibility from Union Mill Road (once constructed)</li> <li>● Depending on the construction of Sections 1 and 2 for utility tie-ins &amp; access</li> </ul>
5	38	<ul style="list-style-type: none"> <li>● Required utility tie-ins from Sections 3 and 4</li> <li>● Proximity to retail center seen as least desirable for marketing</li> </ul>

- Sanitary Sewer Outfall Plan
- Braddock Road/Union Mill Road intersection improvements

Also, in the early phase of preliminary engineering, it was determined that a geotechnical analysis would be conducted to ascertain the quality of soil. Once the quality of the soil is determined, decisions can be made in terms of the earth take-off quantities of the site, that is, the quantity and location of the dirt that can be used as structural fill and the amount of spoil dirt. Due to the enormous cost of importing or exporting dirt to a site, particularly on a project the size of Union Mills, it is critical to determine the specifics of an earth balance early in the process. Typically, having dirt within the site costs approximately \$2.00 to \$3.00 per cubic yard. Importing or exporting dirt to or from the site may increase the cost to \$8.00 to \$12.00 per cubic yard. The results of the soil study were more favorable than initially anticipated, and dirt balance of the project was accomplished absent the need to import or export material.

### Environmental Considerations

**Infrastructure Elements/Off-site Impacts.** From the very earliest planning design stages of a given project, off-site impacts are a serious concern. Off-site impacts are those situations where improvements, grading, utility extensions, and other activities are required for a project but are on

property which the developer does not control. Such improvements can result in the need to acquire costly rights of way or obtain construction easements from unwilling participants. Good engineering practices dictate the need to avoid off-site improvements whenever possible or, at the very least, attempt to get owner approval for the land or easement prior to making a commitment to provide the improvement. The process of gaining approval to construct offsite improvements can be lengthy, expensive, and result in significant delays in the project. Potential off-site impacts were of critical importance to the Union Mills project, particularly with respect to the Lee Highway, Union Mill Road, and Braddock Road improvements, which are discussed below.

*Lee Highway.* The improvements required on Lee Highway were a major design and construction element of the Union Mills project. The engineering staff worked with a traffic engineering consultant to analyze the turning movement volumes at the intersection of Lee Highway at Union Mill Road. The need for reliable estimated traffic data was considered essential since Union Mill Road was going to be a major collector roadway.

A primary issue was whether dual left turn lanes were warranted and/or would be required from Lee Highway onto Union Mill Road. It became evident that lane shifts were going to be required immediately east of the project, at the intersection of Lee Highway and Stringfellow Road

(see Figure 40.16). Of critical concern, therefore, was whether there was sufficient linear distance to provide the storage lane length required without dual turn lanes. There was also concern with the grade differential between the eastbound and westbound lanes of Lee Highway.

A review of the as-built plans for Lee Highway indicated that there was a considerable amount of both underground and overhead utilities within the Lee Highway right of way. There were overhead trunk electric lines, a major 24-in. waterline, a major telephone duct bank, and other utilities. The engineering design in this area required careful data gathering and accurate plotting of the existing infrastructure, involving extensive coordination with all of the affected utility agencies. Given the location of the existing utilities, it was determined that the area needed to provide the improvements was greatly reduced. In addition, the soils reports indicated that a considerable amount of the backfill material over the existing waterline trench was soft and of poor quality. The contractor would have to remove and replace a major portion of the soil as well as recompact it as part of the offsite improvement requirements.

As mentioned earlier, there was also a high degree of sensitivity with respect to the possible requirements for off-site easements and rights of way. Given the potential need to construct dual left turn lanes, there was a possibility that major off-site easements and/or rights-of-way would be necessary both north and east of the property. In addition, dual left turn lanes would cause the improvements to extend over a larger area further to the east. There was also concern about an existing Lee Highway culvert crossing of Little Rocky Run (shown on Figure 40.16, at the extreme western edge of the drawing) and potential retrofitting of the culvert. It was determined that the culvert was inadequate and that the extensions that would be needed could be extremely costly and had major off-site implications, including the need for floodplain easements. It was the design team's opinion that the impacts of the Union Mills project did not warrant improvement to the culvert. Consequently, the road design was structured so as not to impact the box culvert. Reconstruction of that facility would have cost approximately \$250,000. Thus, the early identification of the problem and the design solution resulted in significant cost savings to the project.

An additional design element that resulted in major cost savings was related to bifurcation between the eastbound and westbound lanes. Specifically, the design of the left turn lane had to account for problems associated with the existing cross-slope of the pavement as well as maintaining traffic flow during construction. The design team was concerned that the state transportation department would not allow major disruption of traffic on this heavily traveled roadway. Further, offsite traffic diversion facilities would be expensive and could require the need for additional easement acquisitions. Consequently, the design approach was to use asphaltic overlays to bring the grades of the two directions up to the same elevation. This approach resulted

in asphalt work being carried beyond the immediate frontage area of the project. However, this was considerably more cost-effective than removing the existing roadway and rebuilding it while also maintaining heavy traffic flows. The details of the design are shown in Figure 40.16. As part of this effort, safety at the intersection of Stringfellow Road and Lee Highway was improved due to the horizontal shift in alignment along westbound Lee Highway. This improvement provided superior traffic flow and a greater level of safety without any real impact in construction cost to the developer.

As mentioned earlier, the government's long-range plans for Lee Highway included a service drive adjacent to the Union Mills northern property line. Due to the potential negative impact on the project, a waiver of this requirement was requested and ultimately obtained. Although the county approved the waiver request, the Virginia Department of Transportation (VDOT) conditioned the approval with a requirement of a third eastbound lane in lieu of the service drive. The requirement for the third travel lane came quite late in the process and required some adjustment to the engineering plans. The lesson learned in this instance was that issues with such a potential for negative impact should be identified and resolved as early in the process as possible.

*Union Mill Road.* The preliminary design profile for Union Mill Road had previously been established by the consultant engineer's transportation division as part of its work on the Braddock Road/Union Mill Road intersection improvements. That information was used as the base for the final design of Union Mill Road between Lee Highway and Braddock Road. The design of the roadway was rather constrained due to its proximity to an adjacent subdivision, the Lee Highway tie-in point, as well as its connection to the Braddock Road intersection at the southern end of the project.

Visual and noise impacts on the adjacent subdivision east of Union Mill Road were a consideration. Consequently, the road profile was lowered to the extent possible to minimize this impact. Lowering the road profile also reduced the amount of fill dirt and improved the design of the connections to the entrances to the adjacent townhouses proposed within the project. Since Union Mill Road was the major roadway servicing the project, substantial coordination with the internal site construction elements was needed. This roadway interfaced with three sections of townhomes, the single-family section, and the retail center.

Another design constraint encountered in the design of Union Mills Road was a requirement to provide access to an adjacent parcel at the Lee Highway intersection. The access to this parcel was provided via a service drive connection from Union Mill Road. In addition, poor drainage patterns, utility poles, and existing underground utilities were all located within this area. The design also required the acquisition of off-site grading easements. The developer was unable to obtain the required easements; therefore,

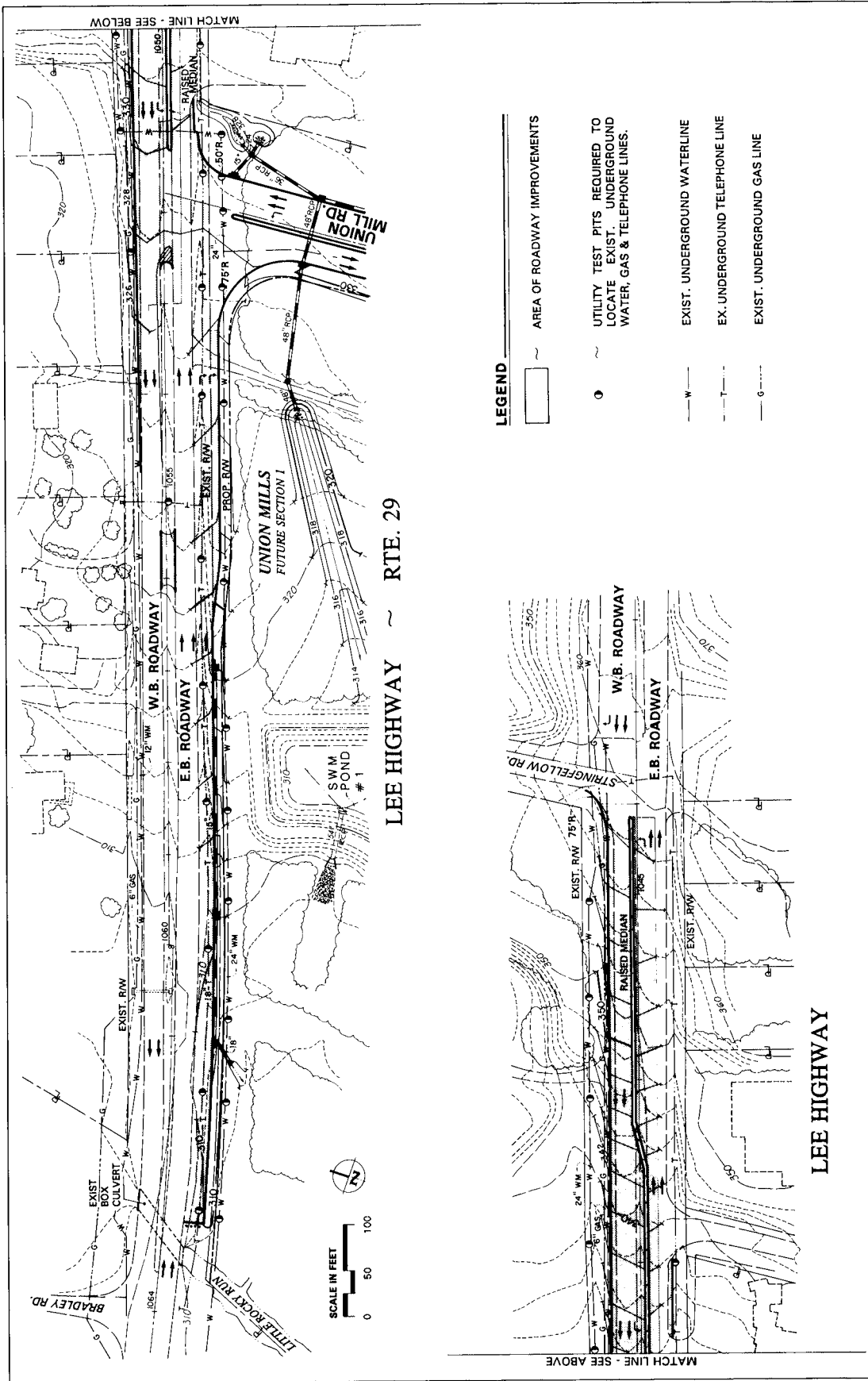


FIGURE 40.16 Plan view for the Lee Highway design.

Fairfax County permitted the developer to delete the connection as a construction requirement. The County did, however, require the developer to post a cash escrow with the County and dedicate sufficient right of way and easements to allow future construction by others.

**Braddock Road Widening.** The widening of a portion of Braddock Road between the retail element and Little Rocky Run on the western edge of the site was required with this project. Figure 40.17 depicts the final design plan for this widening. As shown in Figure 40.17, Little Rocky Run crosses under Braddock Road in four 10 × 8-ft box culverts with the project's property line running down the center of the culvert system. Careful design considerations were necessary to the Braddock Road widening project in proximity to this culvert system to avoid reconstruction of the system, estimated at \$500,000. Further, the soils studies showed that debris under the roadway, in the area of the culverts, rendered the fill unsuitable. The reviewing authority decided that reconstruction of the culverts would not be required, a decision resulting in significant savings to the developer.

Another engineering design effort associated with the Braddock Road widening was the existence of a gas line within portions of the right of way, also shown in Figure 40.17. Note that in some locations this gas line was directly under the proposed curb and gutter and was in an area where the soils study showed that the rock levels were very shallow. Extensive field tests were conducted to locate the gas line accurately both horizontally and vertically. The challenge was to design the storm sewer in such a way as to avoid conflicts with the gas line. Tolerances of only a foot or less were common and storm sewer structures of a special design were installed to avoid having to relocate the gas line. Early problem identification and creative design saved considerable additional construction costs as well as delays if redesign of the gas line was necessary.

While this roadway might appear a minor facility, it had its share of design issues. Beyond the problems discussed above, the design required extensive coordination with the adjacent retail center and with the Braddock Road/Union Mill Road intersection improvements being done by a separate department (Transportation Department). As was the case with the Lee Highway improvements, to make the transition work more effectively for this widening, the use of pavement overlays rather than removing the existing pavement was the approach taken.

**Stormwater Management and Best Management Practices (BMPs).** Upon completion of the site layout and a review for compliance, the next step in the schematic engineering process is to verify the preliminary computations with respect to open space in addition to ensuring that the stormwater management facilities are in compliance with the approved Final Development Plan. Due to the need to revise and relocate some of the stormwater management facilities, a proffer interpretation was requested and resulted in a favorable finding by the Zoning Administrator that the

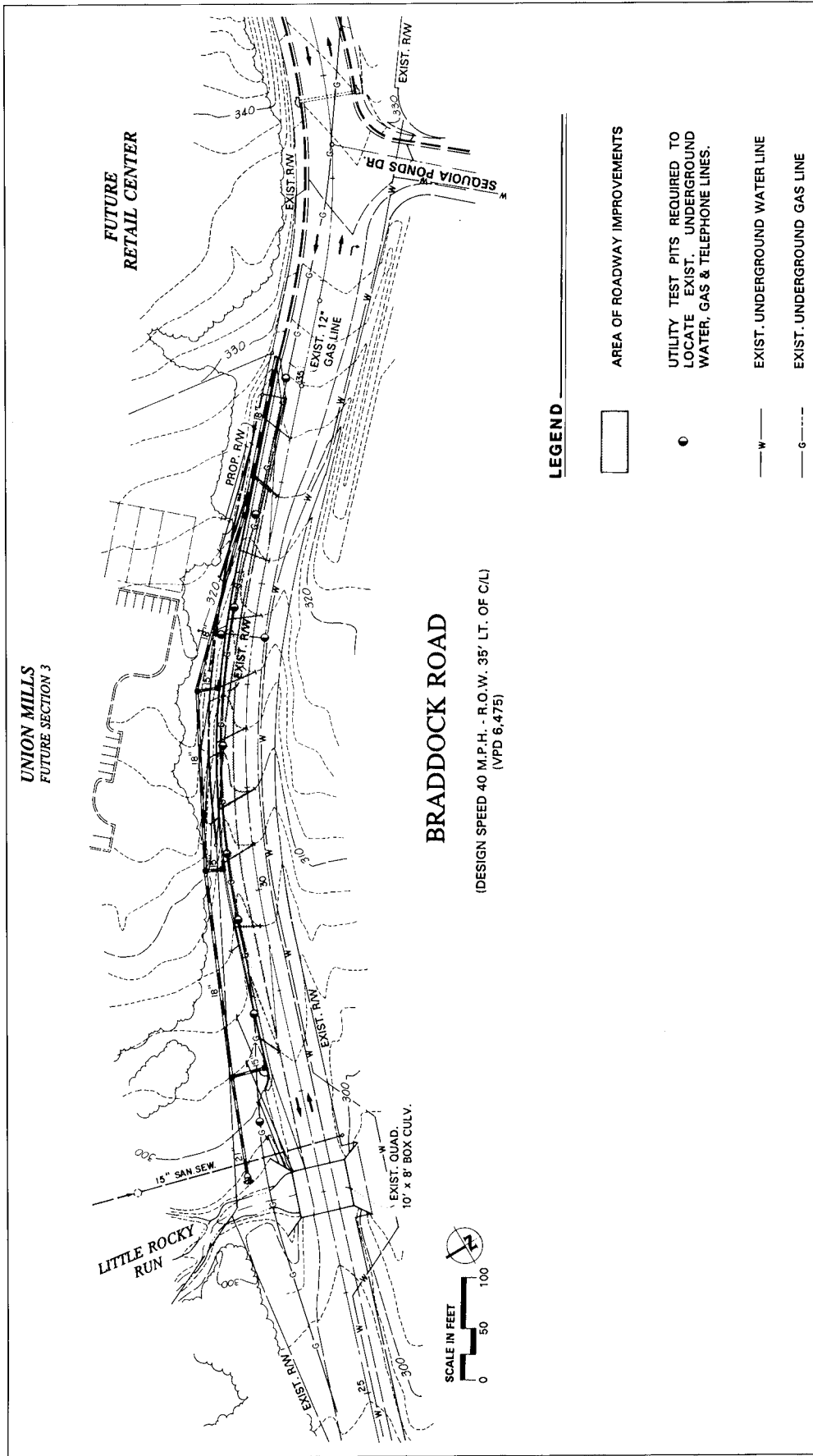
revised SWM facilities were in substantial compliance with the approved FDP. Preliminary stormwater management/best management practices (BMPs) designs were completed at this time. The location of this property within the approved Water-Supply Protection Overlay District (WSPOD) required that the project incorporate BMPs. Unlike stormwater detention, BMPs are techniques to improve water quality, which in Fairfax County requires phosphorus removal efficiency of 50% for the entire project. "Credit" towards such phosphorus removal is based upon a number of criteria, including retention of undisturbed open space, volume controls, placement of ponds in a series, capturing of off-site drainage, and other pond-maximizing criteria. Efficiencies are assigned to each type of phosphorus-removal device, with total efficiencies to be at least 50%.

Originally there were three ponds proposed along Little Rocky Run and one pond in the single-family section. BMP design for this project was difficult because most of the site runoff drained directly into Little Rocky Run, thereby bypassing the ponds.

One of the difficulties in designing the BMP system is that, in contrast to the case with stormwater management, it is difficult to compensate for uncontrolled areas. Therefore, all of the ponds were located as far downstream as possible to maximize the drainage areas to the ponds. Additional efficiencies would be obtained by open space, retention ponds in series, etc., in order to achieve the required 50% phosphorus removal. In addition, the drainage area allocated to each pond was fixed based upon the new site layout; therefore, the pond designer worked very closely with the site design engineers to ensure that sufficient drainage area was provided for each pond. For example, the storm drainage outfalls from the retail center had to be directed into two ponds based upon the fixed drainage area criteria.

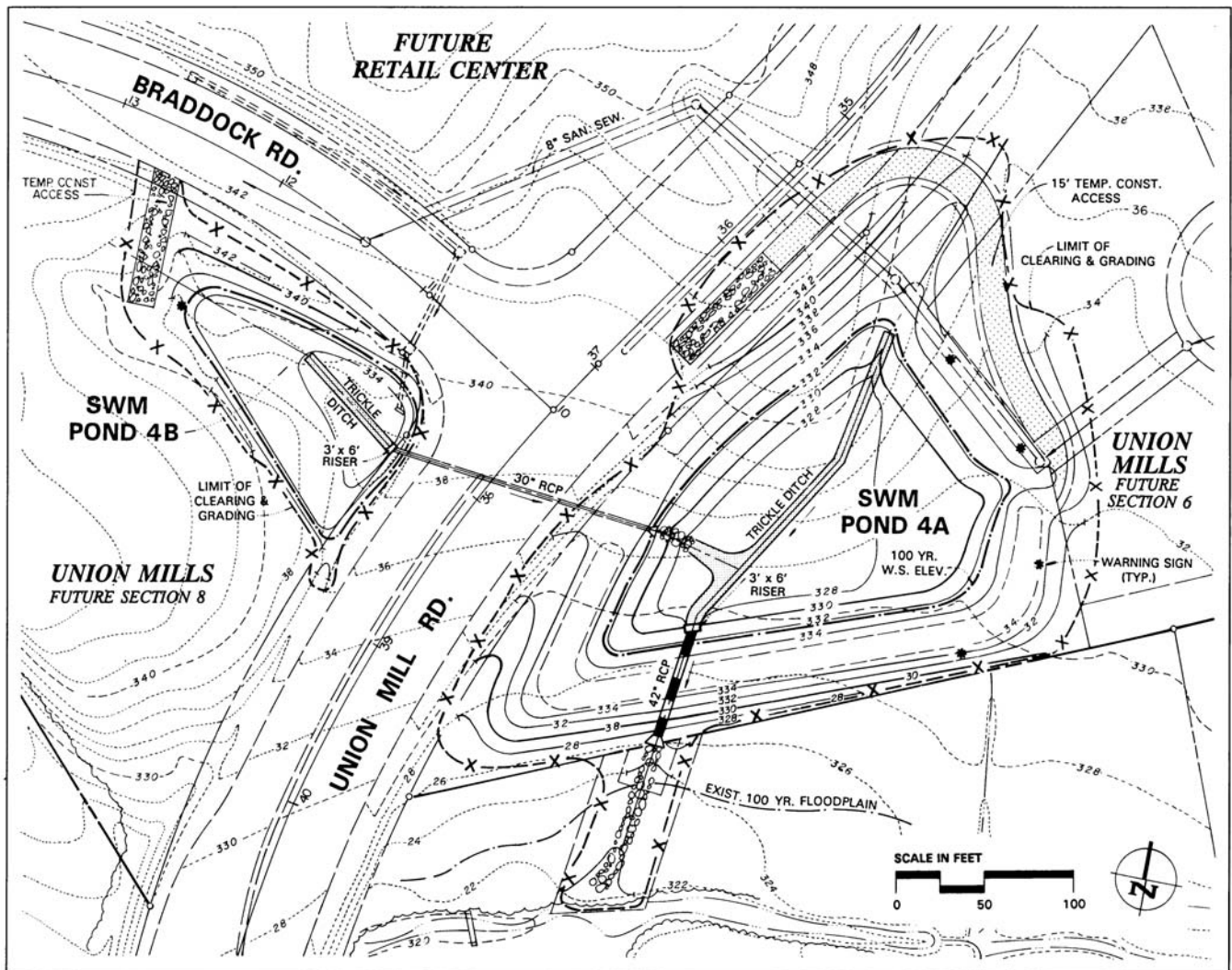
In the design of the ponds, the engineer was able to provide the required efficiency by taking credit for off-site areas, calculating a series pond efficiency in conjunction with a pond immediately adjacent to the project and requiring that undisturbed areas be retained within the project upstream of the ponds.

Pond 4-A, located in the single-family detached (SFD) section (Figure 40.18) proved to be especially difficult to design. Its location, as shown on the FDP, was the best possible place given the constraints: (1) The location was the point closest to the outfall stream; (2) the location thereby minimized the need for extensive offsite easements; and (3) the location provided for the best possible lot yield. Furthermore, this pond had to account for BMP requirements for part of the retail center. Consequently, the pond could not be sized to accommodate the stormwater management requirements and the BMP requirements. The addition of another pond downstream at the day care site coupled with specified undisturbed open space within the SFD enabled this area of the project to meet the BMP and SWM criteria.



**FIGURE 40.17** Plan view for the Braddock Road widening.





**FIGURE 40.18** Stormwater management, ponds 4A and 4B.

Another aspect of the design of the ponds was that each pond had to be fully coordinated with the adjacent townhouses and single-family homes. In view of the fact that some of the dwelling units were sited immediately adjacent to the ponds, considerable coordination with the site designer was required to ensure that utility locations, grades, and the horizontal fit of the lots to the ponds were accomplished. Furthermore, because of the requirement for undisturbed open space within the project, the site designer had to adjust grades and utility locations to ensure the retention of the open space. Several additional observations relating to the design of the stormwater management ponds are in order. Given the topography of the property, all the ponds were ultimately placed on natural slopes rather than in low-lying swales, as is more typical. Further, the natural terrain and proximity of the floodplain required that the ponds be placed on fill material rather than undisturbed soils. Given the fact that townhomes were also placed on fill material above the top of the dam elevations, creative

engineering was required to grade the site, place the utilities, and accomplish the needed earth balance.

## PART IV: FINAL DESIGN

### Overview

The design of a project involves the expertise and experience of a myriad of participants. The conceptual layout and design is a complex undertaking involving a number of steps guided by a wide variety of influences. The planning and zoning process is equally complex, with a number of elements that cannot be quantified, such as political/citizen influence and the many wants and needs of countless other interest groups. As discussed in Part 1 of this chapter, the many design and planning/zoning influences result in a less than perfect development plan that is passed on to the final engineer to make it a reality. Although major strides have been made in engineering technology, the key to a successfully engineered project is the human element—the project engineer.

In addition to the engineering of the infrastructure items previously discussed, the project engineer is responsible for the site design to include the layout of the townhouse units, travel aisles, and parking spaces. In the case of Union Mills, the initial layouts were using AutoCAD (CADD) for all five sections.

The project engineer prepared an overall grading plan, selected unit types, and developed an overall sanitary sewer and storm sewer plan for the project. This initial work was designed to ensure compatibility between the sections and consistency in the design approach for the entire project. The overall design was then disseminated to the design engineers for preparation of the final construction documents. The ability to provide overall plans quickly and cost-effectively to the various engineers working on the section plans resulted in significant saving in cost associated with coordinating with each engineer independently. In addition, the project engineer was able to provide the draftspeople with information needed to begin production of many of the various sheets needed in the final submission package. The following subsections provide the reader with examples of typical final engineering obstacles and how they were overcome in the case of the Union Mills project.

### Typical Townhouse Section

This section discusses typical features of the design and identifies specific problem-solving techniques used on this project. The overall composite plan is shown in Figure 40.15 and should assist in identifying the various sections discussed.

**Section 1.** Several items on Section 1 are important to note. Of particular note is the new sanitary sewer through this section. Initially, the existing sanitary sewer through the site was intended to be saved and to be kept intact. However, when the contractor reviewed this initial design, it was obvious that in some areas it was more cost effective to move the sewer than to design a lot layout to keep it intact.

The design for the widening of Lee Highway for an additional lane included the saving of a substantial tree buffer. Unfortunately, when the design was being finalized, the engineers found that the trees remaining would be fairly sparse and small in size. Some had actually died over the prior winter, and the construction of the third lane required additional tree clearing. Due to the potential noise impact on planned residential units adjacent to the roadway, it became a major concern. The solution was to construct a landscaped berm and a 6-ft-high board fence. This amenity provided the need for visual and noise protection from Lee Highway (see Figure 40.15).

One of the zoning requirements was the construction of an asphalt bike trail throughout the project. The portion of the trail along Union Mill Road was to be 8 ft wide and within the site the trail was to be 6 ft wide. Because the floodplain area is generally wet and the soils not conducive to building a trail, the trail was shifted to a higher elevation. The relocation of the trail outside the marshy area and away

from the storm sewer outfalls, as shown in Figure 40.15, resulted in considerably easier construction and eliminated the need to construct footbridges and additional culvert crossings of the trail.

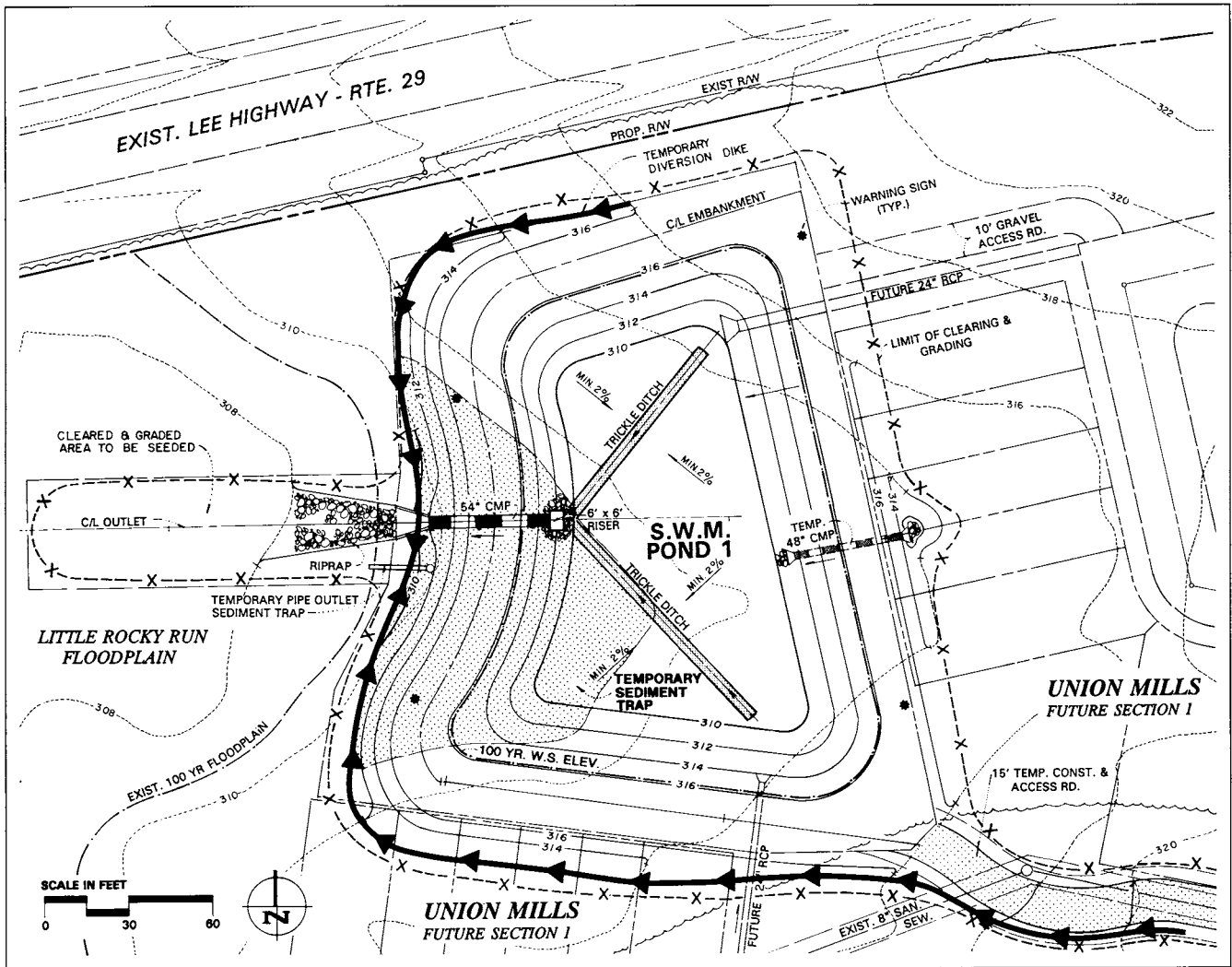
Another noteworthy engineering consideration is the unusual sediment and erosion design feature near Pond 1 (see Figure 40.19). Pond 1 was actually constructed partly on fill and partly on the natural topography. Therefore, the pond could not be used for any temporary sediment and erosion control as is typically the case. As a result, a separate pipe into the pond was called for with separate sediment traps around the periphery of the pond.

Considerable engineering design focus was given to interior street layout and parking. Specifically, note in Figure 40.15 that almost the entire project provides the traditional turnarounds so vehicles can leave the area in a continual motion without needing to stop and back up to exit the site. This is a worthwhile marketing feature and was implemented wherever possible within the project. Also note that the allocation and dispersion of parking spaces throughout this section are balanced and provide close and easy access to all units for owners and guests.

### Single-Family Detached House Sitings

It is evident in looking at the existing topography in Figure 40.20 that there is considerable topographic relief in the single-family detached (SFD) area. As the engineers were performing the house grading and layout, they were aware that the siting of the units was critical. Street design had to best fit this topographic area, and consequently the design criteria to best site these units in this critical area of major cuts and fills went far beyond what is considered normal. Buffer requirements along property lines made the sites extremely tight, and difficulties were encountered, particularly with lots along the filled section of Union Mill Road. The internal streets and driveways were also on fill, and the result included the pipestem driveway design shown for lots 52 and 53 (Figure 40.20). Additionally, the basements of these units were on 5 to 8 ft of fill at some points. In this engineering design process and as a result of the inability to steepen the grade up to the connection to Cavalier Woods Lane, two lots had to be deleted from this area because the buildings had to be sited on an unacceptable amount of fill. In fact, they would have been on 15 ft or more of fill and very close to Union Mill Road. Siting the units in such a manner made no sense from a marketing perspective and could have negatively impacted potential buyers' impressions of the entire area.

The early grading studies were reevaluated and lots were reconfigured to accommodate the two lots deleted from the area adjacent to Cavalier Woods subdivision. This process involved careful maneuvering to save as many trees as possible between the lots and Union Mill Road. The idea was to create and maintain a noise and visual barrier between Union Mill Road, the retail center, and the backs of these lots. As discussed previously, this tree-saving process was



**FIGURE 40.19** Stormwater management, pond #1.

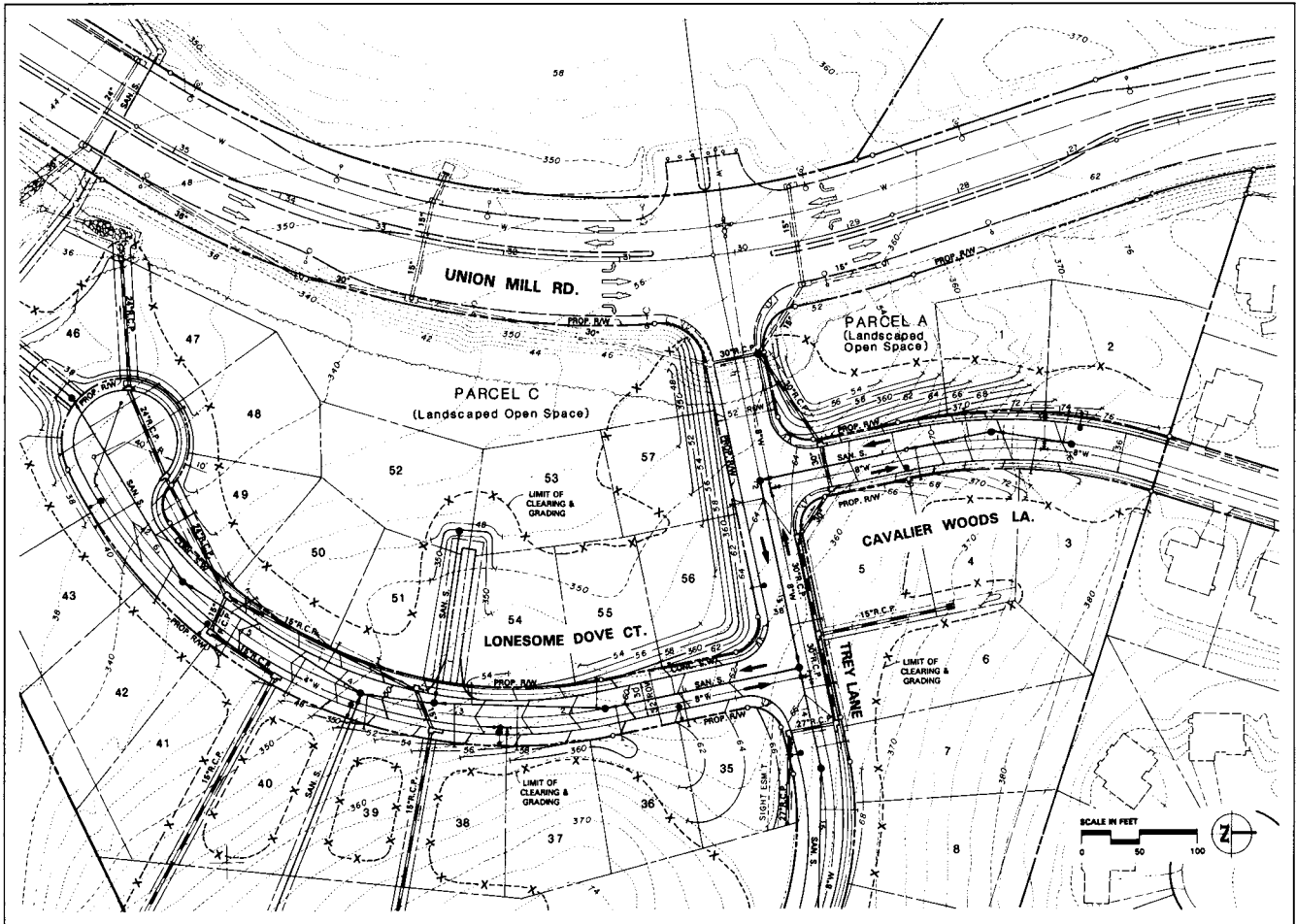
required by the zoning, important aesthetically, and assisted in the meeting of BMP criteria because the two small ponds that these lots drained into barely met that criteria. Evidence of the concern for the preservation of open space is seen in Figure 40.20, where two parcels (A and C) are identified as landscaped open space and are to be dedicated to the Home Owners Association.

Another notable engineering problem resolving the difficulty with the design of a street that connected with a roadway in an adjacent subdivision. This street, Cavalier Woods Lane, is shown in Figure 40.20. This adjacent subdivision had been previously constructed with a temporary turn-around in Section 6 of Centerville Green. At that point in time, the engineer for the other development had not been required to account for the significant drop in elevation from the Cavalier Woods site into the Union Mills site in designing the street. Numerous difficulties were encountered in trying to get Cavalier Woods Lane and Trey Lane to adequately connect between Union Mill Road and the existing Cavalier Woods Lane. The roadway classification required a specific road design criteria which, if held, forced

the engineers of the Union Mills site to raise the adjacent units and also Union Mill Road to accommodate the connection. In the process, options such as cul-de-sac and pipestem design were reviewed. A decision was made to develop a request for a waiver to these design criteria. The design criteria addressed involved design speed and maximum street grade. Considerable effort went into preparing for the discussions with the State transportation department and the County staff, with numerous design options presented. The result was only partially successful; that is, for a few hundred feet of street the impact was that one lot became unbuildable and for two other lots that were remaining on one side of the street (lots 1 and 2) the market value became somewhat marginal in comparison with the others in the same area. With the benefit of hindsight, an earlier identification and presentation of this request may have been more successful.

### Retail Center

Design of the retail center commenced after the infrastructure and residential design development had been started.



**FIGURE 40.20** Part of Centreville Green, section 6.

As with the residential component of the project, specific users for the small shops and the freestanding pad sites had not been selected. However, the user for the anchor store was known. Major design constraints had previously been addressed with the infrastructure plans for the overall Union Mills project. Therefore, the bulk of the design effort was to address the “normal” issues as described above. Moreover, the site was heavily dependent on the prior approvals of various infrastructure plans such as Union Mill Road/Braddock Road intersection, Union Mill Road, various stormwater management ponds, and the sanitary sewer outfall.

The retail center is anchored by a large  $\pm 55,000$ -ft<sup>2</sup> food store, with the remaining small shops and freestanding buildings sited adjacent to or near the anchor. The design of a retail center incorporates many elements in addition to those required of a residential subdivision. Additionally, these elements include handicap access facilities, fire protection, site lighting, loading facilities, trash disposal, and major water services, to name a few. The design engineer often serves as a central coordinator among the client, the architect, mechanical and electrical engineers, utility companies, and other consultants. Unlike a residential section, where a single builder often develops/builds all the units,

a retail center contains many users. The needs and concerns of each of the users must be balanced against and coordinated with the owner/developer. The Union Mills retail center site plan was developed using a building footprint for the anchor store, which had been built in a number of other areas. However, the strip store’s footprints were developed in consultation with the client and the architect. A number of design changes to accommodate the ultimate users for the freestanding pad sites occurred later in the process.

Originally, the retail center was located on a hilly site interspersed with several natural swales. Subsequently, the natural topography provided a major design constraint on the retail center. Typically, retail users dictate that the buildings and parking lots be as flat as possible to provide ease of access for the user. Additionally, the stores are held flat to eliminate the need for steps. The flatness of the building slabs also provides flexibility in the size of the store that can occupy a given space. In order to provide a relatively flat site, deep cuts and fills were required. This necessity proved to be an advantage for the residential portions of the project where fill material was needed.

The previous soil borings had given the designer the information needed to ensure that rock excavation would

not be a construction problem. The storm sewer outfalls into the residential site were stepped down at manholes in order to reduce the high velocities that can be encountered when traversing a steep slope. Typically, in steep terrain, the storm sewer and sanitary sewer are extraordinarily deep. Keeping the pipe slopes very flat minimized the depth of trench to a limited extent.

Typically, with a retail site, a large area is denuded at one time. Siltation and dust control can be a major problem if the site is left exposed for a long period of time. In Union Mills, the storm sewer outfalls were built first and siltation controls installed to preclude silt from washing from the site. Additionally, aggregate subbase was installed at the earliest opportunity to help control dust.

## PART V: PLAN SUBMISSION AND PERMITTING

The preparation, review, and approval of the final engineering documents requires the involvement of local and state governments, the client, and the design consultant. To understand better the review and approval process as it relates to Union Mills requires an understanding of the participants and their role in the review process in Fairfax County as well as the structure of the review process.

In Fairfax County, the governing body, the Board of Supervisors, is ultimately responsible for the review and approval of all engineering plans and plats as well as inspections and monitoring during project construction. However, the Board has delegated these tasks primarily to the Department of Environmental Management (DEM) and its Director. The Director is authorized to review and approve construction plans and plats on behalf of the Board. DEM, in turn, relies on the input from other agencies within the County government as well as other local and State agencies, such as the Fairfax Water Authority, the Virginia Department of Transportation, and the State Health Department, in the review of engineering plans.

Within DEM, several divisions are involved in the design/construction review/approval process. Specifically, the Division of Design Review is responsible for the review and approval of construction plans and plats, as well as other studies, reports, and miscellaneous items relating to the land development process. Union Mills is located within the Springfield District, with that District reviewer serving as the central coordinator for each project. Other agency reviews are processed through this individual. Also contained within DDR are the Arborist Branch, Technical Support Branch, and Special Projects Branch, all of which will review a given project. The Special Projects Branch has responsibility for the review of technical information relating to hydrology, soils, siltation, and erosion control, as well as other specialized studies relating to a given project.

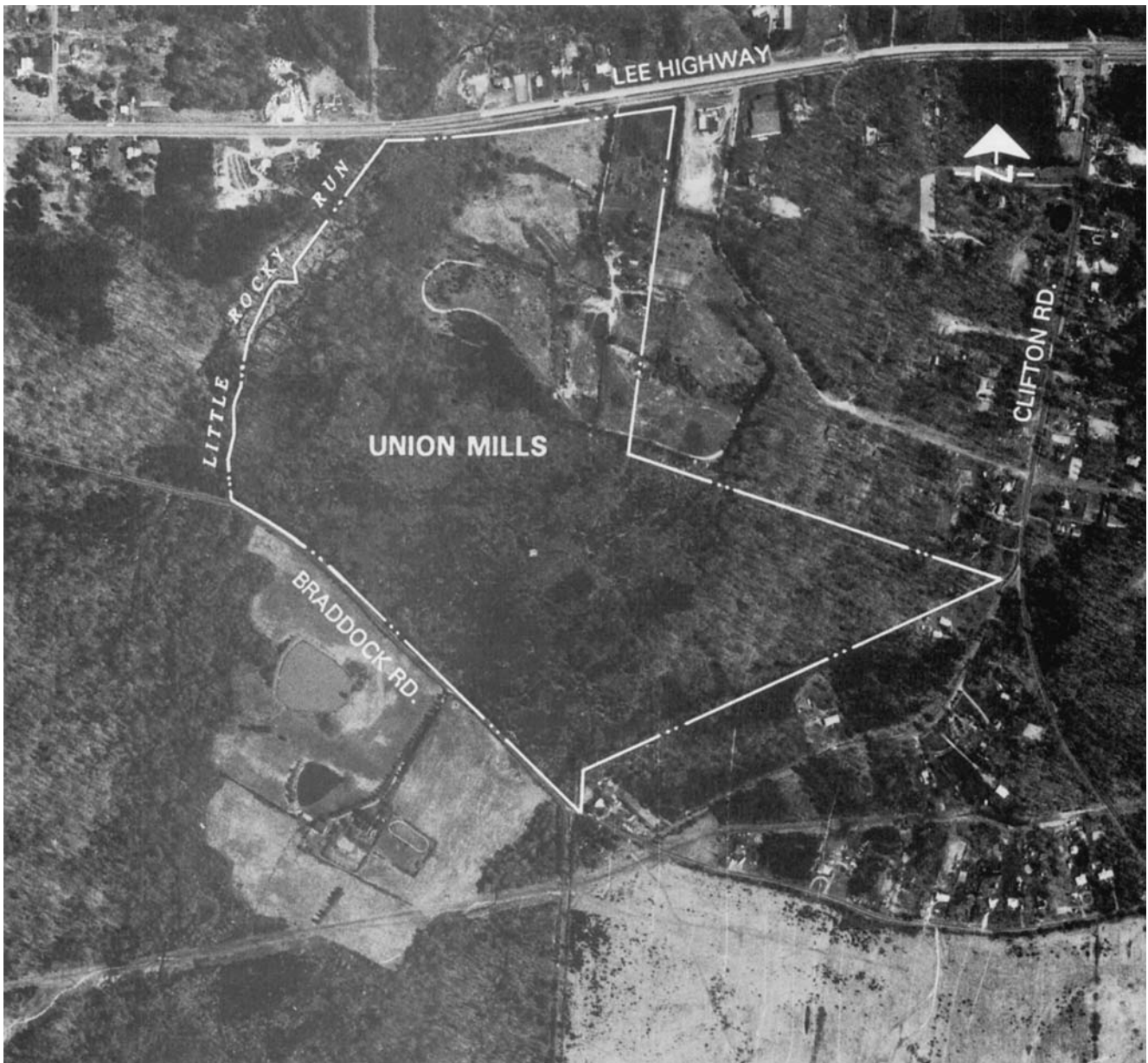
Other agencies involved in the review process include the Fire Marshal, Water Authority, Department of Public Works, Virginia Department of Transportation, Fairfax County Park Authority, Health Department, and the Northern Virginia Water and Soils Conservation Service.

Depending on what issues surface during the review process, other agencies, both state and local, may be asked to review the plans. The review agency must ensure that all projects consistently comply with published standards and policies and that the project does not endanger the health, safety, and welfare of the general public. The process by which construction plans and plats are routed through Fairfax County is shown in Figure 1 of Chapter 34.

At the time Union Mills was being developed, the review and approval process in Fairfax County for construction plans was approximately 15 to 18 months for each plan. The client, the Hazel/Peterson Companies, served as owner and developer, and it was their intention to build the infrastructure components of the project; that is, major roads, sanitary sewer outfalls, stormwater management facilities, and construct the commercial/retail component. The residential components of the project and the day care center would then be sold to individual builders as finished lots. In essence, those purchasing finished lots forgo the delays associated with site plan approval and need only building plan approval to begin construction. From the outset, the client's goal was to expedite the design and review/approval process so construction could begin as quickly as possible. The significant amount of time required for the zoning process, the rising costs of development, dwindling sales activity, and heavy costs of infrastructure development imposed by Fairfax County added economic uncertainty to the project. Thus, in order to expedite the overall process and ensure cost-effective design and construction, Hazel/Peterson Companies utilized the following proactive team approach to the project:

- To ensure continuity throughout the lengthy design process, the developer decided to use the same consultant to perform the planning, zoning, and engineering tasks.
- Hazel/Peterson personnel would interface and coordinate with Fairfax County as needed. The developer would establish overall schedules and priorities with senior management in DEM while specific issues were addressed jointly by junior staff within Hazel/Peterson, Fairfax County, and Dewberry & Davis, the engineering firm.
- Weekly meetings and daily interaction between the developer and design professional would be maintained throughout the life of the project.
- A general contractor was selected by Hazel/Peterson early in the design process to review plans and help develop strategies to reduce construction costs.

The design professional's role in the development process incorporates a variety of skills, such as technical design skills, knowledge of local ordinances, policies, and procedures, and communication. How the land development engineer welds these skills together can greatly influence the



**FIGURE 40.21** Union Mills site (circa 1974). Courtesy Air Survey Corporation, Sterling, Va.

success or failure of a project. A positive engineer/client relationship is integral to the success of the project. The engineering project manager and design team selected for the Union Mills project had an extensive history with the client. Their previously established working relationship allowed for open discussion and joint decision-making. Knowledge of the site was an important factor during the early stages of the Union Mills project. As will be illustrated later, this knowledge of the site enabled the team to identify and resolve issues prior to the start of final design

#### **PART VI: SUMMARY OBSERVATIONS**

The purpose of this case study is to demonstrate the tremendous number of outside influences involved in the

planning and engineering of a major land development project. The influence of citizen activists, special interest groups, and raw politics is ever present. The days of engineering a project without consideration of environmental, social, and aesthetic considerations are long gone. Today's process mandates not only coordination and approval of adjacent neighbors but consideration of future neighbors in terms of transportation improvements, stormwater management, environmental protection, and a myriad of other aspects of the development process.

The Union Mills project was selected for this case study because it had almost all the elements of the development process, from planning and zoning through final site design and engineering. Further, the project was a mixed use with



**FIGURE 40.22** Union Mills site (circa 1993). Courtesy of Air Survey Corporation, Sterling, Va.

townhouses, single-family detached units and a retail center being developed. The mixture of uses provided the opportunity to demonstrate obstacles overcome through the planning and zoning process as well as the diversity of engineering techniques implemented to resolve issues within the various land bays.

The Union Mills project was also notable in that at the time it was being developed, awareness of the environment and traffic congestion resulted in additional policies and ordinances that had a significant impact on the development process. Such changes include the adoption and successful defense in court of the Occoquan Basin Study and related downzoning by the Fairfax County Board of Supervisors and more stringent environmental protection policies

prompted by State government. In addition, a once rural community was being transformed into a more densely populated suburban environment with the corresponding traffic congestion and need for additional public facilities such as schools, fire and police facilities, and more effective stormwater management. The citizen activists and other interest groups used the Union Mills project as a vehicle to voice their concerns regarding the impacts, real or perceived, of the proposed Union Mills project.

The success of this project was due, to a great extent, to the development team that had been established and its technical skills as well as skill in communications and negotiations. It is unrealistic to believe that a project can be approved through a public process without negotiations

and compromise on a wide variety of issues. In the case of Union Mills, compromises were made in the planning, zoning, and engineering elements of the developments. However, the issues were thoughtfully anticipated, and consequently the compromises were disruptive but not fatal to the project. As demonstrated in this study, ongoing effective communications with *all* participants in the process was essential to the success of this project. A comparison of the site before and after development is shown in the aerial photos in Figures 40.21 and 40.22.

In planning and designing a project, thought, and consideration must be given to the possible impacts of the proposal on the existing community and environment. Again, in the case of Union Mills, anticipation of the issues and communication with the appropriate interest groups took place early and continued throughout the development process. Although technology is ever-advancing and changing, the human element remains paramount in the success of a given project. The human element was clearly responsible for the successful planning, engineering, and development of the beautiful community of Union Mills.

## REFERENCES

- Barne, Donna, 1987a, "Hazel Won't Build Center Only Houses Union Mill Plan Dead," *CentreView*, October 22, p. 3.
- Barne, Donna, 1987b, "Neighbors Give Boost to Centreville Green," *CentreView*, October 8, p. 3.
- Barne, Donna, 1987c, "Watchdogs Eye Area Land Use," *CentreView*, October 29, 1987, pp. 1, 3, 24.
- Fairfax County, 1975, *The Fairfax County, Virginia Plan*, 1975, as amended through July 1977.
- Fairfax County, 1982, Staff Report, Application No. PCA 81-5-090, July 22.
- Fairfax County, 1983, *A Guide to the Development Process in Fairfax County, Virginia*, vol. 1, *Land Use and Planning*, Prepared by Office of Research and Statistics, Department of Environmental Management, Office of Comprehensive Planning, revised January 1983.
- Fairfax County, 1986a, *Area III Items Nominated for 1986 Annual Plan Review Consideration*, Office of Comprehensive Planning.
- Fairfax County, 1986b, *Shopping Centers, Meeting the Demand for Retail Commercial Facilities in Western Fairfax County*, Office of Comprehensive Planning, August.
- Fairfax County, 1991a, *The Comprehensive Plan for Fairfax County, Virginia*, 1991 ed., as amended through March 9, 1992.
- Fairfax County, 1991b, Office of Research and Statistics, *1991 Fairfax County Profile*.
- Frank, Richard, President, Western Fairfax County Citizens Association Land Use Committee. 1988. Letter to Mrs. Elaine McConnell, Supervisor, February.
- Frey, Michael R., Supervisor, Fairfax County Board of Supervisors, 1993, Interview, March 24.
- Moncada, Carlos, 1987, "Company Withdraws 'Centreville Green,'" *Times-Courier*, October 22, pp. A1, A8.
- Saxe, Jeffrey, Senior Vice President, Planning, Hazel/Peterson Companies, 1992/93, Interview, September 25, 1992, and June 23, 1993.

## ADDITIONAL READING

- Fairfax County, *The Comprehensive Plan for Fairfax County Introduction/Countywide*, 1986 ed., as amended through October 27, 1986.
- Fairfax County, *Limited Year 1986 Comprehensive Plan Review, Final Staff Report*, Comprehensive Plan Item Number 86-II-3BR(A).
- Fairfax County, *Occoquan Basin Study*, A Report by the Office of Comprehensive Planning, Fairfax County, Virginia, March, 1982.
- Fairfax County, *Occoquan Basin Study—Amendments to the Comprehensive Plan Adopted by the Board of Supervisors June 15, 1982*.
- Fairfax County, *Policy Plan: The Countywide Policy Element of the Comprehensive Plan for Fairfax County, Virginia*, 1990 ed., adopted by the Board of Supervisors August 6, 1990.
- Fairfax County, *The Proffer System in Fairfax County*, Zoning Evaluation Division, Office of Comprehensive Planning, Fairfax County, Virginia, November 1985, revised July, 1989.
- Fairfax County, Staff Report, Application No. FDP 81-S-090-1, January 3, 1985.
- Fairfax County, Staff Report, Application No. PCA 81-S-090-1; CDPA/FDPA 81-S-090-1, July 15, 1987
- Fairfax County, Staff Report, Application No. PCA/CDPA/FDPA 81-S-090-2, RZ 88-S-022, SE 88-S-013, SE 88-S-014, SE 88-S-015, June 21, 1988.
- Fairfax County, Staff Report, Application No. RZ 78-S-039.
- Fairfax County, Staff Report, Application No. RZ 81-S-090, July 22, 1982.
- Fairfax County, Staff Report Addendum, July 11, 1988.
- Frank, Richard, Chairman, Western Fairfax County Citizens Association Land Use Committee, telephone conversation, October 6, 1992.
- Moncada, Carlos, "Accord Forged Citizens Agree Project Bucks Plan Process," *Times-Courier*, October 8, 1987, pp. A1, A12.
- Moncada, Carlos, "Braddock Spur Dropped," *Times-Courier*, October 1, 1987, pp. A1, A8.





# CHAPTER 41

## TECHNICAL APPENDIX

### PROPERTIES OF WATER IN U.S. CUSTOMARY UNITS

TEMP. (°F)	DENSITY (slugs/ft <sup>3</sup> )	SPECIFIC WEIGHT (lb/ft <sup>3</sup> )	HEAT OF VAPORIZATION (Btu/lb)	VISCOSITY <sup>1</sup>		VAPOR PRESSURE (In. of Hg) (lb/in. <sup>2</sup> )	
				ABSOLUTE (lb-s/ft <sup>2</sup> )	KINEMATIC (ft <sup>2</sup> /s)		
32	1.940	62.418	1075.5	3.746	1.931	0.180	0.089
40	1.940	62.426	1071.0	3.229	1.664	0.248	0.122
50	1.940	62.409	1065.3	2.735	1.410	0.362	0.178
60	1.938	62.366	1059.7	2.359	1.217	0.522	0.256
70	1.936	62.301	1054.0	2.050	1.058	0.739	0.363
80	1.934	62.216	1048.4	1.799	0.930	1.032	0.507
90	1.931	62.113	1042.7	1.595	0.826	1.422	0.698
100	1.927	61.994	1037.1	1.424	0.739	1.933	0.950
120	1.918	61.713	1025.6	1.168	0.609	3.448	1.693
140	1.908	61.379	1014.0	0.981	0.514	5.884	2.890
160	1.896	61.000	1002.2	0.838	0.442	9.656	4.742
180	1.883	60.580	990.2	0.726	0.386	15.295	7.512
200	1.868	60.121	977.9	0.637	0.341	23.468	11.526
212	1.860	59.828	970.3	0.593	0.319	29.921	14.696

<sup>1</sup> To obtain values of viscosity, multiply values shown in table by 10<sup>-5</sup>

### PROPERTIES OF WATER IN SI UNITS

TEMP. (°C)	DENSITY (kg/cm <sup>3</sup> )	SPECIFIC WEIGHT (kN/m <sup>3</sup> )	HEAT OF VAPORIZATION (cal/g)	VISCOSITY <sup>2</sup>		VAPOR PRESSURE (mm Hg) (g/cm <sup>2</sup> )	
				ABSOLUTE (cP)	KINEMATIC (cSt)		
0	999.87	9.805	597.3	1.781	1.785	4.58	6.23
5	999.99	9.807	594.5	1.518	1.519	6.54	8.89
10	999.73	9.804	591.7	1.307	1.306	9.20	12.51
15	999.13	9.798	588.9	1.140	1.140	12.78	17.38
20	998.24	9.789	586.0	1.002	1.003	17.53	23.83
25	997.08	9.777	583.2	0.890	0.893	23.76	32.30
30	995.68	9.764	580.4	0.798	0.801	31.83	43.27
40	992.25	9.730	574.7	0.653	0.658	55.34	75.23
50	988.07	9.689	569.0	0.547	0.554	92.56	125.83
60	983.23	9.642	563.2	0.466	0.474	149.46	203.19
70	977.80	9.589	557.4	0.404	0.413	233.79	317.84
80	978.12	9.530	551.4	0.355	0.365	355.28	483.01
90	965.34	9.466	545.3	0.315	0.326	525.89	714.95
100	958.39	9.399	539.1	0.282	0.294	760.00	1033.23

<sup>2</sup> 1 poise (P) = 0.10 N·s/m<sup>2</sup> ∴ 1 centipoise (cP) = 1 × 10<sup>-3</sup> N·s/m<sup>2</sup>  
1 stoke (St) = 1 cm<sup>2</sup>/s ∴ 1 centistoke (cSt) = 1 × 10<sup>-6</sup> m<sup>2</sup>/s

## SCS 24-HR RAINFALL DISTRIBUTION (At ½ Hr. Increments)

TIME (h)	FRACTION OF 24-h RAINFALL DEPTH			
	TYPE I	TYPE IA	TYPE II	TYPE III
0.0	0.00000	0.00000	0.00000	0.00000
0.5	0.00871	0.01000	0.00513	0.00500
1.0	0.01745	0.02000	0.01050	0.01000
1.5	0.02621	0.03500	0.01613	0.01500
2.0	0.03500	0.05000	0.02200	0.02000
2.5	0.04416	0.06600	0.02813	0.02519
3.0	0.05405	0.08200	0.03450	0.03075
3.5	0.06466	0.09800	0.04113	0.03669
4.0	0.07600	0.11600	0.04800	0.04300
4.5	0.08784	0.13500	0.05525	0.04969
5.0	0.09995	0.15600	0.06300	0.05675
5.5	0.11234	0.18000	0.07125	0.06419
6.0	0.12500	0.20600	0.08000	0.07200
6.5	0.13915	0.23700	0.08925	0.08063
7.0	0.15600	0.26800	0.09900	0.09050
7.5	0.17460	0.31000	0.10925	0.10163
8.0	0.19400	0.42500	0.12000	0.11400
8.5	0.21900	0.48000	0.13225	0.12844
9.0	0.25400	0.52000	0.14700	0.14575
9.5	0.30300	0.55000	0.16300	0.16594
10.0	0.51500	0.57700	0.18100	0.18900
10.5	0.58300	0.60100	0.20400	0.21650
11.0	0.62300	0.62400	0.23500	0.25000
11.5	0.65550	0.64500	0.28300	0.29800
12.0	0.68400	0.66400	0.66300	0.50000
12.5	0.70925	0.68300	0.73500	0.70200
13.0	0.73200	0.70100	0.77200	0.75000
13.5	0.75225	0.71900	0.79900	0.78350
14.0	0.77000	0.73600	0.82000	0.81100
14.5	0.78625	0.75281	0.83763	0.83406
15.0	0.80200	0.76924	0.85350	0.85425
15.5	0.81725	0.78529	0.86763	0.87156
16.0	0.83200	0.80096	0.88000	0.88600
16.5	0.84625	0.81625	0.89119	0.89838
17.0	0.86000	0.83116	0.90175	0.90950
17.5	0.87325	0.84569	0.91169	0.91938
18.0	0.88600	0.85984	0.92100	0.92800
18.5	0.89825	0.87361	0.92969	0.93581
19.0	0.91000	0.88700	0.93775	0.94325
19.5	0.92125	0.90001	0.94519	0.95031
20.0	0.94225	0.92489	0.95844	0.96336
20.5	0.94225	0.92489	0.95844	0.96336
21.0	0.95200	0.93676	0.96475	0.96944
21.5	0.96125	0.94825	0.97094	0.97523
22.0	0.97000	0.95936	0.97700	0.98075
22.5	0.97825	0.97009	0.98294	0.98598
23.0	0.98600	0.98044	0.98875	0.99094
23.5	0.99325	0.99041	0.99444	0.99561
24.0	1.00000	1.00000	1.00000	1.00000

**CONVERSION FACTORS**

<i>Multiply</i>	<i>By</i>	<i>To Obtain</i>	<i>Multiply</i>	<i>By</i>	<i>To Obtain</i>
Acre-feet	1,233.49	cubic meters	Feet	$1.894 \times 10^{-4}$	miles (statute)
Acre-feet	43,560	cubic feet	Feet	30.48	centimeters
Acre-feet	0.0504	second-foot-day	Feet/min.	0.5080	centimeters/sec.
Acre-feet	1,235	cubic yard	Feet/sec.	0.3048	meters/second
Acres	10	square chains	Feet/sec	0.6818	miles/hour
Acres	4,046.87	square meters	Feet/sec	0.5921	knots
Acres	0.4047	hectares	Feet/sec	30.48	centimeters/sec.
Acres	$1.562 \times 10^{-3}$	square miles	Foot-pounds	$3.766 \times 10^{-7}$	kilowatt-hours
Acres	43,560.0	square feet	Foot-pounds	$1.286 \times 10^{-3}$	BTU's
Angstrom units	$3.937 \times 10^{-9}$	inches	Foot-pounds	1.356	Joules
Angstrom units	$1 \times 10^{-4}$	microns	Furlongs	660.0	foot
Atmosphere	101.325	kilopascals	Furlongs	0.125	miles (statute)
Atmospheres	14.696	pounds/square inch	Gal/minute	$2.227 \times 10^{-3}$	cubic feet/sec.
Atmospheres	76.0	cms of mercury	Gallons	3.785	liters
Atmospheres	33.8995	feet of water	Gallons/min	8.0208	cubic feet/hour
Atmospheres	20.921	inches of mercury	Gallons of water	8.3453	pounds of water
BTU/hr	0.2931	watts	Grads	0.9	degrees (angle)
BTU's	$2.928 \times 10^{-4}$	kilowatt-hrs	Grads	0.01570797	radians
BTU's	$3.931 \times 10^{-4}$	horsepower-hrs	Grams	$3.527 \times 10^{-2}$	ounces (avoirdupois)
Calories, gram (mean)	$3.9685 \times 10^{-3}$	BTU (mean)	Grams	$2.205 \times 10^{-3}$	pounds
Centares	1.0	square meters	Grams	$3.215 \times 10^{-2}$	ounces (troy)
Centimeters	$3.281 \times 10^{-2}$	feet	Hectares	10,000	square meters
Centimeters	10.0	millimeters	Hectares	$1.076 \times 10^5$	square feet
Centimeters	0.3937	inches	Hectares	2.471	acres
Centimeters	$1 \times 10^{-2}$	meters	Horsepower	745.7	watts
Centimeters	$1 \times 10^{-5}$	kilometers	Horsepower	0.7457	kilowatts
Centimeters/sec	0.03281	feet/sec.	Horsepower	42.42	BTU/min.
Chain	22	yards	Horsepower	550	ft-lbs/sec.
Chain	66	feet	Hours	$5.952 \times 10^{-3}$	weeks
Chain	4	rods	Hours	$4.167 \times 10^{-2}$	days
Chains	792.0	inches	Inches	25.4	millimeters
Cubic centimeters	$2.113 \times 10^{-3}$	pints (U.S. liq.)	Inches	$1.578 \times 10^{-5}$	miles
Cubic centimeters	0.06102	cubic inches	Inches	2.540	centimeters
Cubic feet/sec	0.6463	million gallons/day	Joules	$9.480 \times 10^{-4}$	BTU's
Cubic feet/min.	62.43	pounds water/min.	Joules	$1 \times 10^7$	ergs
Cubic feet/sec.	448.831	gallons/min.	Kilograms	2.205	pounds (avoirdupois)
Cubic feet	0.02832	cubic meters	Kilometers	1,000.0	meters
Cubic foot	7.48	U.S. gallon	Kilometers	3,281.0	feet
Cubic ft/sec.	28.316	liter/sec.	Kilometers	0.6214	miles
Cubic inch	16.3870	cubic centimeters	Kilometers/hr	0.5396	knots
Cubic meter	35.3147	cubic feet	Kilowatt-hours	3,413.0	BTU's
Cubic meters/sec	35.312	cubic feet/sec.	Kilowatts	1.341	horsepower
Cubic meters	1,000	liters	Knots	1.852	kilometers/hr
Cubic meters/sec	15,800	gallons/min.	Knots	0.5144	meters/sec
Cubic yard	27	cubic foot	Knots	1.0	nautical miles/hr.
Cubic yard	0.7646	cubic meter	Knots	1.151	statute miles/hr.
Cubic yards	202.0	gallons	Knots	6,080.0	feet/hour
Day (mean solar)	86,400	seconds	Labors	177.14	acres
Day (sidereal)	86,164.09	seconds	Leagues	4,428.40	meters
Degrees (angle)	0.0174533	radians	Links (gunter's)	66	feet
Degrees (angle)	17.7778	mile	Links (surveyor's)	7.92	inches
Degrees/sec.	0.1667	revolutions/min.	Liters	61.02	cubic inches
Dynes	$1.020 \times 10^{-3}$	grams	Liters	0.2642	gallons (U.S. liq.)
Dynes	$1 \times 10^{-5}$	joules/meter (newtons)	Liters	1,000.0	milliliters
Engineer's link	1	feet	Liters	2.113	pints (U.S. liq.)
Ergs	$7.3670 \times 10^{-8}$	foot-pounds	Liters	1,000.0	cubic centimeters
Ergs	$2.778 \times 10^{-14}$	kilowatt-hours	Liters/min.	$5.886 \times 10^{-4}$	cubic ft/sec.
Ergs	$9.480 \times 10^{-11}$	BTU's	Liters/second	15.85	gallons/day
Fathoms	6.0	feet	Meters	$5.396 \times 10^{-4}$	miles (nautical)
Fathoms	1.839	meters	Meters	100.0	centimeters
Feet	0.3048	meters	Meters	$6.214 \times 10^{-4}$	miles (statute)
Feet	$1.645 \times 10^{-4}$	miles (nautical)	Meters	$1 \times 10^{-3}$	kilometers

CONVERSION FACTORS

<i>Multiply</i>	<i>By</i>	<i>To Obtain</i>	<i>Multiply</i>	<i>By</i>	<i>To Obtain</i>
Meters	3.281	feet	Pounds/square inch	6.895	Newtons/square meter
Meters	1,000.0	millimeters	Pounds/square foot	47.880	Pascals
Microns	$1 \times 10^{-6}$	meters	Quarts (dry)	67.20	cubic inches
Miles (nautical)	6,080.27	feet	Quarts (liq.)	57.75	cubic inches
Miles (nautical)	1.853	kilometers	Quarts (liq.)	0.9463	liters
Miles (nautical)	1.1516	miles (statute)	Quarts (liq.)	0.25	gallons
Miles (statute)	0.8684	miles (nautical)	Radians	3,438.0	minutes
Miles (statute)	1.609	kilometers	Radians	$57^{\circ}17'44.806''$	degrees (angle)
Miles (statute)	320	rods	Radians	57.2957795	degrees (angle)
Miles (statute)	5,280	feet	Radians/sec	9.549	revolutions/min.
Miles (statute)	80	chains (surveyor's)	Revolutions	360.0	degrees
Miles/hour	88.0	feet/min.	Revolutions/min.	6.0	degrees/sec.
Millibar	0.01 <sup>43</sup>	pounds/square inch	Rods	1	poles
Milligram/liter	1.0	parts per million	Rods	16.5	feet
Milligrams/liter	8.345	pounds/million gallons	Rods	5.029	meters
Milliliters	$1 \times 10^{-3}$	liters	Rods	1	perches
Millimeters	$3.937 \times 10^{-2}$	inches	Rods (surveyor's measure)	5.5	yards
Mils	3,037,500	minutes	Seconds	$1.667 \times 10^{-2}$	minutes
Mils	0.05625	degrees (angle)	Seconds (angle)	$4.848137 \times 10^{-6}$	radians
Minutes (angle)	0.29630	mils	Seconds (sidereal)	0.9972696	seconds (mean solar)
Minutes (angle)	0.000290888	radians	Slug/ft <sup>3</sup>	515.4	Kg/cubic meter
Minutes (mean solar)	60	seconds	Slugs	32.17	pounds
Minutes (sidereal)	59.83617	seconds	Square miles	640	acres
Newtons	$1 \times 10^5$	dynes	Square miles	27,878,400	square feet
Newtons/square meter	1	Pascals	Square meters	10.76	square feet
Ounces	$6.25 \times 10^{-2}$	pounds	Surveyor's link	7.92	inches
Ounces	28.349527	grams	Surveyor's link	0.66	feet
Ounces (troy)	1.09714	ounces (avoirdupois)	Tons (long)	1.120	tons (short)
Outs	10	33-foot chains	Tons (long)	2,240.0	pounds
Outs	330	feet	Tons (long)	1,016.0	kilograms
Pints (liq.)	0.125	gallons	Tons (metric)	2,205	pounds
Pints (liq.)	28.87	cubic inches	Tons (short)	907.1848	kilograms
Pints (liq.)	473.2	cubic centimeters	Tons (short)	0.89287	tons (long)
Pints (liq.)	0.5	quarts (liq.)	Tons (short)	2,000.0	pounds
Pounds	0.4536	kilograms	VARA (California)	33	inches
Pounds	14.5833	ounces (troy)	VARA (Texas)	33.333	inches
Pounds	1.21528	pounds (troy)	Watts	$1.341 \times 10^{-3}$	horsepower
Pounds	16.0	ounces	Watts	1.0	joules/sec.
Pounds/cubic ft	16.02	Kilograms/cubic meter	Watts	3.4129	BTU/hour
Pounds (force)	4.4482	Newtons	Yards	$5.682 \times 10^{-4}$	miles (statute)
Pounds (mass)	0.4536	Kilograms	Yards	$4.934 \times 10^{-4}$	miles (nautical)
Pounds/square inch	6.895	Kilo Newton/square meter	Yards	0.9144	meters

RELATIONSHIP BETWEEN TEMPERATURES

$^{\circ}\text{F} = 32 + (\frac{9}{5})^{\circ}\text{C}$		$^{\circ}\text{C} = \frac{5}{9} (^{\circ}\text{F} - 32)$	
$^{\circ}\text{F}$	$^{\circ}\text{C}$	$^{\circ}\text{C}$	$^{\circ}\text{F}$
-20	-28.9	-20	-4
0	-17.8	-10	14
20	-6.7	0	32
32	0.0	10	50
40	4.5	20	68
60	15.5	30	86
80	26.6	40	104
100	37.8	50	122
120	48.9	0	140
140	60.0	70	158
160	71.1	80	176
180	82.2	90	194
212	100.0	100	212

$^{\circ}\text{Rankine } (^{\circ}\text{F Absolute}) = ^{\circ}\text{F} + 459.58$

$^{\circ}\text{Kelvin } (^{\circ}\text{C Absolute}) = ^{\circ}\text{C} + 273.1$

PREFIXES FOR USE WITH SI SYSTEM

Multiple	Prefix	Symbol
$10^{12}$	tera	
$10^9$	giga	G
$10^6$	mega	M
$10^3$	kilo	k
$10^2$	hecto	
10	deka	
$10^{-1}$	deci	d
$10^{-2}$	centi	c
$10^{-3}$	milli	m
$10^{-6}$	micro	$\mu$
$10^{-12}$	nano	n
$10^{-15}$	pico	p

**PHYSICAL PROPERTIES OF SELECTED MATERIALS**

<b>MATERIAL</b>	<b>UNIT WEIGHT Lb/Cu Ft</b>	<b>SPECIFIC GRAVITY*</b>	<b>MELTING POINT °F</b>	<b>BOILING POINT °F</b>	<b>SPECIFIC HEAT (CONST. PRESSURE FOR GASES)</b>
Acetylene	0.0732	0.00117	-114.7	-118.5	0.383
Air	0.0802	0.00129	—	—	0.238
Alcohol:					
Ethyl	49	0.789	-176	173	0.55
Methyl	49	0.792	-144	148.4	0.60
Aluminum (pure)	168	2.702	1220	3270-4530	0.226
Ammonia	0.0481	0.00077	-108	-28	0.520
		1			
Antimony	417	6.684	1167	2620	0.049
Argon	0.1114	0.00178	-308.6	-302.3	0.123
Asbestos	125-175	2.0-2.8	—	—	0.195
Asphalt	56-106	0.9-1.7	—	—	—
Basalt	150-200	2.4-3.2	—	—	0.20-0.24
Bauxite	159	2.55	—	—	—
Benzene	55	0.879	42	176.2	0.42
Bismuth	611	9.80	520	2590-2840	0.034
Borax	106-112	1.7-1.8	—	—	—
Brass:					
low (80 Cu, 20 Zn)	541	8.66	1830	—	—
red (85 Cu, 15Zn)	546	8.75	1880	—	—
Yellow (65 Cu, 35 Zn)	529	8.49	1710	—	—
Brick:					
common	112	1.8	—	—	—
fire	145	2.3	—	—	—
Bronze, comm. 90 Cu, 10 Zn)	549	8.80	1910	—	—
Butane:					
iso-	0.165	0.00264	-240	12	—
n	0.158	0.00252	-213	31	—
Cadmium	539	8.64	610	1410	0.055
Calcite	168	2.7	—	—	—
Calcium	97	1.55	1550	2200	0.149
Carbon:					
diamond	219	3.51	6700	7600	0.12
graphite	141	2.25	6700	7600	0.165
Carbon dioxide	0.1234	0.00198	-70.6	-112	0.20
Carbon monoxide	0.0781	0.00125	-340.6	-312.7	0.25
Cement, Portland:					
loose	94	1.51	—	—	—
solid	197	3.15	—	—	—
Chalk	119-175	1.9-2.8	—	—	0.214
Chlorine	0.2006	0.00321	-155	-30	0.113
Chromium	448	7.19	3430	4000	0.11
Coal:					
anthracite	87-112	1.4-1.8	—	—	—
bituminous	75-100	1.2-1.6	—	—	—
Cobalt	555	8.9	2700	5250	0.099
Concrete	144-150	2.3-2.4	—	—	0.156
Copper	556	8.92	1980	4700	0.092
Dolomite	177	2.84	—	—	—
Earth:					
dry, loose	75	1.2	—	—	—
dry, packed	95	1.5	—	—	—
moist, loose	80	1.3	—	—	—
moist, packed	100	1.6	—	—	—
mud, flowing	105	1.7	—	—	—
mud, packed	110	1.8	—	—	—
Feldspar	160-170	2.56-2.73	—	—	—
Flint	164	2.63	—	—	—
Fluorine	0.1059	0.00169	-365	-305	0.182
Gasoline	41-47	0.66-0.75	—	160-190	0.500

PHYSICAL PROPERTIES OF SELECTED MATERIALS

MATERIAL	UNIT WEIGHT Lb/Cu Ft	SPECIFIC GRAVITY*	MELTING POINT °F	BOILING POINT °F	SPECIFIC HEAT (CONST. PRESSURE FOR GASES)
Glass	153-172	2.45-2.75	—	—	0.16-0.20
Glycerin	78	1.26	64.6	554	0.576
Gneiss	150-170	2.4-2.7	—	—	0.18
Granite	165-170	2.65-2.7	—	—	0.192
Gravel:					
dry, loose	85-105	2.5-2.7	—	—	—
dry, packed	100-120	2.5-2.7	—	—	—
Gypsum	145	2.33	—	—	0.259
Helium	0.0111	0.00017	-458	-452	1.25
		8			
Hydrogen	0.0056	0.00009	-434	-423	3.39
		0			
Ice	55-57	0.88-0.92	—	—	—
Iridium	1400	22.42	4450	9600	0.032
Iron	490	7.86	2800	5430	0.11
Ivory	114-120	1.83-1.92	—	—	—
Lead	708	11.34	621.3	2950	0.031
Leather	54-64	0.86-1.02	—	—	—
Limestone	130-180	2.1-2.86	—	—	—
Linseed oil	58	0.934	-4	550	—
Magnesium	109	1.74	1200	2030	0.249
Manganese	450	7.2	2300	3900	0.115
Marble	160-180	2.6-2.86	—	—	0.21
Mercury	845	13.55	-38.0	673	0.033
Methane	0.0448	0.00071	-300	-258.5	0.593
		7			
Mica	165-200	2.7-3.1	—	—	0.21
Molybdenum	636	10.2	4750	6690	0.06
Neon	0.0562	0.00090	-415.6	-410.6	—
		0			
Nickel	555	8.9	2650	5250	0.11
Nitrogen	0.0781	0.00125	-345.75	-320.4	0.244
Osmium	1405	22.48	4400	9900	0.031
Oxygen	0.0892	0.00142	-361.1	-297.4	0.218
		9			
Paper	44-72	0.7-1.15	—	—	—
Petroleum	54	0.87	—	—	0.50
Porcelain	143-156	2.3-2.5	—	—	0.26
Propane	0.1261	0.00201	-310	-44	—
Pumice	23-56	0.37-0.90	—	—	—
Quartz	160-170	2.6-2.7	—	—	0.17
Salt	49-78	0.78-1.25	—	—	—
Sand:					
dry	90-110	2.4-2.7	—	—	—
wet	118-129	2.4-2.7	—	—	—
Slate	160-205	2.6-3.3	—	—	—
Sulfur	129	2.07	246	832	0.18
Tar, bituminous	64	1.02	—	—	—
Tin	456	7.3	449	4100	0.055
Titanium	281	4.5	3270	> 5400	0.126
Tungsten	1204	19.3	6120	10,650	0.032
Turpentine	54	0.873	14	320	0.411
Vanadium	372	5.96	3110	5430	0.12
Water:					
fresh	62.43	1.00000	32	212	1.000
sea	63.5-64.2	1.02-1.03	—	—	—
Zinc	445	7.14	787	1665	0.093

\*Specific gravity measured at 32°F and 1 atmosphere

September 30, 2001

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Coastal Models:</b>				
Coastal Storm Surges	FEMA Surge (1988)	Tetra Tech, Inc.; Engineering Methods & Applications; Greenhorne & O'Mara; Camp, Dresser & McKee, Inc.	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031	Incorporates modified NWS-23 model for hurricanes and Joint Probability Method. Reportedly more accurate for water elevations than water currents.
	ODISTIM (1975)	Coastal Consultants, Inc.	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031	Computes wind-driven surges propagating in estuaries or rivers described by one-dimensional elements, but only for northeasters since wind direction is fixed.
	Northeast Model (1978)	Stone & Webster Engineering Corp.	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031	Accommodates asymmetrical geometry for extratropical storms but requires separate program to compute resultant coastal surge for winds and pressures.
	FLOW2D (1975) <sup>1</sup>	Resource Analysis, Inc.	Camp, Dresser, & McKee, Inc. Ten Cambridge Center Cambridge, MA 02142	Unsteady flood flow for estuaries and floodplains, but no direct wind effects are considered.
	TABS RMA2 v. 4.3 (October 1996)	U.S. Army Corps of Engineers	Coastal Engineering Research Center Department of the Army Waterways Experiment Station Corps of Engineers 3909 Halls Ferry Road Vicksburg, MS 39180-6199	Two-dimensional steady/unsteady flow model, for water levels and velocities. Computes finite element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows.
	MIKE 21 HD/NHD	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	Solves the non-linear depth-averaged equations of continuity and conservation of momentum. Computes water levels and flows based on a variety of forcing functions. Can include wave-driven currents and setup. Uses a finite difference grid with dynamic nesting grid capabilities. Directly resolving small scale features such as narrow inland channels may result in a large computational costs.
Coastal Wave Heights	DYNLET	U.S. Army Corps of Engineers	Coastal and Hydraulics Laboratory Engineering Research and Development Center 3909 Halls Ferry Road Vicksburg, MS 39180-6199	One-dimensional model of dynamic behavior of tidal flow at inlets. Can be used to predict tide dominated velocities and water level fluctuations at an inlet and interior back bay system. DYNLET solves the full one-dimensional shallow water equations using an implicit finite difference solution.
	WHAFIS 3.0 (1988)	Dames & Moore, revised by Greenhorne & O'Mara	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031	Defines wave heights associated with 100-year flood in coastal areas using modern wave action treatment; incorporates 1977 NAS recommendations on basic approximations for wind speeds, wave breaking criterion, and controlling wave height.
	WHAFIS 3.0 GL (1993)	Dames & Moore, Greenhorne & O'Mara, Dewberry & Davis LLC	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031	Identical wave treatments as WHAFIS 3.0, but with programmed reduction of wind speeds for U.S. shorelines of the Great Lakes.

<sup>1</sup>This model is acceptable for coastal storm surge applications only.



# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Coastal Models (cont'd):</b>				
Coastal Wave Heights	RCPWAVE (1986)	U.S. Army Corps of Engineers	Coastal Engineering Research Center Department of the Army Waterways Experiment Station Corps of Engineers 3909 Halls Ferry Road Vicksburg, MS 39180-6199	Treats linear, monochromatic waves propagating over grid giving coastal bathymetry, providing nearshore wave heights pertinent to proper spacing between transects or to magnitudes of wave setup.
	CHAMP 1.0 (2001)	Dewberry & Davis LLC	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031 <a href="http://www.fema.gov/nfip/tsd/frm_soft.htm">http://www.fema.gov/nfip/tsd/frm_soft.htm</a>	Coastal Hazard Analysis Modeling Program (CHAMP) is a Windows-based program used for wave height analyses (enhances WHAFIS 3.0) and provides summary tables and graphics for mapping.
	MIKE 21 Offshore Spectral Wave Model (OSW)	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	Two-dimensional dynamic wind-wave growth model. Discretizes spectrum in frequency and direction. Does not include breaking. Appropriate for providing regional wave conditions in deep and intermediate depths, providing boundary data for nearshore modeling or basic analysis.
	MIKE 21 Nearshore Spectral Wave Model (NSW)	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	Two-dimensional stationary model for propagation of waves into the nearshore zone (refraction, shoaling, breaking, bed friction, and wind-wave growth). Based on the conservation equation for the spectral wave action density; similar to H1SWA model. Obstructions not directly resolvable in the grid, must be modeled with grid bed roughness coefficients.
	RUNUP 2.0 (1990)	Stone & Webster Engineering Corp., revised by Dewberry & Davis	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031	Executes 1978 guidance by USACE defining wave runup on shore barrier with specified approach and storm conditions; mean wave description determines mean runup elevation.
Coastal Wave Effects	GLWRM (1992)	U.S. Army Corps of Engineers	Department of the Army Detroit District, Corps of Engineers 477 Michigan Avenue Detroit, MI 48266	Developed particularly to analyze wave runup for the three types of situations most frequently encountered on U.S. shorelines of the Great Lakes: sand beach, sloping riprap revetment, and vertical wall.
	EROSION (1988)	Dewberry & Davis LLC	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031	Compares sand dune cross section to 540 ft <sup>2</sup> rule and computes eroded geometry for 100-year flood as either dune face retreat or dune removal by optional methods. Presently needs revisions to improve model and to provide graphics.
	ACES 1.07 (1992)	U.S. Army Corps of Engineers	Coastal Engineering Research Center Department of the Army Waterways Experiment Station Corps of Engineers 3909 Halls Ferry Road Vicksburg, MS 39180-6199	Used for restricted fetch wave growth analysis and runup on vertical structures or revetments.
	CHAMP 1.0 (2001)	Dewberry & Davis LLC	Dewberry & Davis LLC, METS Division 8401 Arlington Blvd. Fairfax, VA 22031 <a href="http://www.fema.gov/nfip/tsd/frm_soft.htm">http://www.fema.gov/nfip/tsd/frm_soft.htm</a>	CHAMP is a Windows-based program used for storm-induced erosion treatments (enhances EROSION) and wave runup analyses (enhances RUNUP 2.0), and provides summary tables and graphics for mapping.

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Hydrologic Models: Determination of Flood Hydrographs</b>				
Single Event	HEC-1 4.0.1 and up <sup>2</sup> (May 1991)	U.S. Army Corps of Engineers	Water Resources Support Center <sup>3</sup> Corps of Engineers Hydrologic Engineering Center (HEC) 609 Second Street Davis, CA 95616-4687	Flood hydrographs at different locations along streams. Calibration runs preferred to determine model parameters.
	HEC-HMS 1.1 and up (March 1998)	U.S. Army Corps of Engineers	U.S. Army Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616-4687 <a href="http://www.hec.usace.army.mil/">http://www.hec.usace.army.mil/</a>	The Hydrologic Modeling System provides a variety of options for simulating precipitation-runoff processes. It has a capability to use gridded rainfall data to simulate runoff. It does not provide snowmelt and snowfall functions; it cannot be used for areas where snowmelt is an important flood hazard source and must be considered in estimation of flood discharges.
	TR-20 (February 1992)	U.S. Department of Agriculture, Natural Resources Conservation Service	U.S. Department of Commerce National Technical Information Service 5285 Port Royal Road Springfield, VA 22161	Flood hydrographs at different locations along streams. Calibration runs preferred to determine model parameters.
	TR-55 (June 1986)	U.S. Department of Agriculture, Natural Resources Conservation Service	U.S. Department of Commerce National Technical Information Service 5285 Port Royal Road Springfield, VA 22161 <a href="http://www.wcc.nrcs.usda.gov/water/quality/common/tr55/tr55.html">http://www.wcc.nrcs.usda.gov/water/quality/common/tr55/tr55.html</a>	Peak discharges and flood hydrographs at a single location.
	SWMM (RUNOFF) 4.30 (May 1994), and 4.31 (January 1997)	U.S. Environmental Protection Agency and Oregon State University	Center for Exposure Assessment Modeling U.S. Environmental Protection Agency Office of Research and Development Environmental Research Laboratory 960 College Station Road Athens, GA 30605-2720 <a href="http://www.epa.gov/ceampub/softwodos.htm">http://www.epa.gov/ceampub/softwodos.htm</a> Department of Civil, Construction, and Environmental Engineering Oregon State University 202 Apperson Hall Corvallis, OR 97331-2302 <a href="http://www.ccee.orst.edu/swmm/">http://www.ccee.orst.edu/swmm/</a> <a href="ftp://ftp.engr.orst.edu/pub/swmmr/jpc">ftp://ftp.engr.orst.edu/pub/swmmr/jpc</a>	Calibration or verification to the actual flood events highly recommended.

<sup>2</sup>The enhancement of these programs in editing and graphical presentation can be obtained from several private companies.  
<sup>3</sup>Program is typically distributed by vendors and may not be available through HEC. A list of vendors may be obtained through HEC.

September 30, 2001

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Hydrologic Models: Determination of Flood Hydrographs (cont'd)</b>				
Single Event	MIKE 11 UHM (June 1999)	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	Simulates flood hydrographs at different locations along streams using unit hydrograph techniques. Three methods are available for calculating infiltration losses and three methods for converting rainfall excess to runoff. The web page is at: <a href="http://www.dhi.dk">http://www.dhi.dk</a>
	DBRM 3.0 (*1993)	Bernard L. Golding, P.E. Consulting Water Resources Engineer Orlando, FL	Center for Microcomputers in Transportation (McTrans) University of Florida 512 Weil Hall Gainesville, FL 32611-6585	Flood hydrographs at different locations along streams. Calibration runs preferred to determine model parameters.
	HYMO	U.S. Department of Agriculture, Natural Resources Conservation Service	U.S. Department of Commerce National Technical Information Service 5285 Port Royal Road Springfield, VA 22161	Flood hydrographs at different locations along streams. Calibration runs preferred to determine model parameters.

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Hydrologic Models: Determination of Flood Hydrographs (cont'd)</b>				
Continuous Event	DR3M (October 1993)	U.S. Geological Survey	U.S. Geological Survey National Center 12201 Sunrise Valley Drive Reston, VA 22092	Calibration to actual flood events required. The web page is at: <a href="http://water.usgs.gov/software/surface_water.html">http://water.usgs.gov/software/surface_water.html</a>
	HSPF 10.10 and up (December 1993)	U.S. Environmental Protection Agency, U.S. Geological Survey	Center for Exposure Assessment Modeling U.S. Environmental Protection Agency Office of Research and Development Environmental Research Laboratory 960 College Station Road Athens, GA 30605-2720	Calibration to actual flood events required. The web page is at: <a href="http://water.usgs.gov/software/surface_water.html">http://water.usgs.gov/software/surface_water.html</a>
	MIKE 11 RR (June 1999)	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	The Rainfall-Runoff Module (RR, formerly NAM) is a lumped-parameter hydrologic model capable of continuously accounting for water storage in surface and sub-surface zones. Flood hydrographs are estimated at different locations along streams. Calibration to actual flood events is required. The web page is at: <a href="http://www.dhi.dk">http://www.dhi.dk</a>
Interior Drainage Analysis	HEC-FH 1.03 and up	U.S. Army Corps of Engineers	U.S. Army Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616-4687	Provides both continuous simulation and hypothetical event analyses. Coincidence frequency analysis (not included in the mode) may be needed for some cases.
<b>Statistical Models:</b>				
	HEC FFA 3.1 (February 1995)	U.S. Army Corps of Engineers	Water Resources Support Center <sup>3</sup> Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616-4687	Performs flood frequency analyses following <i>Bulletin 17B, Guidelines for Determining Flood Flow Frequency</i> , prepared by the Interagency Advisory Committee on Water Data (1982). Supersedes HEC-WRC.
	PEAKFQ 2.4 and up (April 1998)	U.S. Geological Survey	U.S. Geological Survey Hydrologic Analysis Software Support Team 437 National Center Reston, VA 20192 <a href="http://water.usgs.gov/software/surface_water.html">http://water.usgs.gov/software/surface_water.html</a>	Performs flood frequency analyses following <i>Bulletin 17B, Guidelines for Determining Flood Flow Frequency</i> , prepared by the Interagency Advisory Committee on Water Data (1982).
	FAN	FEMA	Michael Baker, Jr., Inc. 3601 Eisenhower Avenue, Suite 600 Alexandria, VA 22304	Determines depth and velocity zones over alluvial fans.

<sup>3</sup>Program is typically distributed by vendors and may not be available through HEC. A list of vendors may be obtained through HEC.

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

September 30, 2001

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
One-dimensional Models	HEC-RAS 2.2 (September 1998)	U.S. Army Corps of Engineers	Water Resources Support Center Corps of Engineers Hydrologic Engineering Center (HEC) 609 Second Street Davis, CA 95616-4687 <a href="http://www.hec.usace.army.mil/">http://www.hec.usace.army.mil/</a>	A HEC-2 file can be imported into HEC-RAS; the user must change the conveyance computations in HEC-RAS and make the necessary modifications to the bridge modeling before running HEC-RAS to duplicate the results obtained using HEC-2. The use of HEC-RAS for restudying a stream previously studied using HEC-2 is encouraged, as long as one of the following conditions is met: 1) the entire stream is rerun using HEC-RAS; or 2) the stream reach remodeled using HEC-RAS is hydraulically independent from the rest of the stream. The WSPRO bridge analysis is recommended for constructed floodplains under subnormal flow conditions. In addition, HEC-RAS version 2.2 that performs the steady flow water-surface profile calculations (SNET) has been updated to version 2.2.1; it should be used for NFIP studies.
	HEC-RAS 3.0	U.S. Army Corps of Engineers	Water Resources Support Center Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616-4687	Under rare circumstances, for bridges with low flow, and weir flow on the overbanks, HEC-RAS 3.0 may not be able to balance the flow using weir flow equation and low flow bridge analysis methods. HEC-RAS 3.0 will then use the energy method, and the computed energy grade elevations and water-surface elevations may be on the high side.
	HEC-2 4.6.2 <sup>2</sup> (May 1991)	U.S. Army Corps of Engineers	Water Resources Support Center <sup>3</sup> Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616-4687	Includes culvert analysis and floodway options.
	WSPRO (June 1988 and up)	U.S. Geological Survey, Federal Highway Administration (FHWA)	Federal Highway Administration (FHWA) web page at: <a href="http://www.fhwa.dot.gov/bridge/hyddescr.htm">http://www.fhwa.dot.gov/bridge/hyddescr.htm</a>	Floodway option is available in June 1998 version. 1988 version is available on the USGS web page at: <a href="http://water.usgs.gov/software/surface_water.html">http://water.usgs.gov/software/surface_water.html</a>
	WSP2 (October 1993)	U.S. Department of Agriculture, Natural Resources Conservation Service	U.S. Department of Commerce National Technical Information Service 5285 Port Royal Road Springfield, VA 22161	October 1993 version includes an encroachment option. Encroachment stations may be developed using FLDWY model (described below).
	FLDWY (May 1989)	U.S. Department of Agriculture, Natural Resources Conservation Service	U.S. Department of Commerce National Technical Information Service 5285 Port Royal Road Springfield, VA 22161	Determines the encroachment stations from equal conveyance reduction method; used in conjunction with WSP2. Encroachment stations developed using this model must be re-entered in WSP2 model to properly develop floodway.
	QUICK-2 1.0 and up (January 1995)	FEMA	Federal Emergency Management Agency Hazard Identification Branch Mitigation Directorate 500 C Street, SW Washington, DC 20472	Intended for use in areas studied by approximate methods (Zone A) only. May be used to develop water-surface elevations at one cross section or a series of cross sections. May not be used to develop a floodway.
	HY8 4.1 and up (November 1992)	U.S. Department of Transportation, Federal Highway Administration (FHWA)	Federal Highway Administration (FHWA) web page at: <a href="http://www.fhwa.dot.gov/bridge/hyddescr.htm">http://www.fhwa.dot.gov/bridge/hyddescr.htm</a>	Computes water-surface elevations for flow through multiple parallel culverts and over the road embankment. Software and related publication are available from Center for Microcomputers in Transportation (McTrans), University of Florida, 512 Weill Hall, Gainesville, FL 32611-6585; and on the web at: <a href="http://www.mctrans.ce.ufl.edu/">http://www.mctrans.ce.ufl.edu/</a>
	WSPGW 12.96 (October 2000)	Los Angeles Flood Control District and Joseph E. Bonadiman & Associates, Inc.	Joseph E. Bonadiman & Associates, Inc. 588 West 6th Street San Bernardino, CA 92410 <a href="http://www.bonadiman.com">http://www.bonadiman.com</a>	Windows version of WSPGW. Computes water-surface profiles and pressure gradients for open channels and closed conduits. Can analyze multiple parallel pipes. Road overtopping cannot be computed. Open channels are analyzed using the standard step method but roughness coefficient can not vary across the channel. Overbank analyses cannot be done. Multiple parallel pipe analysis assumes equal distribution between pipes so pipes must be of similar material, geometry, slope, and inlet configuration. Floodway function is not available. Demo version available from: <a href="http://www.civildesign.com">http://www.civildesign.com</a>

<sup>2</sup>The enhancement of these programs in editing and graphical presentation can be obtained from several private companies.  
<sup>3</sup>Program is typically distributed by vendors and may not be available through HEC. A list of vendors may be obtained through HEC.

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

September 30, 2001

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Hydraulic Models: Determination of Water-surface Elevations for Riverine Analysis (cont'd)</b>				
One-dimensional Unsteady Flow Models	FEQ 8.92 and FEQUTL 4.68 (1997, both)	Delbert D. Franz, Linsley Kraeger Associates; and Charles S. Melching, USGS	U.S. Geological Survey 221 North Broadway Avenue Urbana, IL 61801 <a href="http://water.usgs.gov/software/surface_water.html">http://water.usgs.gov/software/surface_water.html</a> and technical support available at <a href="http://www-1.usgs.gov/proj/feq/">http://www-1.usgs.gov/proj/feq/</a>	The FEQ model is a computer program for the solution of full, dynamic equations of motion for one-dimensional unsteady flow in open channels and control structures. The hydraulic characteristics for the floodplain (including the channel, overbanks, and all control structures affecting the movement of flow) are computed by its companion program FEQUTL and used by the FEQ program. Calibration or verification to the actual flood events highly recommended. Type 5 culvert flow computations of FEQUTL need verification with results obtained using methodology or models accepted for NFIP use. Floodway concept formulation is unavailable.
	Advanced ICPR 2.20 (October 2000)	Streamline Technologies, Inc.	8961 University Boulevard Winter Park, FL 32792	Previous versions of the model are not acceptable. Calibration or verification to the actual flood events highly recommended. Floodway concept formulation unavailable.
	DAMBRK	National Weather Service	Hydrologic Research Laboratory Office of Hydrology National Weather Service, NOAA 1345 East-West Highway Silver Spring, MD 20910	Calibration runs required to determine roughness coefficients. More review anticipated for flow through structures. Floodway concept formulation unavailable. The National Weather Service is no longer supporting the program; it should not be used for new studies.
	SWMM 4.30 (May 1994), and 4.31 (January 1997)	U.S. Environmental Protection Agency and Oregon State University	Center for Exposure Assessment Modeling U.S. Environmental Protection Agency Office of Research and Development Environmental Research Laboratory 960 College Station Road Athens, GA 30605-2720 <a href="http://www.epa.gov/ceam/pub/softwdos.htm">http://www.epa.gov/ceam/pub/softwdos.htm</a> Department of Civil, Construction, and Environmental Engineering Oregon State University 202 Apperson Hall Corvallis, OR 97331-2302 <a href="http://www.ccae.orst.edu/swmm/">http://www.ccae.orst.edu/swmm/</a> <a href="http://ftp.engr.orst.edu/pub/swmm/pc">http://ftp.engr.orst.edu/pub/swmm/pc</a>	Calibration or verification to the actual flood events highly recommended. Structural loss calculations unavailable and must be accommodated via roughness factor manipulation. Floodway concept formulation unavailable. Preferably, for NFIP purposes, head losses at bridges should be verified using WSPRO; losses at culverts should be verified using the U.S. Geological Survey's six equations for culvert analysis. Losses at storm sewer junctions should also be verified with separate calculations; contact FEMA for guidance with these calculations.
	NETWORK (DWOPER)	National Weather Service	Hydrologic Research Laboratory Office of Hydrology National Weather Service, NOAA 1345 East-West Highway Silver Spring, MD 20910	Less flexible than DAMBRK in specifying Mannings "n" values. Calibration to actual flood events required. Floodway concept formulation unavailable. The National Weather Service is no longer supporting the program; it should not be used for new studies.
	UNET 4.0 (April 2001)	U.S. Army Corps of Engineers	Water Resources Support Center Corps of Engineers Hydrologic Engineering Center (HEC) 609 Second Street Davis, CA 95616-4687	Calibration or verification to the actual flood events highly recommended. Comparison of bridge and culvert modeling to other numerical models reveals significant differences in results; these differences may be investigated in the near future. Floodway option is not accepted for NFIP usage.
	FLDWAV (November 1998)	National Weather Service	Hydrologic Research Laboratory Office of Hydrology National Weather Service, NOAA 1345 East-West Highway Silver Spring, MD 20910	Includes all the features of DAMBRK and DWOPER plus additional capabilities. It is a computer program for the solution of the fully dynamic equations of motion for one-dimensional flow in open channels and control structures. Floodway concept formulation is unavailable. Calibration to actual flood events required. This model has the capability to model sediment transport. Program is supported by NWS.
	MIKE 11 HD (June 1999)	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	Hydrodynamic model for the solution of the fully dynamic equations of motion for one-dimensional flow in open channels and control structures. The floodplain can be modeled separately from the main channel. Bridge flow computations need verification with results obtained using methodologies or models accepted for NFIP usage. Calibration to actual flood events required. Floodway concept formulation is unavailable. This model has the capability to model sediment transport. The web page is at: <a href="http://www.dhi.dk">http://www.dhi.dk</a>
	FLO-2D v. 2000.11 (December 2000)	Jimmy S. O'Brien, Ph.D., P.E.	FLO-2D Software, Inc. Tetra Tech, ISG P.O. Box 66 Nurtroso, AZ 85932	Hydrodynamic model for the solution of the fully dynamic equations of motion for one-dimensional flow in open channels and two-dimensional flow in the floodplain. Bridge or culvert computations must be accomplished external to FLO-2D using methodologies or models accepted for NFIP usage. Calibration to actual flood events required. Floodway computation is unavailable.

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Hydraulic Models: Determination of Water-surface Elevations for Riverine Analysis (cont'd)</b>				
Two-dimensional Steady/Unsteady Flow Models	TABS -RMA2 v. 4.3 (October 1996) -RMA4 v. 4.5 (July 2000)	U.S. Army Corps of Engineers	Coastal Engineering Research Center Department of the Army Waterways Experiment Station Corps of Engineers 3909 Halls Ferry Road Vicksburg, MS 39180-6199	Limitations on split flows. Floodway concept formulation unavailable. More review anticipated for treatment of structures.
	FESWMS 2DH 1.1 and up (June 1995)	U.S. Geological Survey	U.S. Geological Survey National Center 12201 Sunrise Valley Drive Reston, VA 22092 <a href="http://water.usgs.gov/software/surface_water.html">http://water.usgs.gov/software/surface_water.html</a>	Region 10 has conducted study in Oregon. Floodway concept formulation unavailable. This model has the capability to model sediment transport.
	FLO-2D v. 2000.11 (December 2000)	Jimmy S. O'Brien, Ph.D., P.E.	FLO-2D Software, Inc. Tetra Tech, ISG P.O. Box 66 Nutrioso, AZ 85932	Hydrodynamic model that has the capabilities of modeling unconfined flows, complex channels, sediment transport, and mud and debris flows. It can be used for alluvial fan modeling.
Floodway Analysis	SFD	U.S. Army Corps of Engineers/FEMA	Federal Emergency Management Agency Hazard Identification Branch Mitigation Directorate 500 C Street, SW Washington, DC 20472	Simplified floodway procedure for streams with no regulatory floodway limits.
	PSUPRO	Pennsylvania State University/ U.S. Army Corps of Engineers/FEMA	Federal Emergency Management Agency Hazard Identification Branch Mitigation Directorate 500 C Street, SW Washington, DC 20472	Encroachment analysis for streams with no regulatory floodway limits.

September 30, 2001

# Numerical Models Accepted by FEMA for NFIP Usage (Nationally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Sediment Transport Models: Determination of Sediment Transport for Riverine Analysis<sup>1</sup> (sediment modeling must reflect current conditions only)</b>				
	HEC-6 4.1 (August 1993)	U.S. Army Corps of Engineers	U.S. Army Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616	One-dimensional movable boundary open-channel flow model for simulating and predicting changes in river profiles resulting from scour and deposition. The model was designed to simulate changes over moderate time periods such as years but applications to single flood events are possible. Single flood event analyses should be performed with caution. The program and documentation are available at the web page: <a href="http://www.fhec.usace.army.mil/software/software_distrib/index.html">http://www.fhec.usace.army.mil/software/software_distrib/index.html</a>
	MIKE 11 (June 1999)	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	Computes transport of non-cohesive and cohesive sediments. For non-cohesive sediment transport, two modes of computation are available. Sediment transport rates and accumulated volumes of deposition and erosion are reported at designated cross sections (explicit mode) or the bed level of the channel is updated through the sediment continuity equation and feedback with the hydrodynamic module (morphological mode). For cohesive sediment transport, the streambed is modeled with multiple layers with separate sediment properties for each layer. The web page is at: <a href="http://www.dhi.dk">http://www.dhi.dk</a>
	TABS -SED2D 4.5 (June 2000)	U.S. Army Corps of Engineers	Coastal Engineering Research Center Department of the Army Waterways Experiment Station Corps of Engineers 3909 Halls Ferry Road Vicksburg, MS 39180-6199	Two-dimensional model for depth-averaged transport of cohesive- or a representative grain size of noncohesive-sediments and their deposition, erosion, and formation of bed deposits.

<sup>1</sup>Sediment transport analysis capabilities are features in the following hydraulic models: FLO-2D, FESWMS, and FLDWAV.



# Numerical Models Accepted by FEMA for NFIP Usage (Locally Accepted Models)

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
<b>Hydrologic Models: Determination of Flood Hydrographs</b>				
Single Event	AHYMO 97 (August 1997)	Albuquerque Metropolitan Arroyo Flood Control Authority, Anderson-Hydro	Anderson-Hydro 13537 Terragon Drive, NE Albuquerque, NM 87112	Flood hydrographs at different locations along streams. <b>Only accepted for usage and the default parameters in the model applicable within New Mexico.</b>
	Colorado Urban Hydrograph Procedure (CUHFP/PC) (May 1996)	Denver Urban Drainage and Flood Control District	Denver Urban Drainage and Flood Control District 2480 West 26th Avenue, Suite 156-B Denver, CO 80211	Flood hydrographs at different locations along streams. Hydrographs are routed using UDSWMM2-PC (a modified version of the Runoff Block of EPA's SWMM). <b>Only accepted for usage and the default parameters in the model applicable within the Denver, Colorado, metro area.</b>
<b>Hydraulic Models: Determination of Water-surface Elevations for Riverine Analysis</b>				
One-dimensional Unsteady Flow Models	HCSWMM 4.31B (August 2000)	Stormwater Management Section Public Works Department Hillsborough County, Florida	Stormwater Management Section Public Works Department Hillsborough County, Florida 601 E. Kennedy Boulevard, 21 <sup>st</sup> Floor P.O. Box 1110 Tampa, FL 33601	Modified version of EPA SWMM 4.31. The major modifications are: integrated the SCS-CN method into the model to calculate the rainfall-runoff process; allow up to 21 different Manning's coefficients for each cross-section; added 4 more fields to C1 line to calculate the exit, entrance, and other minor losses, and to stretch the pipe based on stability condition automatically create an ASCII file, HYDROG.DAT, containing hydrograph for each subbasin generated after each run. <b>Only accepted for usage and applicable within Hillsborough County, Florida.</b> The web page is at: <a href="http://www.hillsboroughcounty.org/publicworks/engineering/home.html">http://www.hillsboroughcounty.org/publicworks/engineering/home.html</a>
Two-dimensional Unsteady Flow Models	SHEET2D 9 (July 2000)	Tomasello Consulting Engineers, Inc.	Tomasello Consulting Engineers, Inc. 5906 Center Street Jupiter, FL 33458	DOS program applied to a grid network presenting the topography and hydrologic parameters of the watershed. Computes runoff for applied rainfall distribution for each grid based on SCS formula and input soil and depression storage. Routes overland flow via two-dimensional dynamic equations. Channel routings performed with special grids with assigned HEC-2 type cross sections. Sheetflow barriers used to represent dikes and roads separating sheetflow grids. Special grids are used to define cascading water management systems with stage/storage and hydrologic parameter inputs that connect to the sheetflow regime via structures described in input. Outputs include high water elevation, discharge from structures, sheetflow hydrographs, and stage hydrographs for any point in the model. <b>Only accepted for usage within Big Cypress Basin, Florida.</b>
Two-dimensional Steady/Unsteady Flow Models	DHM 21 (August 1987)	Theodore V. Hromacka II, Ph.D., P.E., P.H.	Department of Mathematics, Geology, and Environmental Studies California State University, Fullerton Fullerton, CA 92958-9020 thromacka@full.com	Diffusion flow model which can route unconfined surface and open channel flows. Can be used to model alluvial flooding. Rainfall-runoff output can be used for hydrologic studies. Kinematic routing optional. Floodway concept formulation unavailable. <b>Only accepted for usage within the San Bernardino County Flood Control District, California.</b>
<b>Sediment Transport Models: Determination of Sediment Transport for Riverine Analysis (sediment modeling must reflect current conditions only)</b>				
	AHYMO 97 (August 1997)	Albuquerque Metropolitan Arroyo Flood Control Authority, Anderson-Hydro	Anderson-Hydro 13537 Terragon Drive, NE Albuquerque, NM 87112	Computes sediment wash load using the Modified Universal Soil Loss Equation and bed load using power function relations. The values of computed hydrographs are multiplied by factors to simulate the effects of sediment bulking. <b>Only accepted for usage and the default parameters in the model applicable to channels within New Mexico.</b>

## **MINIMUM STANDARD DETAIL REQUIREMENTS FOR ALTA/ACSM LAND TITLE SURVEYS**

**as adopted by  
American Land Title Association  
American Congress on Surveying & Mapping  
and  
National Society of Professional Surveyors**

It is recognized that members of the American Land Title Association (ALTA) have specific needs, peculiar to title insurance matters, which require particular information for acceptance by title insurance companies when said companies are asked to insure title to land without exceptions as to the many matters which might be discoverable from survey and inspection and not be evidenced by the public records. In the general interest of the public, the surveying profession, title insurers and abstractors, ALTA, the American Congress on Surveying and Mapping (ACSM) and the National Society of Professional Surveyors, Inc. (NSPS) jointly promulgate and set forth such details and criteria for standards. It is understood that local variations may require local adjustments to suit local situations, and often must be applied. It is recognized that title insurance companies are entitled to rely on the survey furnished to them being of the appropriate professional quality, both as to completeness and as to accuracy. It is equally recognized that for the performance of a survey, the surveyor will be provided with appropriate data which can be relied upon in the preparation of the survey.

For a survey of real property and the plat or map of the survey to be acceptable to a title insurance company for purposes of insuring title to said real property free and clear of survey matters (except those matters disclosed by the survey and indicated on the plat or map), certain specific and pertinent information shall be presented for the distinct and clear understanding between the client (insured), the title insurance company (insurer), and the surveyor (the person professionally responsible for the survey). These requirements are:

1. The client shall request the survey or arrange for the survey to be requested and shall provide a written authorization to proceed with the survey from the person responsible for paying for the survey. Unless specifically authorized in writing by the insurer, the insurer shall not be responsible for any costs associated with the preparation of the survey. The request shall specify that an "**ALTA/ACSM LAND TITLE SURVEY**" is required, meeting the then-current accuracy standards jointly adopted by ALTA, ACSM and NSPS. The request shall also designate which of the optional items listed in Table A are to be incorporated. The request shall set forth the record description of the property. Complete copies of the record description of the property, any record easements benefiting the property, the record easements or servitudes and covenants affecting the property ("Record Documents"), documents of record referred to in the Record Documents, and any other documents containing desired appropriate information affecting the property being surveyed and to which the survey shall make reference shall be provided to the surveyor for notation on the plat or map of survey.
2. The plat or map of such survey shall bear the name, address, telephone number, and signature of the professional land surveyor who made the survey, his or her official seal and registration number, the date the survey was completed and the dates of all revisions, and the caption "**ALTA/ACSM Land Title Survey**" with the certification set forth in paragraph 8.
3. An "**ALTA/ACSM LAND TITLE SURVEY**" shall be in accordance with the then-current "Accuracy Standards for Land Title Surveys" ("Accuracy Standards") as adopted, from time to time, by the American Congress on Surveying and Mapping, the National Society of Professional Surveyors, and the American Land Title Association and incorporated herein by reference.
4. On the plat or map of an "**ALTA/ACSM LAND TITLE SURVEY**," the survey boundary shall be drawn to a convenient scale, with that scale clearly indicated. A graphic scale, shown in feet or meters or both, shall be included. A north arrow shall be shown and when practicable, the plat or map of survey shall be oriented so that north is at the top of the drawing. Symbols or abbreviations used shall be identified on the face of the plat or map by use of a legend or other means. If necessary for clarity, supplementary or exaggerated diagrams shall be presented accurately on the plat or map. The plat or map shall be a minimum size of 8½ by 11 inches.
5. The survey shall be performed on the ground and the plat or map of an "**ALTA/ACSM LAND TITLE SURVEY**" shall contain, in addition to the required items already specified above, the following applicable information:
  - (a) All data necessary to indicate the mathematical dimensions and relationships of the boundary represented, with angles given directly or by bearings, and with the length and radius of each curve, together with elements necessary to mathematically define each curve. The point of beginning of the surveyor's description shall be shown as well as the remote point of beginning if different. A bearing base shall refer to some well-fixed bearing line, so that the bearings may be easily re-established. All bearings around the boundary shall read in a clockwise direction wherever possible. The North arrow shall be referenced to its bearing base and should that bearing base differ from record title, that difference shall be noted.
  - (b) When record bearings or angles or distances differ from measured bearings, angles or distances, both the record and measured bearings, angles, and distances shall be clearly indicated. If the record description fails to form a

### **Exhibit 5**

Page 1

mathematically closed figure, the surveyor shall so indicate.

- (c) Measured and record distances from corners of parcels surveyed to the nearest right-of-way lines of streets in urban or suburban areas, together with recovered lot corners and evidence of lot corners, shall be noted. The distances to the nearest intersecting street shall be indicated and verified. Names and widths of streets and highways abutting the property surveyed and widths of rights of way shall be given. Any use contrary to the above shall be noted. Observable evidence of access (or lack thereof) to such abutting streets or highways shall be indicated. Observable evidence of private roads shall be so indicated. Streets abutting the premises, which have been described in Record Documents, but not physically opened, shall be shown and so noted.
- (d) The identifying titles of all recorded plats, filed maps, right of way maps, or similar documents which the survey represents, wholly or in part, shall be shown with their appropriate recording data, filing dates and map numbers, and the lot, block, and section numbers or letters of the surveyed premises. For non-platted adjoining land, names, and recording data identifying adjoining owners as they appear of record shall be shown. For platted adjoining land, the recording data of the subdivision plat shall be shown. The survey shall indicate platted setback or building restriction lines which have been recorded in subdivision plats or which appear in a Record Document which has been delivered to the surveyor. Contiguity, gores, and overlaps along the exterior boundaries of the surveyed premises, where ascertainable from field evidence or Record Documents, or interior to those exterior boundaries, shall be clearly indicated or noted. Where only a part of a recorded lot or parcel is included in the survey, the balance of the lot or parcel shall be indicated.
- (e) All evidence of monuments shall be shown and noted to indicate which were found and which were placed. All evidence of monuments found beyond the surveyed premises on which establishment of the corners of the surveyed premises are dependent, and their application related to the survey shall be indicated.
- (f) The character of any and all evidence of possession shall be stated and the location of such evidence carefully given in relation to both the measured boundary lines and those established by the record. An absence of notation on the survey shall be presumptive of no observable evidence of possession.
- (g) The location of all buildings upon the plot or parcel shall be shown and their locations defined by measurements perpendicular to the boundaries. If there are no buildings erected on the property being surveyed, the plat or map shall bear the statement, "No buildings." Proper street numbers shall be shown where available.
- (h) All easements evidenced by a Record Document which have been delivered to the surveyor shall be shown, both those burdening and those benefiting the property surveyed, indicating recording information. If such an easement cannot be located, a note to this effect shall be included. Observable evidence of easements and/or servitudes of all kinds, such as those created by roads; rights-of-way; water courses; drains; telephone, telegraph, or electric lines; water, sewer, oil or gas pipelines on or across the surveyed property and on adjoining properties if they appear to affect the surveyed property, shall be located and noted. If the surveyor has knowledge of any such easements and/or servitudes, not observable at the time the present survey is made, such lack of observable evidence shall be noted. Surface indications, if any, of underground easements and/or servitudes shall also be shown.
- (i) The character and location of all walls, buildings, fences, and other visible improvements within five feet of each side of the boundary lines shall be noted. Without expressing a legal opinion, physical evidence of all encroaching structural appurtenances and projections, such as fire escapes, bay windows, windows and doors that open out, flue pipes, stoops, eaves, cornices, areaways, steps, trim, etc., by or on adjoining property or on abutting streets, on any easement or over setback lines shown by Record Documents shall be indicated with the extent of such encroachment or projection. If the client wishes to have additional information with regard to appurtenances such as whether or not such appurtenances are independent, division, or party walls and are plumb, the client will assume the responsibility of obtaining such permissions as are necessary for the surveyor to enter upon the properties to make such determinations.
- (j) Driveways and alleys on or crossing the property must be shown. Where there is evidence of use by other than the occupants of the property, the surveyor must so indicate on the plat or map. Where driveways or alleys on adjoining properties encroach, in whole or in part, on the property being surveyed, the surveyor must so indicate on the plat or map with appropriate measurements.
- (k) As accurately as the evidence permits, the location of cemeteries and burial grounds (i) disclosed in the process of researching title to the premises or (ii) observed in the process of performing the field work for the survey, shall be shown.
- (l) Ponds, lakes, springs, or rivers bordering on or running through the premises being surveyed shall be shown.

6. As a minimum requirement, the surveyor shall furnish two sets of prints of the plat or map of survey to the title insurance company or the client. If the plat or map of survey consists of more than one sheet, the sheets shall be numbered, the total number of sheets indicated and match lines be shown on each sheet. The prints shall be on durable and dimensionally stable material of a quality standard acceptable to the title insurance company. At least two copies of the boundary description prepared from the survey shall be similarly furnished by the surveyor and shall be on the face of the plat or map of survey, if practicable, or otherwise attached to and incorporated in the plat or map. Reference to date of the "ALTA/ACSM LAND TITLE SURVEY," surveyor's file number (if any), political subdivision, section, township and range, along with appropriate aliquot parts thereof, and similar information shown on the plat or map of survey shall be included with the boundary description.

7. Water boundaries necessarily are subject to change due to erosion or accretion by tidal action or the flow of rivers and streams. A realignment of water bodies may also occur due to many reasons such as deliberate cutting and filling of bordering

## Exhibit 5

### Page 2

lands or by avulsion. Recorded surveys of natural water boundaries are not relied upon by title insurers for location of title.

When a property to be surveyed for title insurance purposes contains a natural water boundary, the surveyor shall measure the location of the boundary according to appropriate surveying methods and note on the plat or map the date of the measurement and the caveat that the boundary is subject to change due to natural causes and that it may or may not represent the actual location of the limit of title. When the surveyor is aware of changes in such boundaries, the extent of those changes shall be identified.

8. When the surveyor has met all of the minimum standard detail requirements for an ALTA/ACSM Land Title Survey, the following certification shall be made on the plat:

**To (name of client), (name of lender, if known), (name of title insurance company, if known), (name of others as instructed by client):**

**This is to certify that this map or plat and the survey on which it is based were made in accordance with "Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys," jointly established and adopted by ALTA, ACSM and NSPS in 1999, and includes Items \_\_\_\_ of Table A thereof. Pursuant to the Accuracy Standards as adopted by ALTA, NSPS, and ACSM and in effect on the date of this certification, undersigned further certifies that [Surveyor to complete certificate with the appropriate ONE of the following three phrases]**

- **the Positional Uncertainties resulting from the survey measurements made on the survey do not exceed the allowable Positional Tolerance.**
- **the survey measurements were made in accordance with the "Minimum Angle, Distance, and Closure Requirements for Survey Measurements Which Control Land Boundaries for ALTA/ACSM Land Title Surveys."**
- **proper field procedures, instrumentation, and adequate survey personnel were employed in order to achieve results comparable to those outlined in the "Minimum Angle, Distance, and Closure Requirements for Survey Measurements Which Control Land Boundaries for ALTA/ACSM Land Title Surveys."**

Date: \_\_\_\_\_ (signed) \_\_\_\_\_ (seal)  
Registration No.

*Adopted by the American Land Title Association on October 6, 1999.  
Adopted by the Board of Direction, American Congress on Surveying and Mapping on October 20, 1999.  
Adopted by the Board of Directors, National Society of Professional Surveyors on October 19, 1999.  
American Land Title Association, 1828 L St., N.W., Suite 705, Washington, D.C. 20036.  
American Congress on Surveying and Mapping, 5410 Grosvenor Lane, Bethesda, MD 20814  
National Society of Professional Surveyors, 5410 Grosvenor Lane, Bethesda, MD 20814*

## Exhibit 5

Page 3

## TABLE A

## OPTIONAL SURVEY RESPONSIBILITIES AND SPECIFICATIONS

**NOTE:** The items of Table A must be negotiated between the surveyor and client. It may be necessary for the surveyor to qualify or expand upon the description of these items, e.g. in reference to Item 6, there may be a need for an interpretation of a restriction. The surveyor cannot make a certification on the basis of an interpretation.

If checked, the following optional items are to be included in the ALTA/ACSM LAND TITLE SURVEY:

1.  Monuments placed (or a reference monument or witness to the corner) at all major corners of the boundary of the property, unless already marked or referenced by an existing monument or witness to the corner.
2.  Vicinity map showing the property surveyed in reference to nearby highway(s) or major street intersection(s).
3.  Flood zone designation (with proper annotation based on Federal Flood Insurance Rate Maps or the state or local equivalent, by scaled map location and graphic plotting only.)
4.  Land area as specified by the client.
5.  Contours and the datum of the elevations.
6.  Identify and show if possible, setback, height, and floor space area restrictions of record or disclosed by applicable zoning or building codes (in addition to those recorded in subdivision maps). If none, so state.
7.  (a) Exterior dimensions of all buildings at ground level  
 (b) Square footage of:
  - (1) exterior footprint of all buildings at ground level
  - (2) gross floor area of all buildings; or
  - (3) other areas to be defined by the client (c) Measured height of all buildings above grade at a defined location. If no defined location is provided, the point of measurement shall be shown.
8.  Substantial, visible improvements (in addition to buildings) such as signs, parking areas or structures, swimming pools, etc.
9.  Parking areas and, if striped, the striping and the type (e.g. handicapped, motorcycle, regular, etc.) and number of parking spaces.
10.  Indication of access to a public way such as curb cuts and driveways.
11.  Location of utilities (representative examples of which are shown below) existing on or serving the surveyed property as determined by:
  - (a) Observed evidence
  - (b) Observed evidence together with plans and markings provided by client, utility companies, and other appropriate sources (with reference as to the source of information)
    - railroad tracks and sidings;
    - manholes, catch basins, valve vaults or other surface indications of subterranean uses;
    - wires and cables (including their function) crossing the surveyed premises, all poles on or within ten feet of the surveyed premises, and the dimensions of all crosswires or overhangs affecting the surveyed premises; and
    - utility company installations on the surveyed premises.
12.  Governmental Agency survey-related requirements as specified by the client.
13.  Names of adjoining owners of platted lands.
14.  Observable evidence of earth moving work, building construction or building additions within recent months.
15.  Any changes in street right of way lines either completed or proposed, and available from the controlling jurisdiction. Observable evidence of recent street or sidewalk construction or repairs.

## Exhibit 5

## Page 4

---

16. \_\_\_\_\_ *Observable evidence of site use as a solid waste dump, sump or sanitary landfill.*

17. \_\_\_\_\_

\_\_\_\_\_

## Exhibit 5

Page 5

## Accuracy Standards for ALTA/ACSM Land Title Surveys

### Introduction

These Accuracy Standards address Positional Uncertainty and Minimum Angle, Distance and Closure Requirements for ALTA-ACSM Land Title Surveys. In order to meet these standards, the Surveyor must assure that the Positional Uncertainties resulting from the survey measurements made on the survey do not exceed the allowable Positional Tolerance. If the size or configuration of the property to be surveyed or the relief, vegetation or improvements on the property will result in survey measurements for which the Positional Uncertainty will exceed the allowable Positional Tolerance, the surveyor must alternatively apply the within table of "Minimum Angle, Distance and Closure Requirements for Survey Measurements Which Control Land Boundaries for ALTA-ACSM Land Title Surveys" to the measurements made on the survey or employ, in his or her judgment, proper field procedures, instrumentation and adequate survey personnel in order to achieve comparable results.

The lines and corners on any property survey have uncertainty in location which is the result of (1) availability and condition of reference monuments, (2) occupation or possession lines as they may differ from record lines, (3) clarity or ambiguity of the record descriptions or plats of the surveyed tracts and its adjoiners and (4) Positional Uncertainty.

The first three sources of uncertainty must be weighed as evidence in the determination of where, in the professional surveyor's opinion, the boundary lines and corners should be placed. Positional Uncertainty is related to how accurately the surveyor is able to monument or report those positions.

Of these four sources of uncertainty, only Positional Uncertainty is controllable, although due to the inherent error in any measurement, it cannot be eliminated. The first three can be estimated based on evidence; Positional Uncertainty can be estimated using statistical means.

The surveyor should, to the extent necessary to achieve the standards contained herein, compensate or correct for systematic errors, including those associated with instrument calibration. The surveyor shall use appropriate

error propagation and other measurement design theory to select the proper instruments, field procedures, geometric layouts and computational procedures to control and adjust random errors in order to achieve the allowable Positional Tolerance or required traverse closure.

If radial survey methods are used to locate or establish points on the survey, the surveyor shall apply appropriate procedures in order to assure that the allowable Positional Tolerance of such points is not exceeded.

### Definitions:

"Positional Uncertainty" is the uncertainty in location, due to random errors in measurement, of any physical point on a property survey, based on the 95% confidence level.

"Positional Tolerance" is the maximum acceptable amount of Positional Uncertainty for any physical point on a property survey relative to any other physical point on the survey, including lead-in courses.

### Computation of Positional Uncertainty

The Positional Uncertainty of any physical survey monument on a survey, whether the location of that point was established using GPS or conventional surveying methods, may be computed using a minimally constrained, correctly weighted least squares adjustment of the points on the survey.

### Application of Minimum Angle, Distance and Closure Requirements

The combined precision of a survey can be statistically assured by dictating a combination of survey closure and specified procedures for an ALTA/ACSM Land Title Survey. ACSM, NSPS and ALTA have adopted the following specific procedures in order to assure the combined precision of an ALTA/ACSM Land Title Survey. The statistical base for these specifications is on file at ACSM and available for inspection

## Exhibit 5

Page 6

**Positional Tolerances for ALTA/ACSM Land Title Surveys**

0.07 feet (or 20 mm) + 50 ppm

**Minimum Angle, Distance and Closure Requirements for Survey Measurements Which Control Land Boundaries for ALTA-ACSM Land Title Surveys (Note 1)**

Direct Reading of Instrument (Note 2)	Instrument Reading Estimated (Note 3)	Number of Observations Per Station (Note 4)	Spread from Mean of D&R Not To Exceed (Note 5)	Angle Closure (Where N = No. of Stations) Not to Exceed	Linear Closure (Note 6)	Distance Measurement (Note 7)	Minimum Length of Measurements (Notes 8, 9 and 10)
20" <1'> <b>10"</b>	5" <0.1'> NA	2 D&R	5" <0.1'> <b>5"</b>	10" times square root of N	1:15,000	EDM or Doubletape with Steel Tape	(8) 81m, (9) 153m, (10) 20m

Note (1) The use of a more precise instrument does not change the other requirements, such as number of angles turned, etc.

Note (2) Instrument must have a direct reading of at least the amount specified (not an estimated reading), i.e.: 20" = Micrometer reading theodolite, <1'> = Scale reading theodolite, **10"** (boldface) = Electronic reading theodolite.

Note (3) Instrument must have the capability of allowing an estimated reading below the direct reading to the specified reading.

Note (4) D&R means the Direct and Reverse positions of the instrument telescope, i.e., two angles in the direct and two angles in the reverse position be measured and meaned.

Note (5) Any angle measured that exceeds the specified amount from the mean must be rejected and the set of angles re-measured.

Note (6) Ratio of closure after angles are balanced and closure calculated.

Note (7) All distance measurements must be made with a properly calibrated EDM or Steel tape, applying atmospheric, temperature, sag, tension, slope, scale factor and sea level corrections as necessary.

Note (8) EDM having an error of 5mm, independent of distance measured (Manufacturer's specifications).

Note (9) EDM having an error of 10mm, independent of distance measured (Manufacturer's specifications).

Note (10) Calibrated steel tape.



