CHAPTER 19

PRELIMINARY ENGINEERING AND HYDROLOGIC ANALYSIS

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INTRODUCTION

Preliminary engineering is an integral component of schematic design, the third step in the land development design process. Schematic design normally occurs after the completion of feasibility and site analysis (step 1) and conceptual design (step 2). At the conclusion of schematic design, the engineer is ready to initiate the last stage of the design process, which is final design. This chapter focuses on the preliminary engineering tasks associated with the development of the schematic design.

Preliminary engineering involves the refinement and development of the information obtained in the previous two steps in the design process. This refinement ordinarily leads to the creation of a deliverable often in the form of a graphic, such as a site plan or a general development plan, but it may include accompanying narrative reports, checklists, estimates, and so on, for review by the developer/builder and local governing agencies. Generally, preliminary engineering is performed by civil engineers; however, planners, architects, landscape architects, the developer/builder, environmental scientists, and reviewers at the local governing agencies, as well as citizens, may be contributors or stakeholders in this process.

The preliminary engineering stage of the design process includes an engineering review of the conceptual design, feasibility, and site analysis. During the earlier design stages, the development program was determined, pertinent information gathered, the site analyzed with all opportunities and constraints presented, and a conceptual site design prepared. At this stage it is time to fine-tune the site design by refining the previous assumptions, performing preliminary engineering analyses and evaluations, and validating the previous assumptions. This is especially important if there are specific, measurable development objectives that are to be achieved in the design such as LEED certification, other third-party green building certification, or (enhanced) compliance with specific regulatory requirements.

Although preliminary engineering often requires a specific level of design detail necessary for review and approval by local governing agencies, a preliminary engineering deliverable is not always required. In many cases, the developer/builder typically requires this level of detail prior to incurring the costs associated with developing the final design. The deliverables produced during the preliminary engineering phase represent a final check of the development program before proceeding with the more detailed final engineering design. Oftentimes, the developer/builder will utilize the information prepared in the preliminary engineering phase to prepare initial construction cost estimates in order to ensure the final design is achievable as originally envisioned or to identify adjustments to the design that must be made to meet project budget constraints. This provides not only the developer/builder and/or local governing review agencies, but also citizens, with a greater sense of comfort about the feasibility and construction costs associated with the proposed development program.

At this stage, the developer/builder might be the owner of the property or the contract purchaser with contingencies or options built into the contract pending the obtaining of an approved rezoning or other related regulatory approvals. Other contingencies may include a specific lot yield, development density, green building rating, or desired gross floor area of a building.

It is important to understand that the development program is always subject to change, particularly when it is presented to citizens and/or public review agencies or when a developer is faced with meeting budget constraints. Hence, the preliminary engineering phase of the design process is oftentimes an evolutionary and iterative process that can last for months or even years as the development approval process (namely, rezoning or entitlement) progresses. For instance, it is not uncommon for changes to the site grading and layout to occur as a result of comments from regulatory agencies or citizens requesting additional open-space and tree preservation areas.

COMPONENTS OF A PRELIMINARY ENGINEERING STUDY

Although the level of detail required in 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 ordinarily required.

A comprehensive base map is an essential part of the preliminary engineering analysis. Ideally, this map includes field-run topographic survey information and site boundary, as well as the location of any existing structures and other physical features of the site. The base map should also include the site opportunities such as buildable areas and natural site amenities and the site constraints such as wetlands, floodplains, mature trees, and environmental corridors. For purposes of schematic-level design, these items should be field-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 minimize problems when preparing the final documents for construction. The following sections of this chapter present a detailed look at each of the components of a preliminary engineering study.

Site Layout and Roadway Design

The first step in developing the preliminary engineering plans for a schematic design is to formalize the conceptual design layout into a geometrically accurate layout. This layout may include the horizontal alignment and configuration of roadways and lot layout in the case of a subdivision plan or arrangement of buildings and parking areas in the case of a site plan. Widths of proposed pavements should be developed based on expected traffic volumes determined from completed traffic studies, or as local requirements may dictate. Curbs, gutters, and sidewalks should also be depicted as appropriate and site access to adjacent roadways illustrated, including any provisions for pedestrian and/or bicycle access improvements or signalization that may be warranted. 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 proposed on-site and off-site improvements should be shown, including easements necessary for utilities and ingress-egress (access).

It is important not to forget that every site layout requires the designer to think in three dimensions. In other words, one should not lose sight of the fact that elevations vary across the layout and the vertical component of the design must be considered when developing the horizontal site geometry. Oftentimes, this becomes an iterative process where the conceptual layout is refined based on existing and proposed site elevations that must be taken into consideration. Part of this refinement can be accomplished by development of profiles of roadways and major site circulation elements. Several computer-aided design and drafting (CADD) software packages allow profiles to be generated fairly quickly based on proposed roadway alignments sampled from a three-dimensional digital terrain model (DTM) of the site topography. Development of roadway profiles allows preparation of a proposed profile with vertical geometric accuracy. They also provide a means to evaluate sight distance and initial analysis of cut/fill requirements.

Once profiles have been developed, typical cross sections for roadways can be prepared based on the required roadway widths, cross slope, and any superelevation requirements. Again, many CADD software packages allow the designer to iteratively evaluate and design various road cross-section configurations and link them to the proposed profile to quickly generate contours along the road alignments that facilitate development of the preliminary site-grading plan.

When preparing a preliminary site plan, similar three-dimensional considerations must be given to the location of parking areas, building placement, vehicular and pedestrian circulation paths, and landscaped areas. The overall development of the site layout plan should take these grading requirements into consideration so that the grading concept is developed as the layout progresses, as opposed to following after the layout is complete. See Figure 19.1.

HYDROLOGIC ANALYSIS

The emphasis on vertical or three-dimensional thinking during preliminary site layout is due in large part to the increasing importance of drainage and stormwater management considerations. Site layout, grading, and drainage are interrelated: stormwater management systems are integral parts of a functional site and require early attention in order to ensure sufficient space is allotted and adequate outfall scenarios assessed. In order to develop a comprehensive stormwater management program—one that meets applicable jurisdictional guidelines and adheres to client/developer expectations and project goals—the engineer must have a thorough understanding of the site hydrologic conditions.

The hydrologic conditions of a site—the topography, soils, land cover condition, and rainfall pattern—are assessed pre- and postdevelopment and compared. This comparison determines the applicable stormwater management requirements and reveals feasible strategies for managing site runoff.

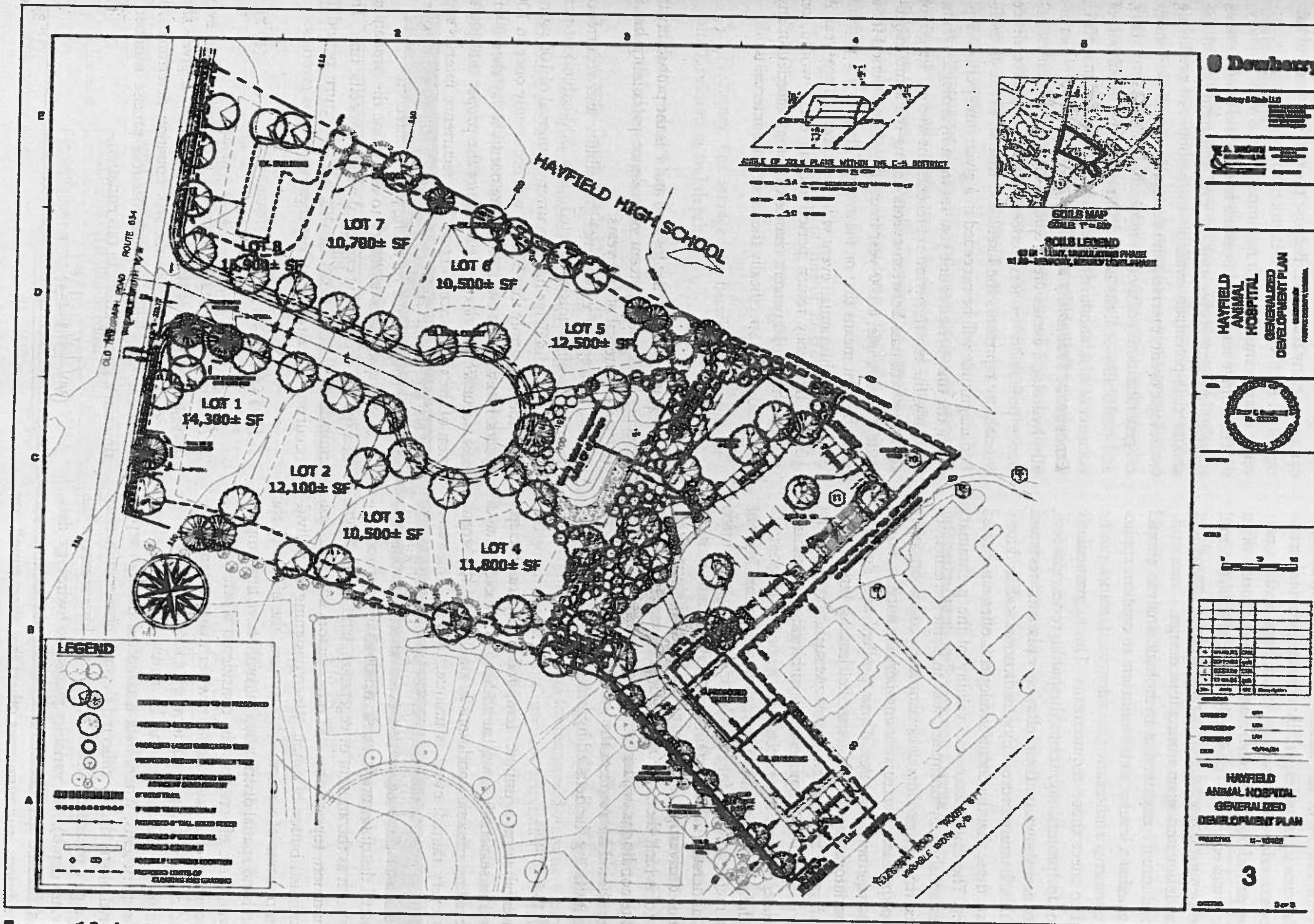


FIGURE 19.1 Example of a preliminary site plan.

Like all components of the design process, the hydrologic analysis is often iterative, especially when it comes to assessing the postdevelopment scenario. Hydrologic analysis performed as part of the preliminary engineering effort is often updated, checked, and rechecked as the site layout and grading scheme are resolved in increasing detail throughout schematic design and again during final design.

The preliminary engineering hydrologic analysis should be performed at a level of detail sufficient to confirm applicable requirements and ensure that adequate facilities can be provided to meet those requirements. This is particularly important for projects subject to the public review process, as the facilities shown on the preliminary plan and reviewed in the public hearing process by jurisdictional staff and local citizens are those that the client/developer is often committed to deliver. The importance and validity of the preliminary plan is established by ordinance and in some jurisdictions reaffirmed through proffers or development conditions often related to specific infrastructure components such as stormwater management facilities. These proffers and development conditions can be very specific, leaving little wiggle room during the final design process. As such, it is important for the preliminary engineering hydrologic analysis and related design efforts to be accurate but conservative.

The following discussion constitutes a primer on the popular methodologies for hydrologic analysis. It is important to understand that certain sites may warrant more or less detailed investigation, and, at times, alternate methodologies (of which there are many) may be required due to either site conditions or jurisdictional design standards. The engineer should be aware of the jurisdictional requirements prior to initiating the detailed hydrologic study.

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 event affects the temporal and spatial distribution of runoff. Additionally, surface conditions such as the amount of vegetation, land use, soil type and condition, topography, and other factors affect the volume and distribution of runoff. When designing the individual components of a storm drain system, the effects of the temporal and spatial distribution of runoff have little impact. However, these effects must be considered when designing larger components, such as stormwater management facilities and major culverts, or determining the floodplain.

Hydrologic data is historic by nature. Unlike conventional experiments, where data is collected through repetition, hydrologic data is collected through observation of an event (e.g., a measured amount of rainfall for a storm or the floodwater 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, and probability analysis is used to predict the likelihood of occurrence of a random event such as a given design storm.

Exceedence Probability and Recurrence Interval. Generally, 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. Typically, the time period is one year for most hydrologic events. Similarly, the return period represents the *average* length of time that will pass between events having the same magnitude. For example, a 100-year frequency return period for a rainfall event means that *on the average*, there is a 1 percent chance that this rainfall event will be *exceeded* in any year. A 10-year frequency return period rainfall event would, on average, have a 10 percent chance of being exceeded in any year, and so on. Specifically the recurrence interval is:

$$T_r = \frac{1}{P} \times 100 \tag{19.1}$$

where T_r is the recurrence interval and P is the probability in percent. Hence a 1 percent exceedence probability has a recurrence interval of 100 years.

The concept of exceedence probability and recurrence interval is often misinterpreted. If a 500-cubic-foot-per-second (cfs) discharge 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.

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:

$$P = 1 - \left(1 - \frac{1}{T_R}\right)^N \tag{19.2}$$

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 that an event will occur a precise number of times in a given period. In this equation,

$$P = \frac{(N!)\left(\frac{1}{T_r}\right)^I \left(1 - \frac{1}{T_r}\right)^{N-I}}{I!(N-I)!} \tag{19.3}$$

Here, I is the exact number of times the event with T_r occurs in N successive years.

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:

1. *Duration:* The length of time of the storm event
2. *Depth:* The total amount of precipitation for the duration of the storm event
3. *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:

4. *Intensity:* Depth divided by duration
5. *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 also 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 19.1 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 the type of data required to generate the design storm. Simple types of computational procedures, such as the rational method, require basic intensity-duration-frequency 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). Since then, the National Oceanic and Atmospheric Administration's National Weather Service (NOAA's NWS) has updated portions of the country's rainfall data by introducing NOAA Atlas 14. NOAA Atlas 14 updated the rainfall data for many mid-Atlantic, Ohio Valley, and southwest states. Current precipitation frequency data can be found at the NWS website (http://hdsc.nws.noaa.gov/hdsc/pfds/index.html). Although rainfall data has been updated by the NWS, not all localities have adopted the change

TABLE 19.1 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.

yet. The engineer should always check with the jurisdiction to determine the applicable rainfall values to use for design purposes.

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 (and not covered by Atlas 14), the TP-40 information has been superseded by NOAA's 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 19.2 through 19.7. (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.)

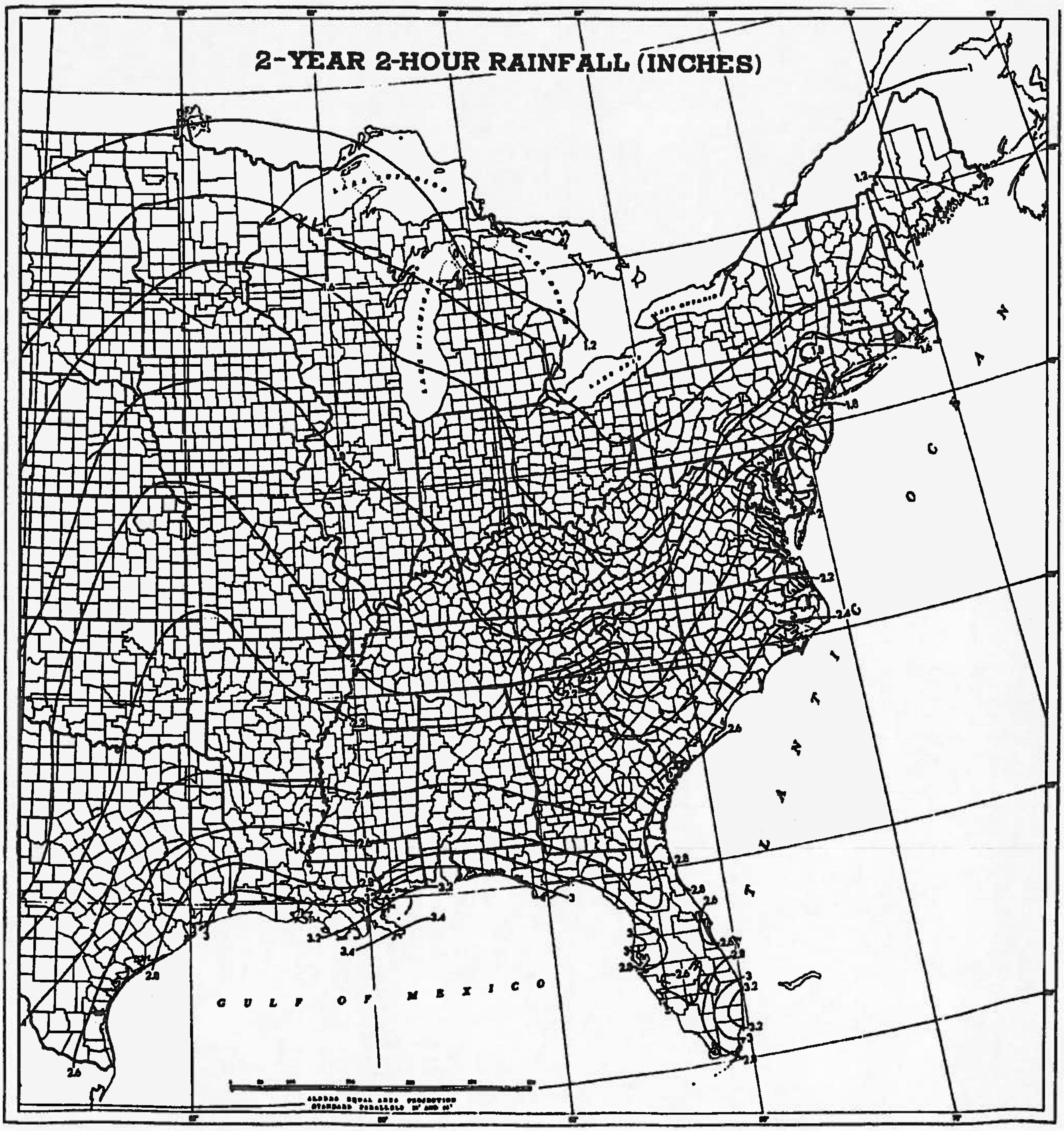


FIGURE 19.2 Isopluvial map.

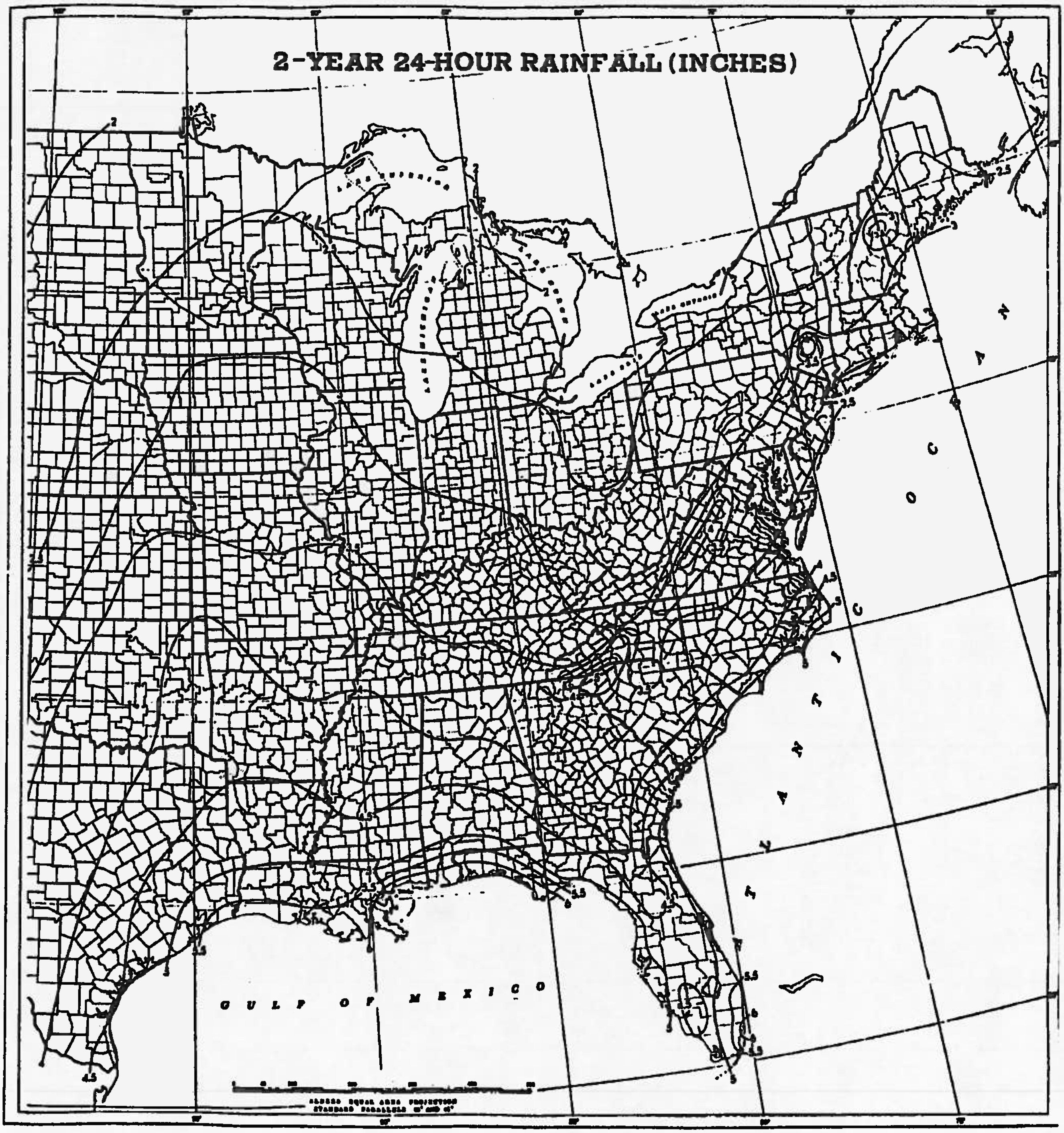


Figure 19.3 Isopluvial map.

Table 19.2 lists the relevant precipitation atlas series or references currently available for use in the United States. Other selected depth-frequency references are listed in Table 19.3.

Point depths, such as those on TP-40, apply to areas less than about 10 square miles. Reductions 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 percentage of reduction to be applied to point depths for large catchment areas. However, since most catchments in land development projects are less 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

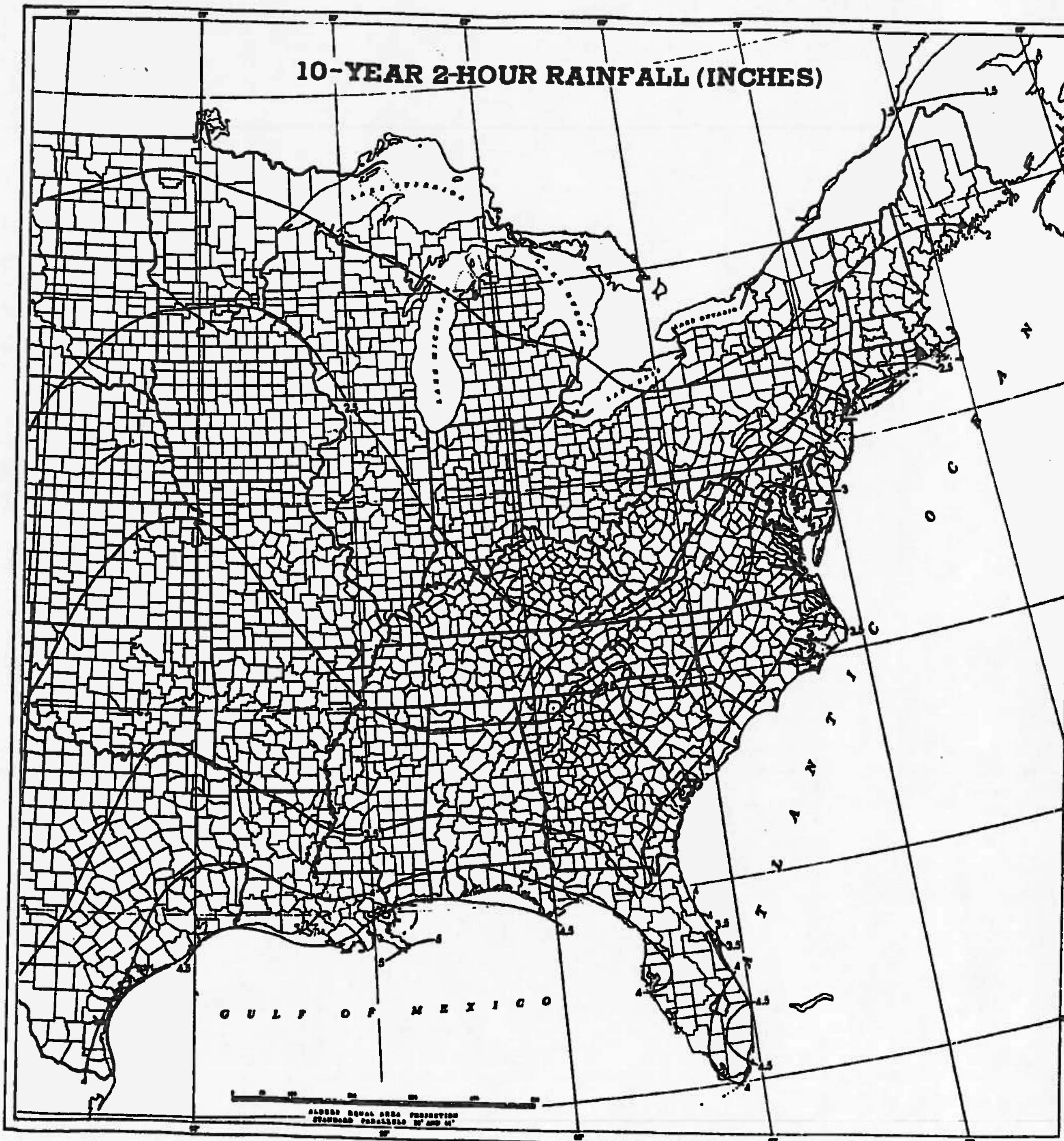


FIGURE 19.4 Isopluvial map.

ordinate (y axis), duration along the abscissa (x axis), 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. Note: This is quite different from the misconception that they represent an actual duration or actual time history 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 used mainly in conjunction with the rational method for determining peak runoff. See Figure 19.8 for a typical IDF curve.

IDF curves are available through many local agencies such as the state highway departments and the Natural

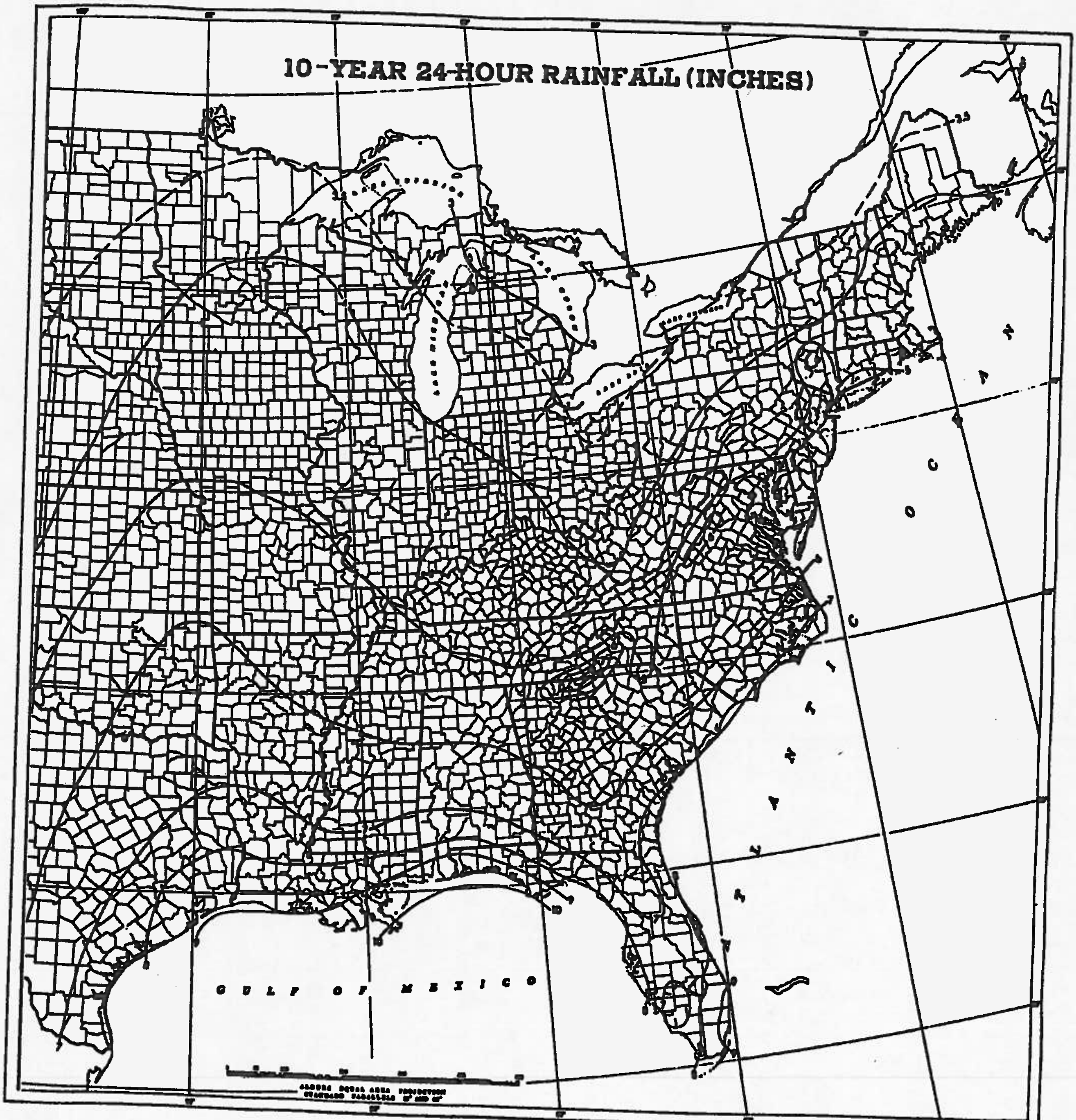


Figure 19.5 Ten-year 24-hour rainfall map.

Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS). In rare cases where IDF curves cannot be obtained, they can be developed from current and applicable U.S. Weather Service maps or from frequency analysis using local rainfall information. Discussions on developing IDF curves through frequency analysis are provided in Chow (1959) and Kibler (1982) in the references for this chapter. Besides NOAA Atlas 14, TP-40, and HYDRO-35, the NWS provides other documents relating the depth-duration-frequency of storms, as listed in Table 19.3.

Rational Method Hydrology

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

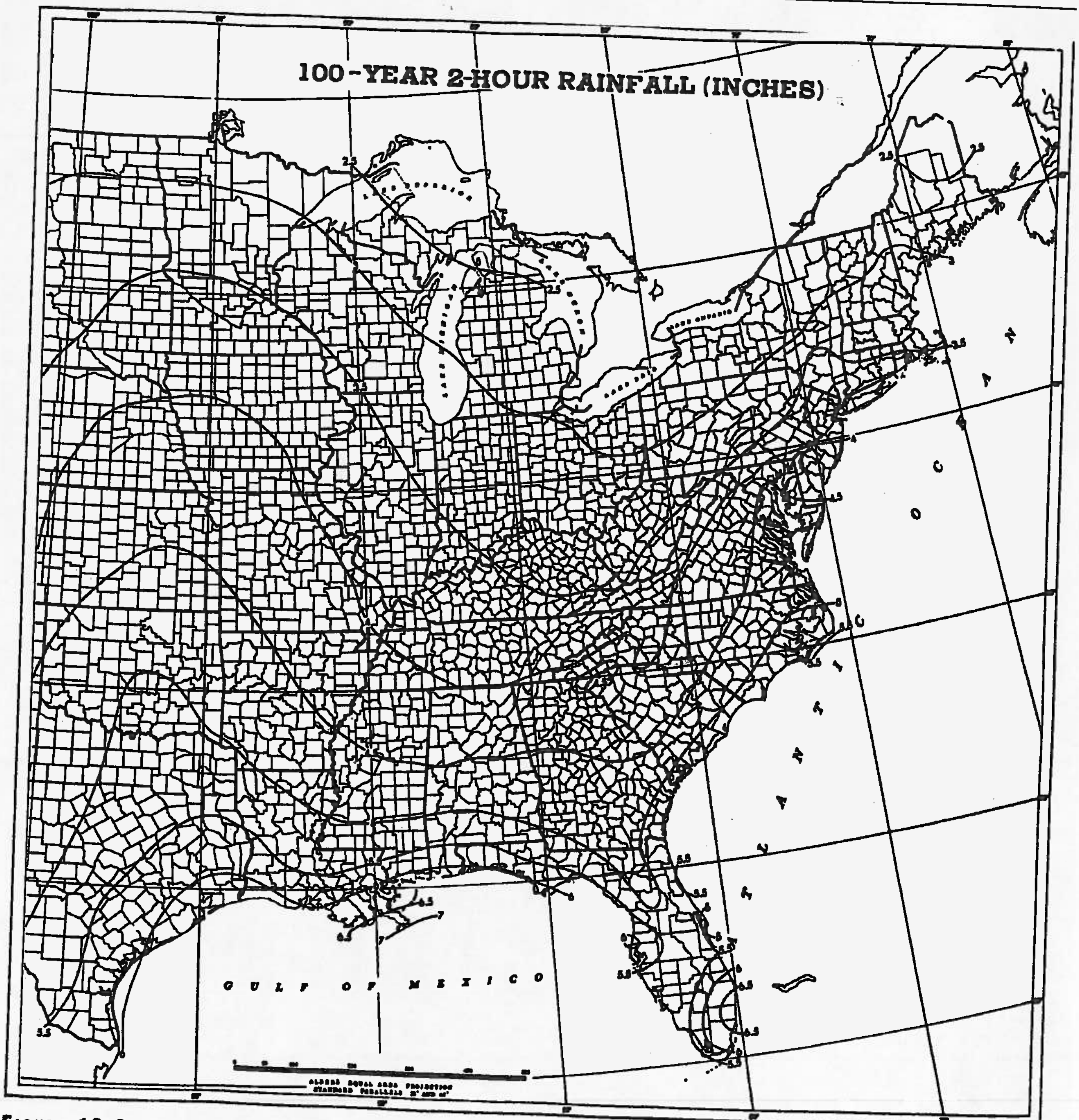


FIGURE 19.6 One-hundred-year 2-hour isopluvial map.

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:

$$Q_P = CiA \tag{19.4}$$

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 average rainfall intensity

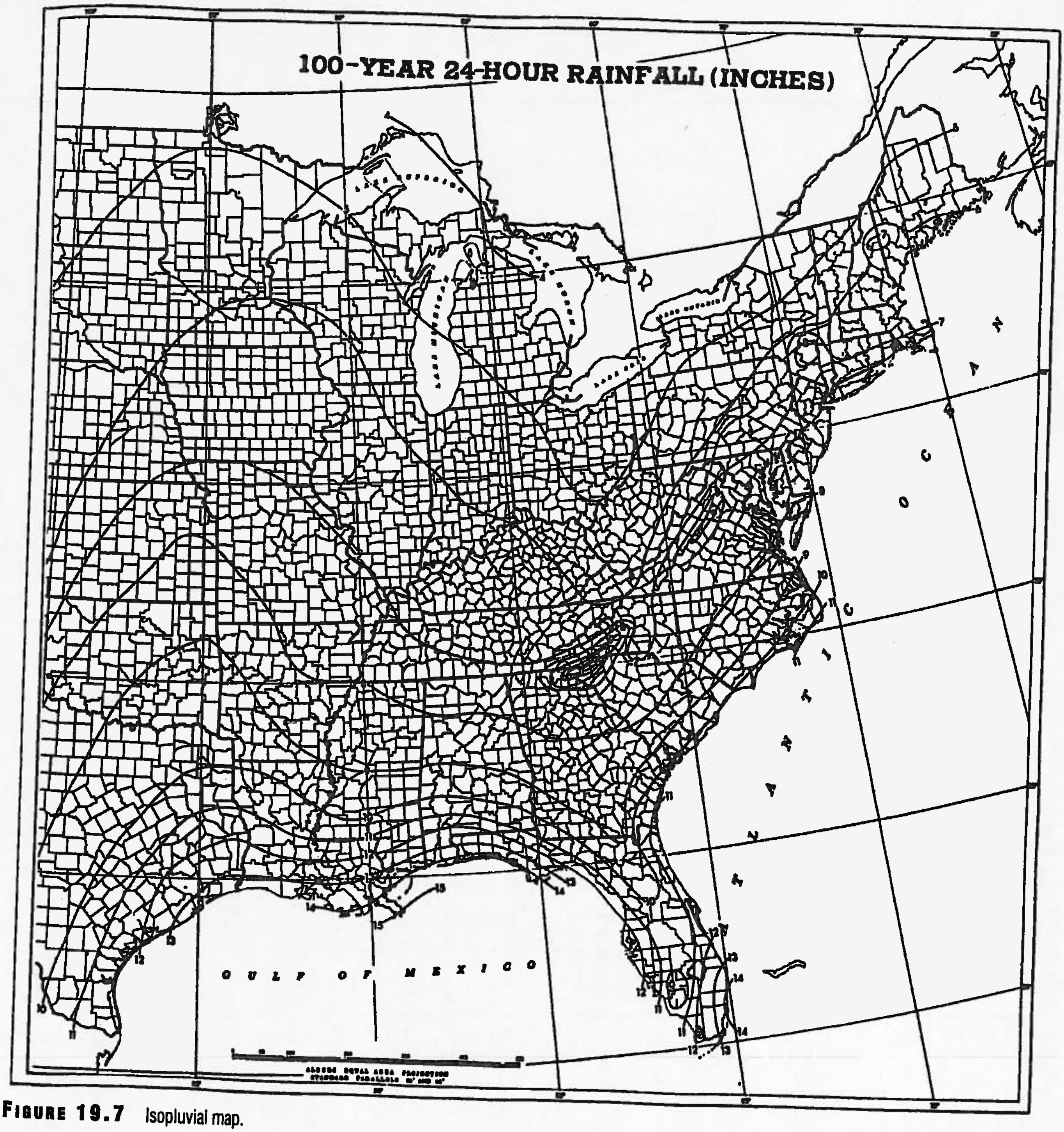


Figure 19.7 Isopluvial map.

(in/hr). The precision of the peak discharge depends on the estimated values of C and i. The average 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 maximum restrictions on the applicability of the rational method, ranging from 20 acres 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 Equation 19.4, 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

TABLE 19.2 Current NWS Precipitation Frequency Publications

Location	5 min.–60 min.	1 hr.–24 hr.	2 day–10 day
Arizona, Nevada, New Mexico, Utah, and Southeast California	NOAA Atlas 14 (2003)	NOAA Atlas 14 (2003)	NOAA Atlas 14 (2003)
Remainder of the Western U.S.	Arkell & Richards (1986) Frederick & Miller (1979)	NOAA Atlas 2 (1973)	Tech. Paper 49 (1964)
Delaware, Illinois, Indiana, Kentucky, Maryland, New Jersey, North Carolina, Ohio, Pennsylvania, South Carolina, Tennessee, Virginia, West Virginia, and Washington, DC	NOAA Atlas 14, Volume 2 (June 2004)	NOAA Atlas 14, Volume 2 (June 2004)	NOAA Atlas 14, Volume 2 (June 2004)
Remainder of the Eastern U.S.	Tech. Memo 35 (1977)	Tech. Paper 40 (1961)	Tech. Paper 49 (1964)
Hawaii	Tech. Paper 43 (1962)	Tech. Paper 43 (1962)	Tech. Paper 51 (1965)
Alaska	Tech. Paper 47 (1963)	Tech. Paper 47 (1963)	Tech. Paper 52 (1965)
Puerto Rico	NOAA Atlas 14, Volume 3	NOAA Atlas 14, Volume 3	NOAA Atlas 14, Volume 3

that accounts for abstractions (losses before runoff begins, including mainly interception, infiltration, and surface storage), antecedent runoff conditions (index of the runoff potential of the soil before a storm event), and other variables affecting the runoff rate. Table 19.4 identifies the version of the runoff coefficient used by the American Society of Civil Engineers (ASCE), and the standards used in Austin, Texas, are shown as an example in Table 19.5.

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, Equation 19.4 becomes:

$$Q_P = C_f CiA \qquad (19.5)$$

where the correction factor C_f varies by recurrence interval. Comparing the City of Austin example in Table 19.5, the C coefficient would remain the same for all storms; however, the C_f factor would change for storms greater than the two-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. The engineer should check with the local agency to determine whether correction factors exist.

Use of hydrologic soil groups is more common in NRCS hydrology (discussed later); however, Table 19.6 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:

$$C_w = \frac{\sum_{i=1}^{m} C_i A_i}{A_T} \qquad (19.6)$$

where C_w is the weighted C coefficient, A_i is the area of the subarea 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 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 accelerate the travel time to the outlet point, while an area closer to the outlet point could 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 a flowing 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 often draining to a catchment, such as a street inlet. Channel time is that part of the flow time

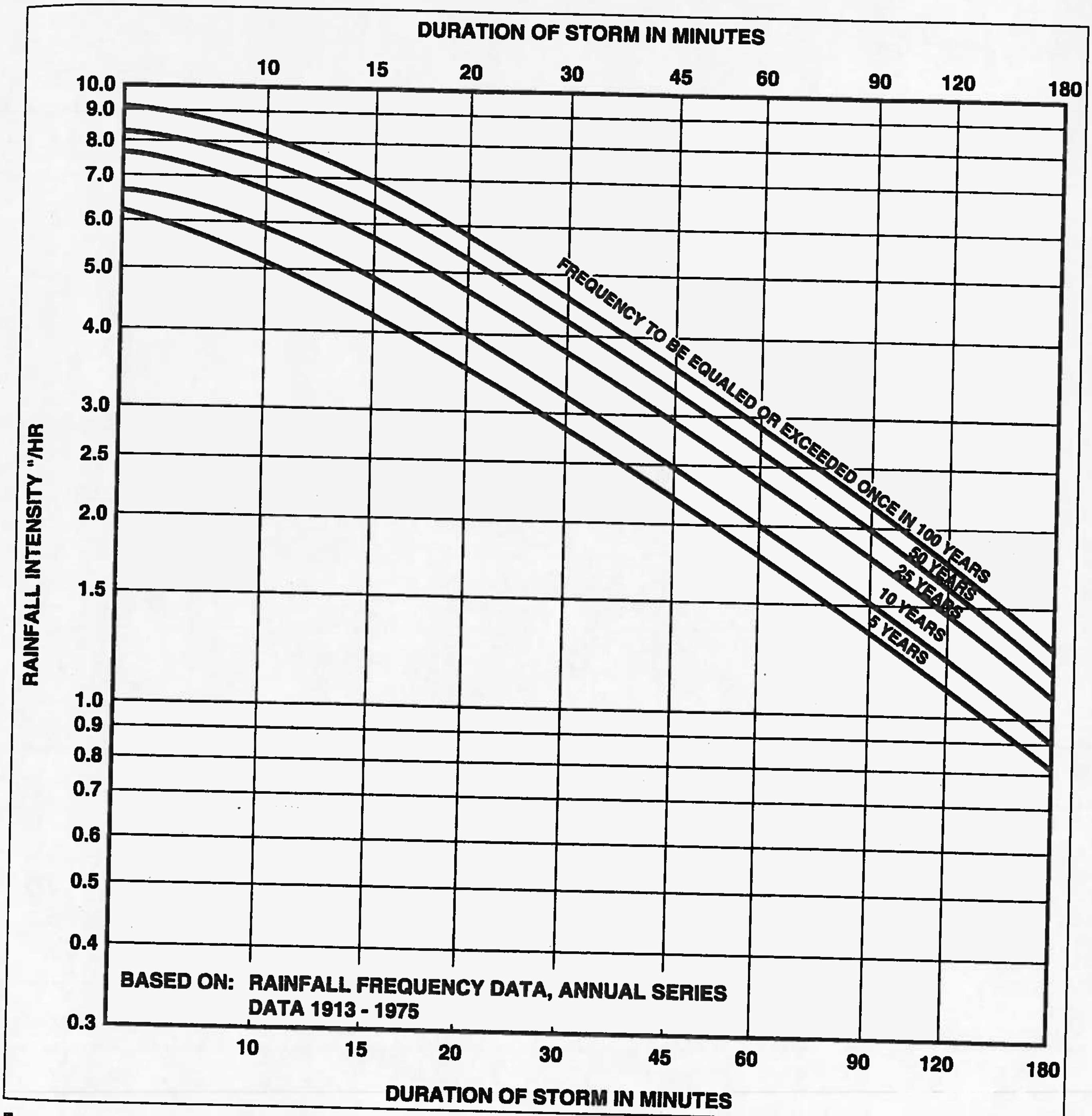


Figure 19.8 Intensity-duration-frequency curves.

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 NRCS's hydrologic definitions of sheet flow and overland flow (see discussion later in this chapter).

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 feet. For grassed areas the time of concentration may vary between 5 to 30 minutes for flow paths between 100 and 500+ feet.

There are numerous empirical methods to determine the inlet time of concentration. The method selected depends

TABLE 19.3 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.

on the information available and the preferences dictated by local review agencies. Some of the various methods are listed in Table 19.7. Two of these methods are subsequently discussed.

One of the better-known methods relating 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 paved surfaces (see Table 19.7). The Kirpich equation is:

$$t_c = 0.0078\left(\frac{L^{0.77}}{S^{0.385}}\right) \tag{19.7}$$

TABLE 19.4 Runoff Coefficients *C* Recurrence Interval ≤ 10 years*

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

*For 25- to 100-year recurrence intervals, multiply coefficient by 1.1 and 1.25, respectively, and the product cannot exceed 1.0.

TABLE 19.5 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.)							
Poor condition (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
Fair condition (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
Good condition (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.

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 = ΔElev/L.

Manning's kinematic solution, in the following form, can be used to compute sheet flow travel time.

$$T_t = \frac{(0.93)(NL)^{1/m}}{(i)^{(m-1/m)}\, S^{1/2m}} \tag{19.8}$$

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 19.8), L is the flow length in feet, i is rainfall intensity (inches/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 feet. The solution to this equation is a trial-and-error procedure performed as follows:

1. Assume a value of i.
2. Use Equation 19.8 to find T_t.
3. Find the actual rainfall intensity from an 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.

TABLE 19.6 Runoff Coefficients for the Rational Formula by Hydrologic Soil Group and Slope Range

	A			B			C			D		
LAND USE	**0–2%**	**2–6%**	**6%+**	**0–2%**	**2–6%**	**6%+**	**0–2%**	**2–6%**	**6%+**	**0–2%**	**2–6%**	**6%+**
Cultivated land	0.08* 0.14†	0.13 0.18	0.16 0.22	0.11 0.16	0.15 0.21	0.21 0.28	0.14 0.20	0.19 0.25	0.26 0.34	0.18 0.24	0.23 0.29	0.31 0.41
Pasture	0.12 0.15	0.20 0.25	0.30 0.37	0.18 0.23	0.28 0.34	0.37 0.45	0.24 0.30	0.34 0.42	0.44 0.52	0.30 0.37	0.40 0.50	0.50 0.62
Meadow	0.10 0.14	0.16 0.22	0.25 0.30	0.14 0.20	0.22 0.28	0.30 0.37	0.20 0.26	0.28 0.35	0.36 0.44	0.24 0.30	0.30 0.40	0.40 0.50
Forest	0.05 0.08	0.08 0.11	0.11 0.14	0.08 0.10	0.11 0.14	0.14 0.18	0.10 0.12	0.13 0.16	0.16 0.20	0.12 0.15	0.16 0.20	0.20 0.25
Residential lot Size ⅛ acre	0.25 0.33	0.28 0.37	0.31 0.40	0.27 0.35	0.30 0.39	0.35 0.44	0.30 0.38	0.33 0.42	0.38 0.49	0.33 0.41	0.36 0.45	0.42 0.54
Lot size ¼ acre	0.22 0.30	0.26 0.34	0.29 0.37	0.24 0.33	0.29 0.37	0.33 0.42	0.27 0.36	0.31 0.40	0.36 0.47	0.30 0.38	0.34 0.42	0.40 0.52
Lot size ⅓ acre	0.19 0.28	0.23 0.32	0.26 0.35	0.22 0.30	0.26 0.35	0.30 0.39	0.25 0.33	0.29 0.38	0.34 0.45	0.28 0.36	0.32 0.40	0.39 0.50
Lot size ½ acre	0.16 0.25	0.20 0.29	0.24 0.32	0.19 0.28	0.23 0.32	0.28 0.36	0.22 0.31	0.27 0.35	0.32 0.42	0.26 0.34	0.30 0.38	0.37 0.46
Lot size 1 acre	0.14 0.22	0.19 0.26	0.22 0.29	0.17 0.24	0.21 0.23	0.26 0.34	0.20 0.28	0.25 0.32	0.31 0.40	0.24 0.31	0.29 0.35	0.35 0.46
Industrial	0.67 0.85	0.68 0.85	0.68 0.86	0.68 0.85	0.68 0.86	0.69 0.86	0.68 0.86	0.69 0.86	0.69 0.87	0.69 0.86	0.69 0.86	0.70 0.88
Commercial	0.71 0.88	0.71 0.88	0.72 0.89	0.71 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.90	0.72 0.89	0.72 0.89	0.72 0.90
Streets	0.70 0.76	0.71 0.77	0.72 0.79	0.71 0.80	0.72 0.82	0.74 0.84	0.72 0.84	0.73 0.85	0.76 0.89	0.73 0.89	0.75 0.91	0.78 0.95
Open space	0.05 0.11	0.10 0.16	0.14 0.20	0.08 0.14	0.13 0.19	0.19 0.26	0.12 0.18	0.17 0.23	0.24 0.32	0.16 0.22	0.21 0.27	0.28 0.39
Parking	0.85 0.95	0.86 0.96	0.87 0.97	0.85 0.95	0.86 0.96	0.87 0.97	0.85 0.95	0.86 0.96	0.87 0.97	0.85 0.95	0.86 0.96	0.87 0.97

Source: Kibler, D.F., et al., 1982. *Recommended Hydrologic Procedures for Computing Urban Runoff in Pennsylvania.* Commonwealth of Pa. Harrisburg Pa.: Dept. of Environmental Resources.

*Runoff coefficients for storm recurrence intervals less than 25 years.

†Runoff coefficients for storm recurrence intervals of 25 years or more.

Figures 19.9 and 19.10 represent nomographs used to solve the time of concentration equations for the Kirpich method and the Manning's kinematic solution, respectively.

Often, large catchments require the consideration of several flow paths in determining which represents the time of 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

TABLE 19.7 Summary of Time of Concentration Formulas

METHOD AND DATE	FORMULA FOR t_c (min)	REMARKS
Kirpich (1940)	$t_c = 0.0078 L^{0.77} S^{-0.385}$ 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)	$t_c = 60\left(11.9\dfrac{L^3}{H}\right)^{0.385}$ 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)	$t_c = \dfrac{41.025(0.007\, i + c)L^{0.33}}{S^{0.33} i^{0.67}}$ 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 ≤500.
Federal Aviation Administration (1970)	$t_c = 1.8(1.1 - C)\dfrac{L^{0.5}}{S^{0.33}}$ 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)	$t_c = \dfrac{0.94\, L^{0.6} n^{0.6}}{i^{0.4} S^{0.3}}$ 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)	$t_c = \dfrac{1}{60} \leq \dfrac{L}{V}$ 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, DC.

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 following example shows when the t_c should be carefully considered for nonhomogeneous catchments. Consider the situation given in Figure 19.11 and determine the peak discharge at the outlet, using the rational method.

The rainfall intensity is given as:

$$i = \frac{97.5}{T_d^{0.83} + 6.88} \qquad (19.9)$$

TABLE 19.8 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. 1990. *HEC-1 Flood Hydrograph Package Users Manual.*

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:

$$C_w = \frac{(0.2)(60)+(0.7)(30)}{90} = 0.37 \qquad (19.10)$$

The peak discharge per the rational method is 83.6 cfs. Compare this to the peak discharge only from the developed area of 108.4 cfs:

$$\begin{aligned} Q_p &= (30\text{ ac})(0.7)(5.16\text{ in/hr}) \\ &= 108.4\text{ cfs } (i = 5.16\text{ in/hr for } t_c = 20\text{ min}) \end{aligned} \qquad (19.11)$$

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 19.12. For this discussion, the rainfall intensity is given by Equation 19.8. 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:

$$C_N = \frac{(0.7)(30)+(0.3)(80)}{(30+80)} = 0.41 \qquad (19.12)$$

The peak discharge is:

$$Q_p = (110\text{ ac})(0.41)(2.65\text{ in/hr}) = 119.5\text{ cfs} \qquad (19.13)$$

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 – 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:

$$Q_p = i\left[(CA)_B + \left(\frac{(t_c)_{inc}}{t_c}(CA)\right)_A\right] \qquad (19.14)$$

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 19.9.

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 to 1 square mile (640 acres) have been proposed. Certainly for the relatively small catchments (<20 acres) encountered in minor storm drain design, the rational method should be satisfactory for use.

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 in 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 reflecting current conditions.

To summarize, the basic assumptions in the rational method are:

- 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.

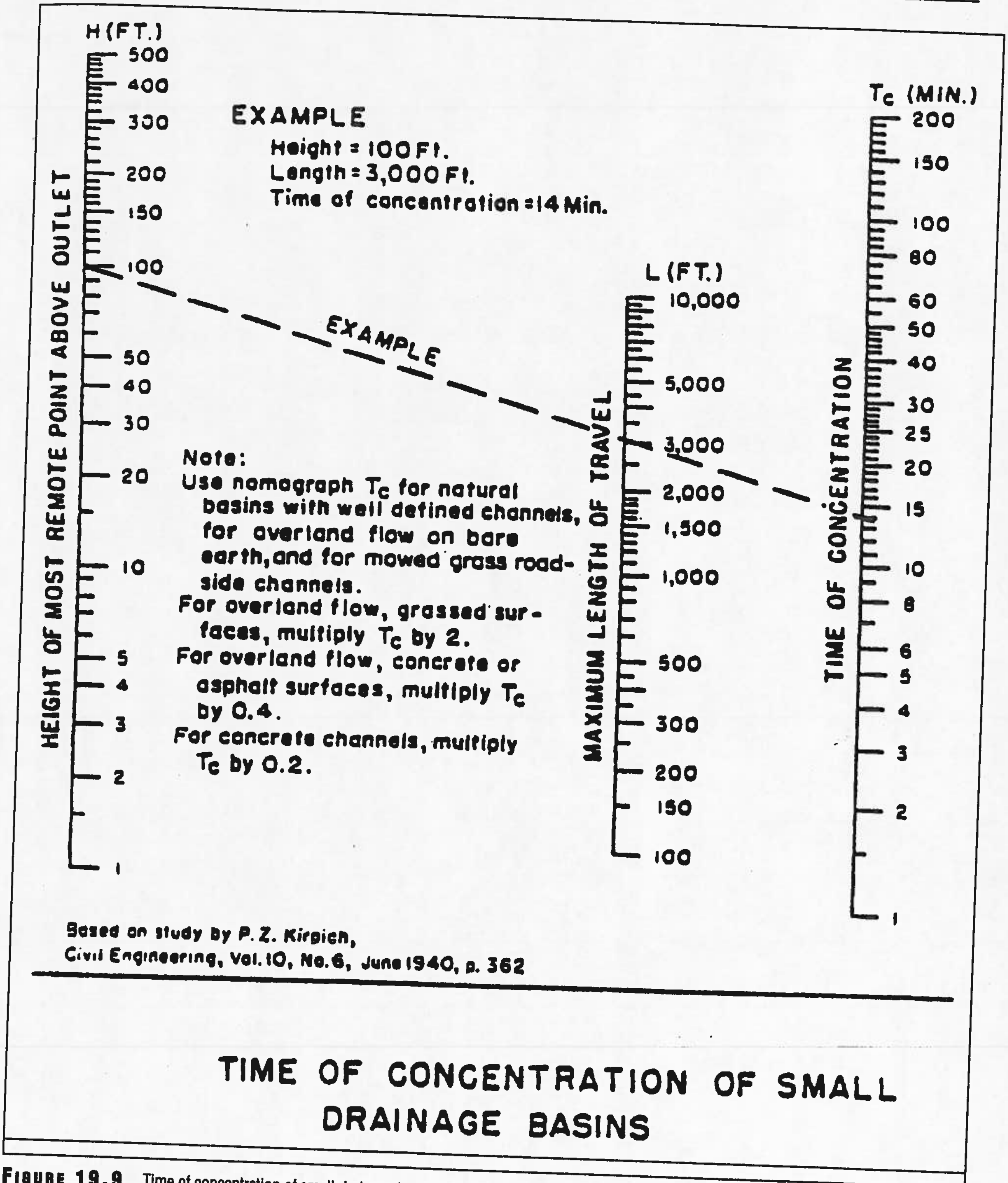
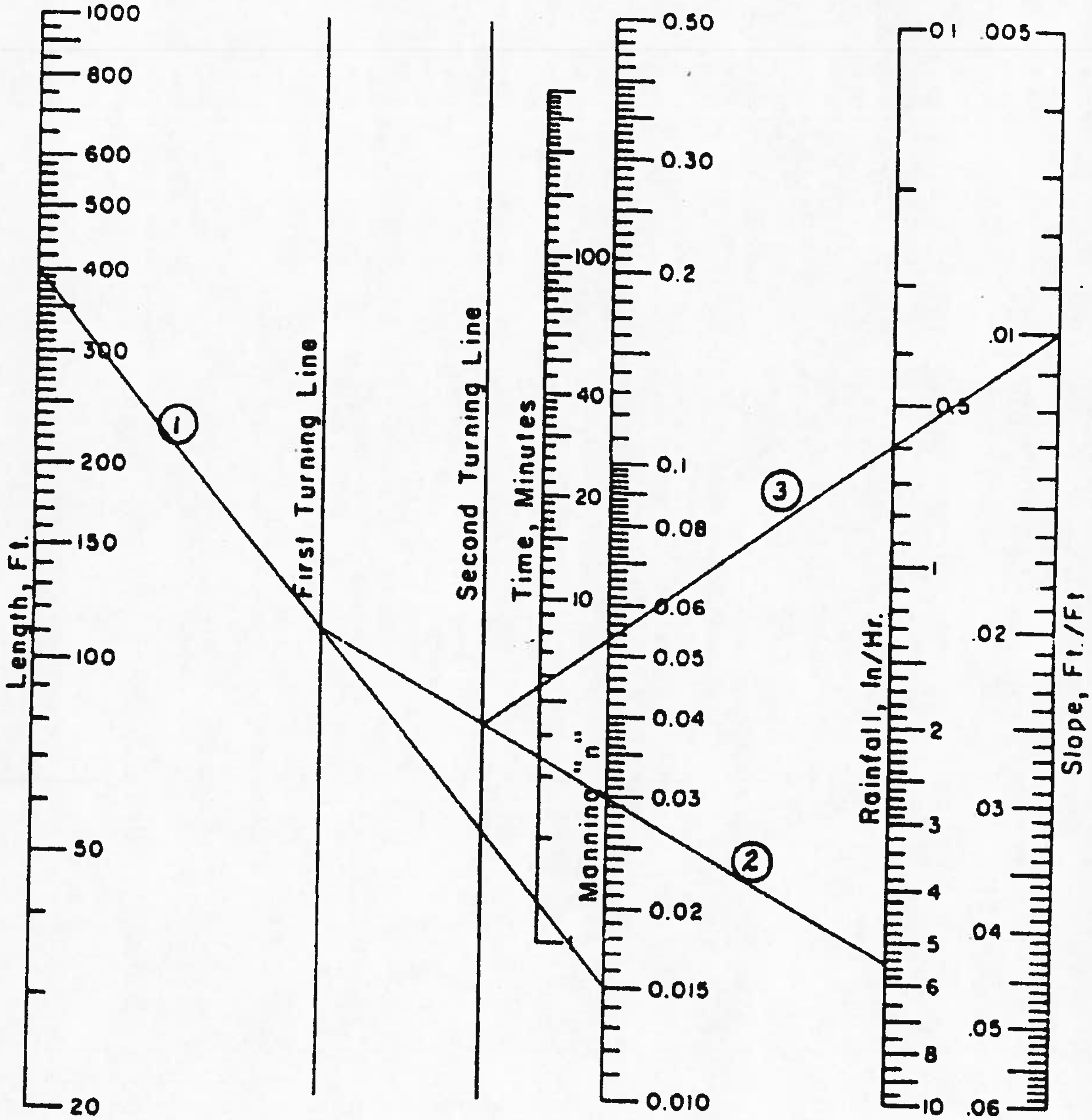


FIGURE 19.9 Time of concentration of small drainage basins (Kirpich method).

Equation solved by nomograph:

$$t_c\,(min) = .94\,\frac{L^{0.6} n^{0.6}}{i^{0.4} S_o^{0.3}}$$



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_o = 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Nomograph for determining time of concentration for overland flow, Kinematic Wave Formulation. *(After Ragan.)*

FIGURE 19.10 Time of concentration for overland flow, Manning's kinematic wave formulation.

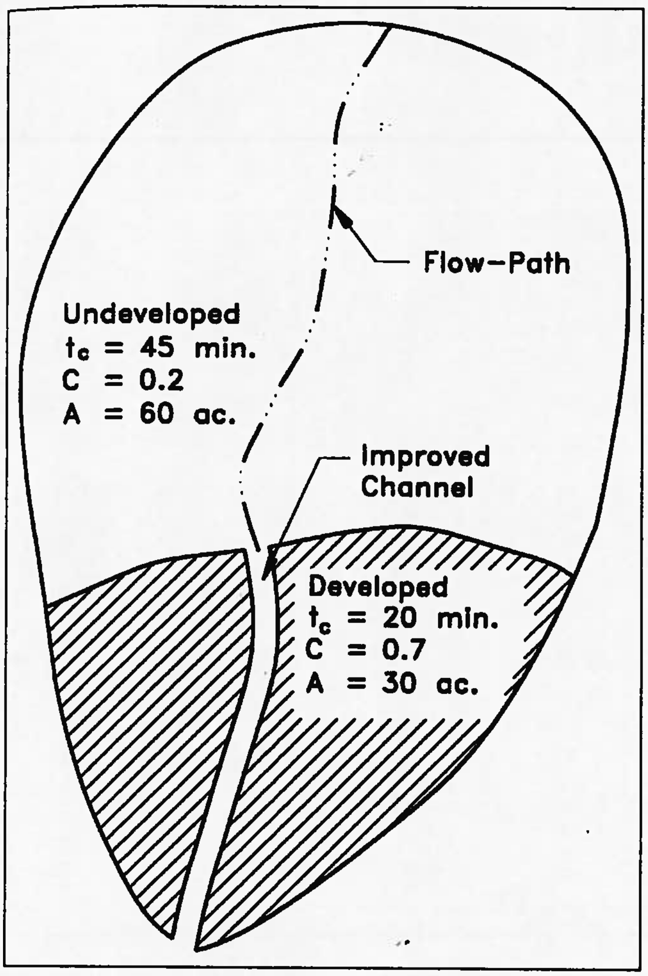


FIGURE 19.11 Catchment with development at the lower end.

■ The maximum rate of runoff occurs when constant rainfall intensity falls on a catchment for as long as, 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 Kuichling in 1877. Runoff coefficients are typically higher for the less frequent storms because of the reduction effect of the rainfall abstractions. Runoff coefficients are also increased for the higher-intensity rainfalls for the same reason.

■ The frequency of the peak discharge is the same as that of the rainfall intensity for the given time of concentration. This may not necessarily be 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 to catchments with time of concentrations greater than 60 minutes.

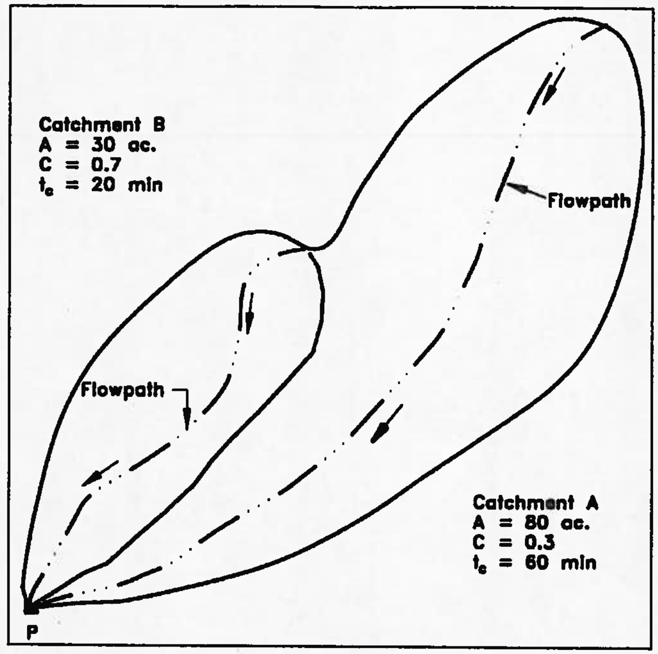


FIGURE 19.12 Composite catchments. (Ponce, Miguel Victor, 1989, *Engineering Hydrology: Principles and Practices*, pp. 128, 129, 138. Adopted by permission of Pearson.)

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 illustrates the procedure. Consider the schematic layout in Figure 19.13. The requirement is to determine the discharge at point *e* using the rational method for a 10-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.[1] 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 (Table 19.10). This example is to illustrate the application of the rational method; hence, the pipe design

[1]Some exceptions to this generalized rule have previously been discussed.

TABLE 19.9 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

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 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 planimetering or otherwise measuring the area.[2] The runoff coefficient is determined from the land use, topography, and other contributing factors. From the IDF curve, shown in Figure 19.8, the rainfall intensity is 6.5 in/hr and the corresponding peak discharge (from Equation 19.4) 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:

$$Q_p = i \sum (CA) = (5.8)\,2.6 = 15.1 \text{ cfs} \tag{19.15}$$

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 21.8 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.

NRCS Methodology for Computing Runoff

For relatively small catchments (<200 acres) the rational method 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 Natural Resources Conservation Service

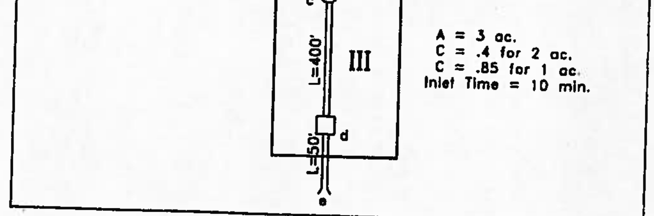


FIGURE 19.13 Schematic for application of the rational method.

[2]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.

TABLE 19.10 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	To	Incr. Area ac	C	$A \times C$	Accum. (ac)	t_c (min) to Upper End (or to inlet)	Rainfall Intensity (in/hr)	Q_p (cfs)	Diameter (in)	L (ft)	Slope	Vel (fps)	Q_{max} (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		2.5		2.0	7.0		
b	c	5	0.2	1.0	1.0	10	5.9	5.9		100		2.5		0.7	10.7		
c	d	—	—	—	2.6	10.7	5.8	15.1		400		2.5		2.7	13.4		
d	e	3	.55	1.65	4.3	13.4	5.3	22.8		50		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_c 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 $\text{Slope} = \dfrac{\text{Upper Inv. Elev} - \text{Lower Inv. Elev}}{L}$
13 Velocity of discharge in pipe = Q_p/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.

(NRCS), is widely accepted, well documented, and available for use on the computer. 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 NRCS methods is found in Technical Release 55 (TR-55), Urban Hydrology for Small Watersheds, and the new TR-55 Win program. These documents are available from the Government Printing Office, Washington, D.C. Because the NRCS method is routinely used for stormwater management design, a brief discussion of the primary components and principles is provided in the following sections.

For the design of larger stormwater management (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 (PMF), 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.

Rainfall. The NRCS has developed four synthetic rainfall distributions that are indicative of the rainfall intensities inherent to geographic regions of the United States. These four standard rainfall distributions, labeled Type I, Type Ia, Type II, and Type III, have been developed from numerous publications. Since most rainfall data is reported on a 24-hour basis, the NRCS used 24 hours as the duration for these distributions. The location of the peak rainfall intensity (early, center, or late peaking) in each storm is intended to mimic the location of the peak intensity for the particular region of the United States. 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 United States. Type II and III storms have peak intensities occurring around the midpoint of the duration. Specific geographical areas are shown in Figure 19.14.

Runoff Volume Using Runoff Curve Numbers. The fundamental equation in the NRCS hydrologic method for computing the volume of runoff from a catchment area is:

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \tag{19.16}$$

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 (i.e., losses before runoff begins).

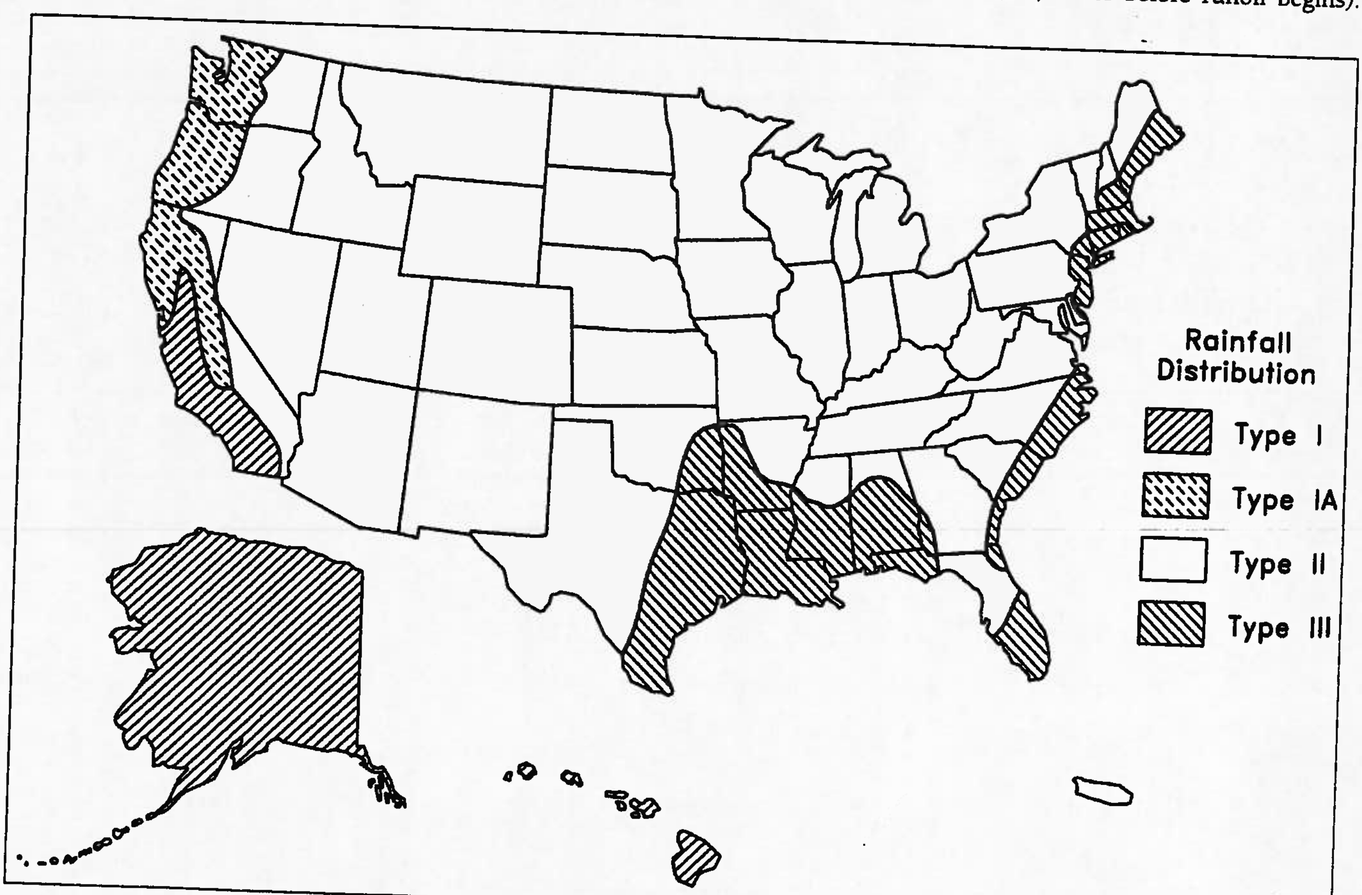


FIGURE 19.14 Approximate geographic boundaries for NRCS rainfall distributions. (*Source:* USDA, TR-55)

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:

$$S=\frac{1000}{CN}-10 \tag{19.17}$$

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 these relationships, the runoff can be expressed in terms of S:

$$Q=\frac{(P-0.2\,S)^2}{P+0.8\,S} \tag{19.18}$$

In part, the *CN* for a particular soil depends on the HSG classification. Soils are divided into four hydrologic soil groups, A, B, C, and D, according to their minimum infiltration rate. Soils classified in hydrologic group A generally have high infiltration rates (sand), whereas the HSG D has the lowest infiltration rates (clay). 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, specialized photography (infrared, etc.), and land use maps. Hydrologic condition (poor, fair, or good) is a measure of the effects of the cover type on infiltration and runoff.

Table 19.11 presents CN values for several types of soils and cover types. It should be noted that these CN values are based on an average antecedent runoff condition. The NRCS publishes soil surveys for the majority of localities in the United States and contains soil classification information. The surveys are found in many different formats including databases and GIS layers. The NRCS is constantly updating information available electronically; refer to http://soils.usda.gov/ for the latest information.

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 coverage of each set of conditions and consulting an NRCS curve number table. Figure 19.15 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 19.16 is used to tabulate the data and determine the composite CN.

Time of Concentration. The NRCS method for determining t_c consists of computing the travel times associated with runoff over three distinct types of flowpaths, as described here.

1. Sheet flow is the initial phase of runoff characterized as flow over plane surfaces. The flow depth is very shallow (<0.1 ft); consequently, the Manning's roughness coefficient is modified to reflect the increased effects of drag from surface irregularities. Sheet flow is assumed to occur for distances less than 300 ft. The NRCS has issued recent guidance suggesting that 100 ft is the likely maximum distance for sheet flow calculations. The travel time is given as:

$$T_t=\frac{0.007\,(nL)^{0.8}}{(P_2)^{0.5}\,S^{0.4}} \tag{19.19}$$

which is the kinematic solution to Manning's equation. In Equation 19.19, T_t represents the travel time in hours, n is the effective Manning's n as given in Table 19.12, L is the flow length in feet (<300 ft), P_2 is the 2-year/24-hour rainfall (in) given in Figure 19.3, and s is the slope of the hydraulic grade line, which is assumed to be the same as the average land slope (ft/ft). See Table 19.12.

2. Shallow concentrated flow occurs after sheet flow. The travel time for shallow concentrated flow is:

$$T=\frac{L}{3600\,V} \tag{19.20}$$

where $V=16.1345\,(s)^{0.5}$ for unpaved surfaces
$20.3282\,(s)^{0.5}$ 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, which, according to NRCS, are visible on aerial photographs, appear as blue lines on USGS quadrangle sheets, and dissipate where surveyed cross-section information begins, or based on field verification. Travel time in open channels is determined from the average velocity at bank-full flow. Manning's equation for open channel velocity is:

$$V=\frac{1.49}{n}\,R^{2/3}\,S^{1/2} \tag{19.21}$$

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 Equation 19.20 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 whether 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 func-

TABLE 19.11A Runoff Curve Numbers for Urban Areas[1]

COVER DESCRIPTION		CURVE NUMBERS FOR HYDROLOGIC SOIL GROUP			
Cover Type and Hydrologic Condition	**Average % Impervious Area[2]**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
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)[4]		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:					
⅛ acre or less (town houses)	65	77	85	90	92
¼ acre	38	61	75	83	87
⅓ acre	30	57	72	81	86
½ acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 19.11c).					

[1]Average runoff condition and $I_a = 0.2S$. For range in humid regions, use Table 19.11b.
[2]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 19.14 or 19.15.
[3]CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open-space cover type.
[4]Composite CNs for natural desert landscaping should be computed using Figure 19.14 or 19.15 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.
[5]Comparable CNs to use for the design of temporary measures during grading and construction should be computed using Figure 19.14 or 19.15, based on the degree of development (impervious area percentage) and the CNs for the newly graded pervious area.
Source: USDA, TR-55.

TABLE 19.11B (Continued)

COVER DESCRIPTION		CURVE NUMBERS FOR HYDROLOGIC SOIL GROUP			
Cover Type	**Hydrologic Condition[1]**	**A[2]**	**B**	**C**	**D**
Pasture, grassland, or range-continuous forage for grazing[1]	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[2]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[3]	48	65	73
Woods-grass combination (orchard or tree farm)[4]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[5]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[3]	55	70	77
Farmsteads-buildings, lanes, driveways, and surrounding lots	—	59	74	82	86

[1]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.
[2]Poor: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
[3]Actual curve number is less than 30: use CN = 30 for runoff computations.
[4]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.
[5]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.

tion of time. In the most simplistic case, a hydrograph has a rising limb that reflects rainfall characteristics, a crest segment, and a recession curve that reflects watershed characteristics. A hydrograph may be classified as a natural hydrograph, one that is derived from observed data from a stream flow gauge, or it may be classified as a synthetic hydrograph, one that 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 19.17):

- *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. 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 definitions, the hydrograph shape is affected by the intensity, duration, and distribution (both temporally 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

TABLE 19.11C (*Continued*)

COVER DESCRIPTION		CURVE NUMBERS FOR HYDROLOGIC SOIL GROUP			
COVER TYPE	HYDROLOGIC CONDITION[1]	A[2]	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

[1]Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: >70% ground cover.
[2]Curve numbers for group A have been developed only for desert shrub.
Source: USDA, TR-55.

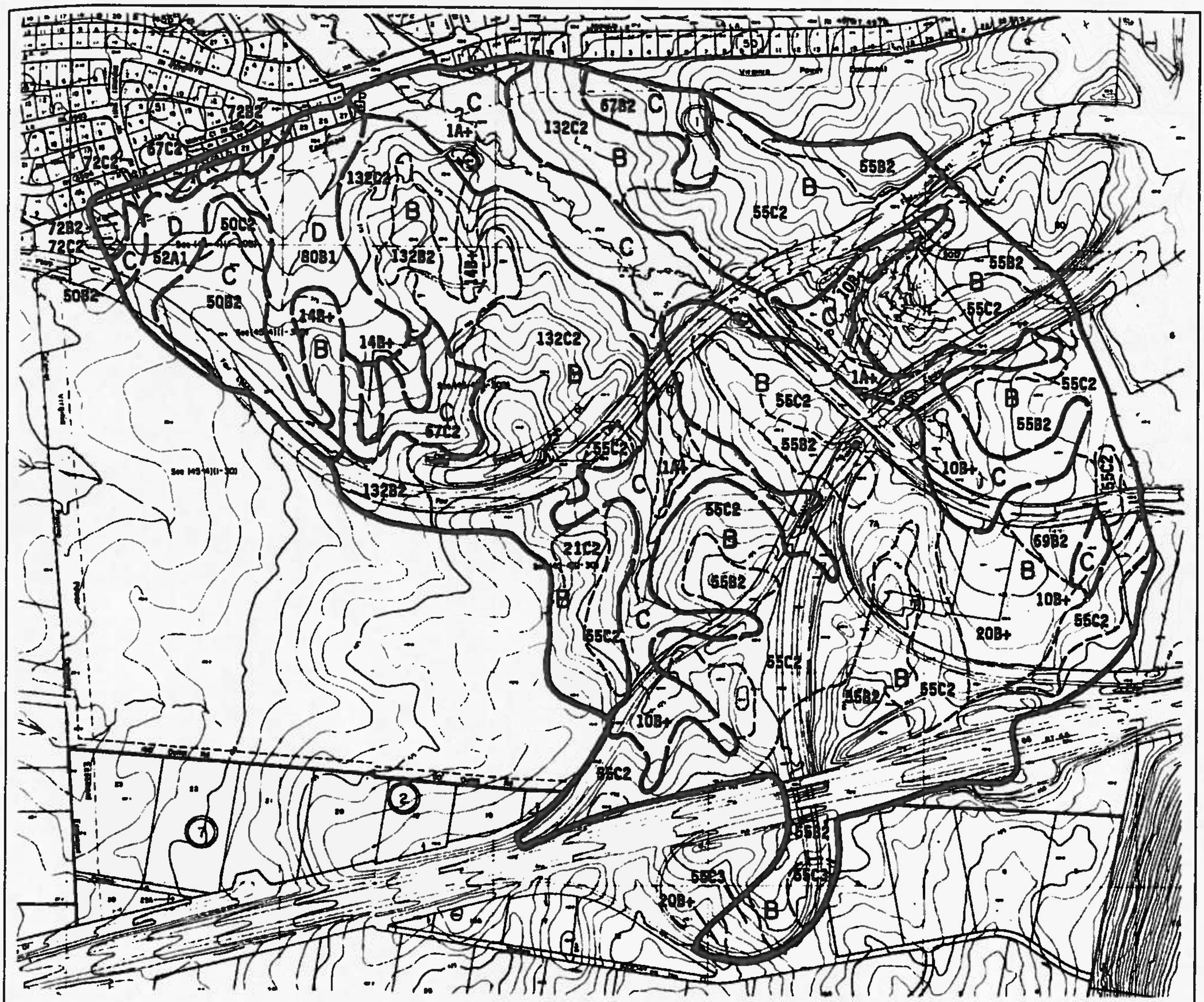
longer time to peak. These concepts are illustrated in Figure 19.18.

Generally, land development activities will decrease the time of concentration and increase the imperviousness for a basin. These changes decrease the time to peak discharge as well as increase the peak discharge and the total runoff volume; it is these increases that must be mitigated with stormwater management methods. A low-impact development (LID) or sustainable design approach to stormwater management seeks to replicate the predeveloped hydrologic conditions and reduce these inherent increases related to site development. By employing a variety of SWM techniques or other site features that maintain or even elongate the time of concentration or improve the land cover condition, the structural SWM facilities required can be minimized—see "Stormwater Management Facility Design" later in this chapter and Chapter 22 for additional guidelines on stormwater management facility design.

Unit Hydrographs. A unit hydrograph (UH) is defined as a runoff hydrograph generated from a unit depth of rainfall excess occurring at a constant 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, that is, a 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 inch over a duration of *D* hours (see Figure 19.19).

The physical features of a watershed vary 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 shape similar to 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-addition procedures (convolution) to obtain the direct runoff hydrograph. This can be done only if the assumptions of linearity are valid, that is, the time base remains constant regardless of the runoff depth. Figure 19.19 shows how to obtain a direct runoff hydrograph using the multiplication-translation-addition procedure.



HYDROLOGIC SOILS GROUPS

CONVERSION OF SOIL TYPES TO SOIL GROUPS					
SOIL TYPE	SOIL NAME	SOIL GROUP	SOIL TYPE	SOIL NAME	SOIL GROUP
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 19.15 Hydrologic soils designation on topographic mapping.

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[1] Table 15-1	CN[1] Fig. 2-3[2]	CN[1] Fig. 2-4[2]	Area ☐ acres ☐ mi² ☐ %	Product of CN x area
				Totals =		

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = ____ = ______ ; Use CN = ☐

2. Runoff

	Storm #1	Storm #2	Storm #3
Frequency .... yr			
Rainfall, P (24-hour) .... in			
Runoff, Q .... in			

FIGURE 19.16 Runoff curve number worksheet. (Source: TR-55)

TABLE 19.12 Roughness Coefficient for Sheet Flow

SURFACE DESCRIPTION	n*
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.005
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[†]	0.24
Bermuda grass	0.41
Range (natural)	
Woods:[‡]	
Light underbrush	0.40
Dense underbrush	0.80

*The n values are a composite of information compiled by Engman (1986).
[†]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[‡]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.
Source: USDA, TR-55.

Mathematically the ordinates for the direct runoff hydrograph for a complex storm pattern are found by:

$$\begin{aligned} q_1 &= p_1 u_1 \\ q_2 &= p_2 u_1 + p_1 u_2 \\ q_3 &= p_3 u_1 + p_2 u_2 + p_1 u_3 \\ q_4 &= p_4 u_1 + p_3 u_2 + p_2 u_3 + p_1 u_4 \\ q_5 &= p_5 u_1 + p_4 u_2 + p_3 u_3 + p_2 u_4 + p_1 u_5 \\ &\cdots \\ &\cdots \\ &\cdots \end{aligned} \tag{19.22}$$

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, Equation 19.22 is written as:

$$[Q] = [P][U]$$

$$\begin{bmatrix} q_1 \\ q_2 \\ q_3 \\ q_4 \\ q_5 \\ \cdot \\ \cdot \\ \cdot \end{bmatrix} = \begin{bmatrix} p_1 & 0 & 0 & 0 & 0 & 0 & \ldots \\ p_2 & p_1 & 0 & 0 & 0 & 0 & \ldots \\ p_3 & p_2 & p_1 & 0 & 0 & 0 & \ldots \\ p_4 & p_3 & p_2 & p_1 & 0 & 0 & \ldots \\ p_5 & p_4 & p_3 & p_2 & p_1 & 0 & \ldots \\ \cdot & & & \cdot & & & \\ \cdot & & & \cdot & & & \\ \cdot & & & \cdot & & & \end{bmatrix} \begin{bmatrix} u_1 \\ u_2 \\ u_3 \\ u_4 \\ u_5 \\ \cdot \\ \cdot \\ \cdot \end{bmatrix} \tag{19.23}$$

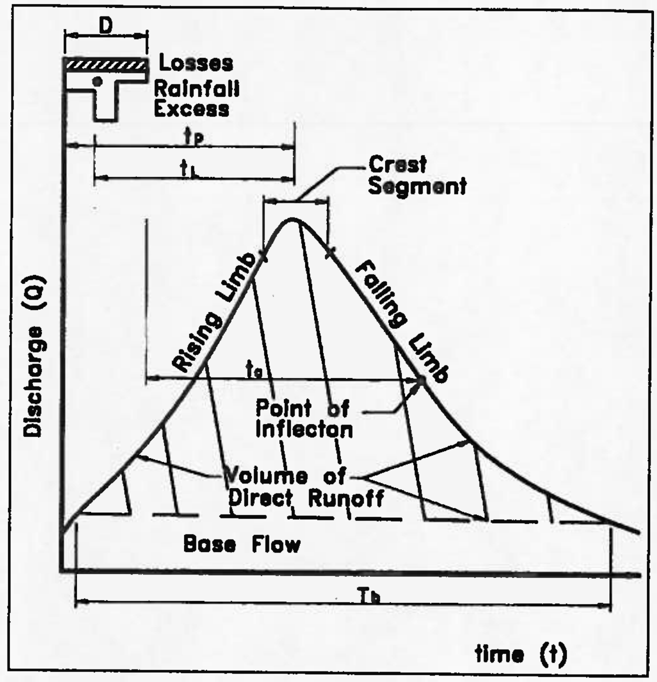


FIGURE 19.17 Elements of a hydrograph.

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 NRCS method and the modified rational formula, Snyder's unit hydrograph, Clark's unit hydrograph, and numerous others.

The NRCS Dimensionless Unit Hydrograph. The NRCS and others have developed natural unit hydrographs from data collected from catchments of widely varied sizes and locations, which were then converted to an "average" synthetic curvilinear dimensionless unit hydrograph, shown in Figure 19.20. This is usually done in relatively small geographic regions where significant hydrologic data exists—for exam-

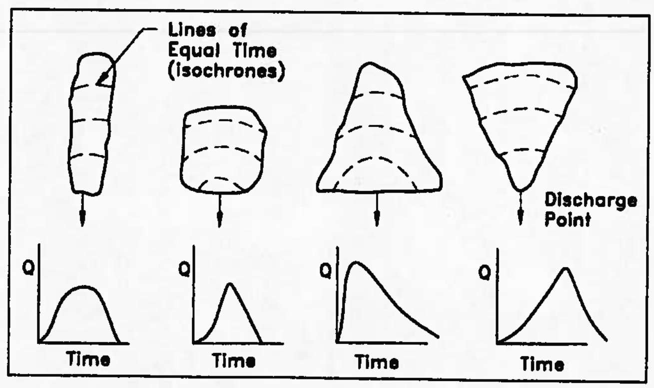


FIGURE 19.18 Effect of basin shape and t_c on hydrograph shape. (Courtesy of Wanielista, Martin P. *Hydrology and Water Quality Control*. New York: John Wiley & Sons. Reprinted by permission of John Wiley & Sons, Inc.)

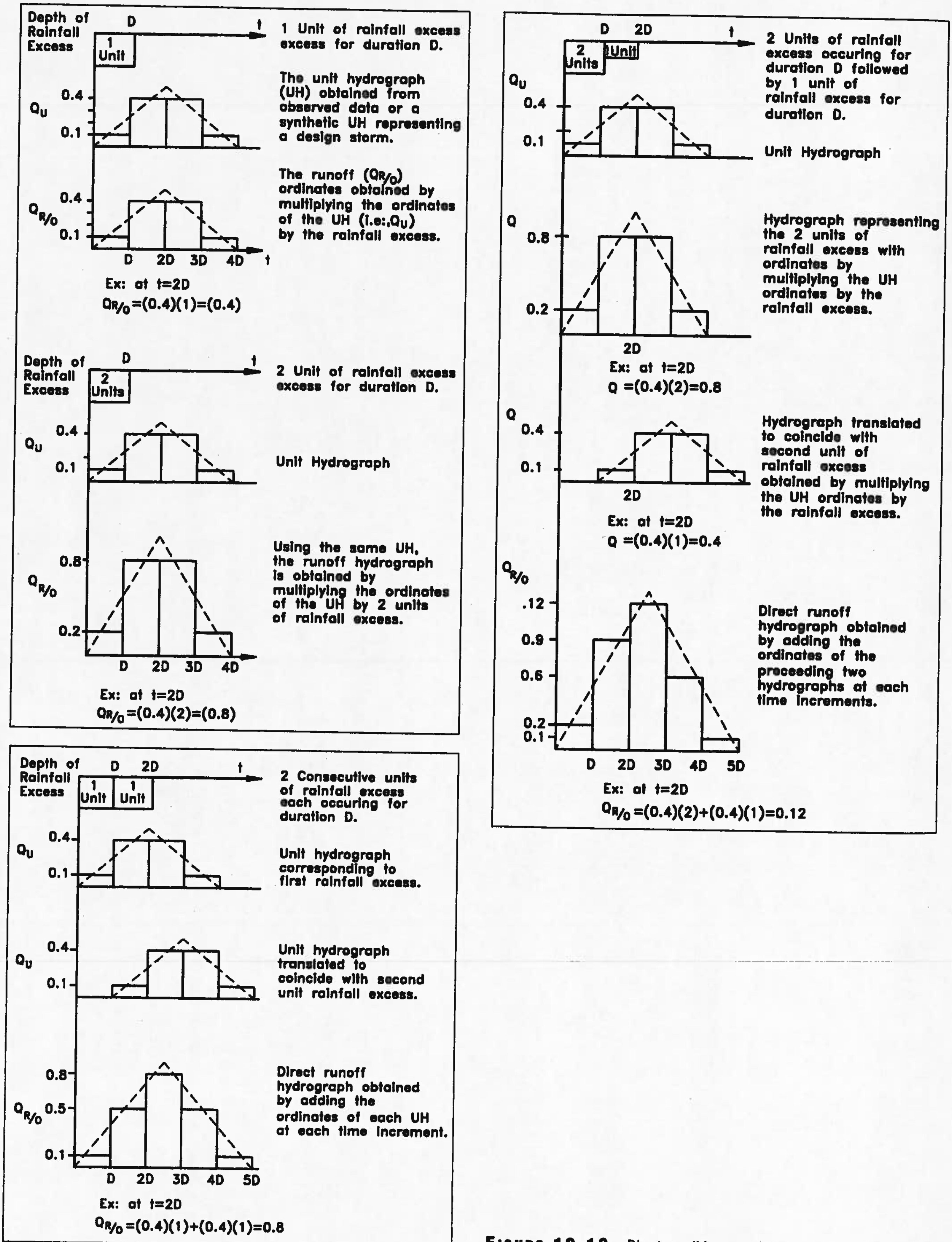


FIGURE 19.19 Direct runoff from a unit hydrograph.

ple, on the Delmarva peninsula. The engineer should contact the local NRCS office to confirm the applicable unit hydrograph. 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 identical. 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:

$$Q = \frac{1}{2}\, q_p (T_p + T_r) \tag{19.24}$$

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 rewritten as:

$$q_p = \frac{484AQ}{T_p} \tag{19.25}$$

where q_p is the peak discharge for the unit hydrograph, A is drainage area in square miles, and 484 is included to place ⅜ 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 peak rate factor. The peak rate factor has been known to vary from 600 for mountainous regions to 200 in very flat swampy areas. The local NRCS office should be contacted to determine whether 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:

$$T_p = \frac{D}{2} + 0.6t_c = \frac{2}{3}\, t_c \tag{19.26}$$

With proper substitution into Equation 19.25, q_p can be expressed in terms of t_c as:

$$q_p = \frac{726AQ}{t_c} \tag{19.27}$$

EXAMPLE 1

Determine the NRCS synthetic UH for a predominantly commercial catchment with the following characteristics: area = 200 acres, average land slope = 4 percent, t_c = 20 minutes, and HSG B.

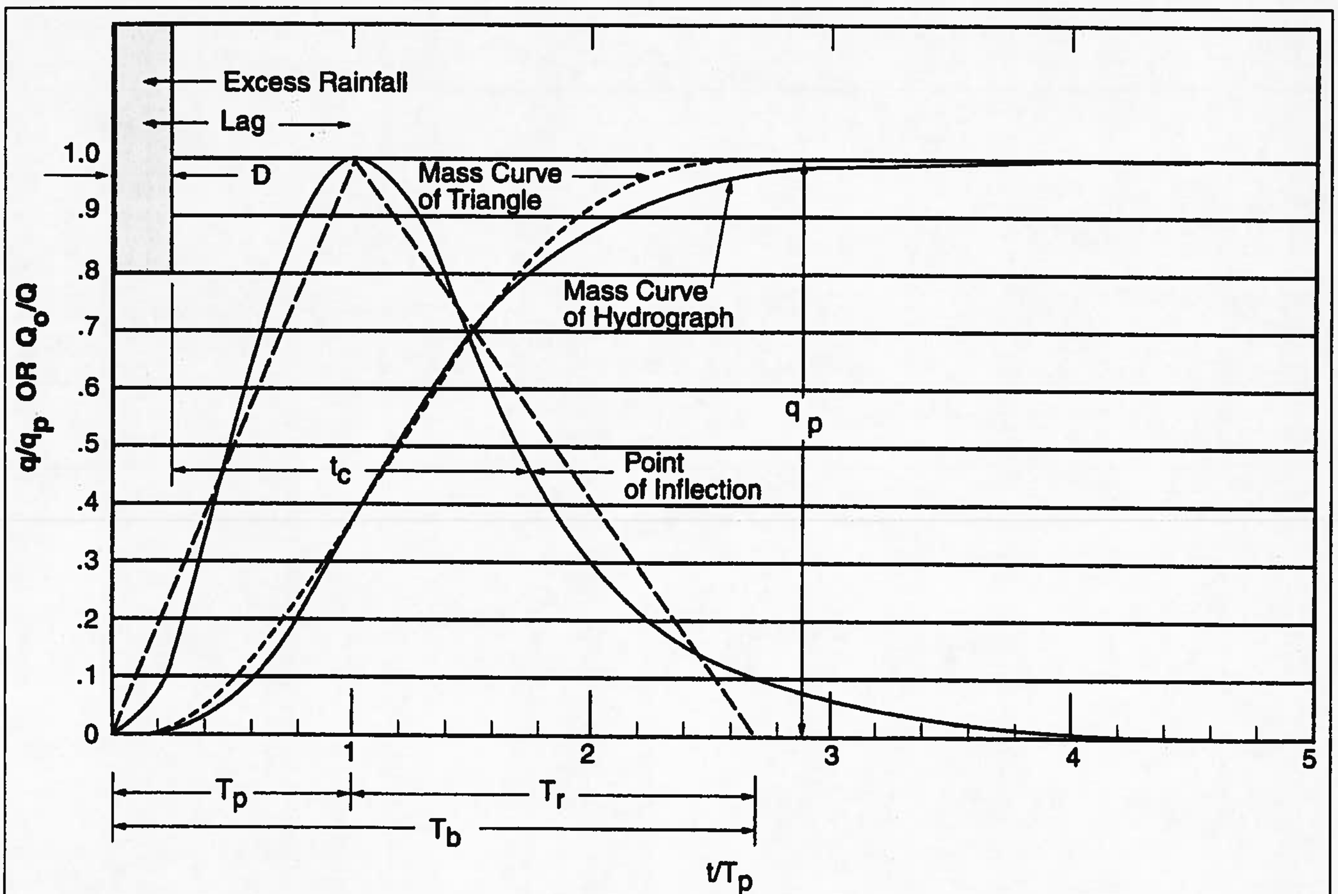


FIGURE 19.20 Dimensionless curvilinear UH with NRCS's triangular UH. (*Source:* USDA, NEH-4.)

From Table 19.11, the CN for commercial land use and HSG B is 92. From Equation 19.27, the peak discharge for the UH is:

$$q_p = \frac{726\,(200\text{ ac})\,(1\text{ in})}{(640\text{ ac/mi}^2)\left(\dfrac{45\text{ min}}{60\text{ min/hr}}\right)} = 303\text{ cfs} \tag{19.28}$$

The time to peak is:

$$t_p = \frac{2}{3}\,t_c = \frac{2}{3}\,(20\text{ min}) = 13.3\text{ min} \tag{19.29}$$

The time base is:

$$t_b = \frac{8}{3}\,t_p = \frac{8}{3}\,(13.4\text{ min}) = 35.7\text{ min} \tag{19.30}$$

The triangular hydrograph is shown in Figure 19.21.

Computer Models for Rainfall-Runoff Relationships. Advances in computer software applications (CADD-based and otherwise) have made hydrologic modeling easier and quicker. Although many designers use these software programs as a "black box," it is recommended that users have some understanding of the hydrologic processes as well as 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 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 perform hydrology and manipulate hydrographs. These models may be used to generate hydrographs from either synthetic or historical design storms, combine hydrographs together, and perform storm drain design and channel and pond routings. The type of computer model selected depends on the user, available data, and, possibly, any preferences by the reviewing agencies.

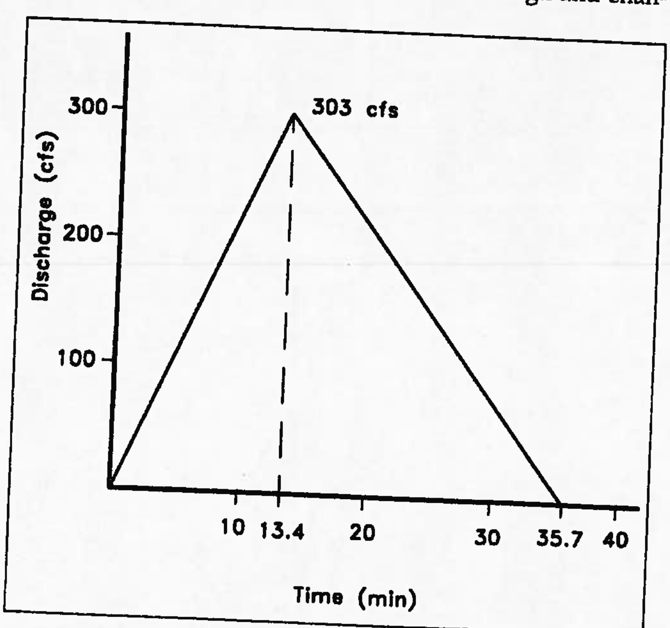


FIGURE 19.21 Synthetic triangular unit hydrograph for example.

Some of the more popular models in use are listed in Table 19.13.

NRCS Graphical Peak Discharge. A peak discharge can be obtained from a direct runoff hydrograph. When only a peak discharge is needed, the NRCS's graphical peak discharge method can be used instead of developing an entire hydrograph. NRCS's graphical method for determining the peak discharge, appropriate for homogeneous (i.e., uniform CN) watersheds and for 0.1 hr $\le t_c \le$ 10 hr, is found by:

$$q_p = q_u A_m Q F_p \tag{19.31}$$

where q_p = peak discharge (cfs), q_u = unit (hydrograph) peak discharge (ft^3/mi^2/in), A_m = drainage area (mi^2), Q = runoff (in, from Equation 19.16), and F_p = pond and swamp adjustment factor (from Table 19.14).

Before engaging in this method, consider the following limitations:

- The watershed should be homogeneous with respect to the CN.
- The weighted CN should be greater than 40, $0.1 \le t_c \le 10$ hr, and $0.1 \le I_a/P \le 0.5$.
- 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.

To compute the peak discharge for a watershed:

1. From appropriate maps obtain the 24-hour rainfall P for the design storm.
2. Determine the CN for the watershed.
3. Determine I_a from:

$$I_a = 0.2\left(\frac{1000}{CN}\right) - 10 \tag{19.32}$$

4. The unit peak discharge (q_u) in cubic feet per square mile per inch of runoff is determined from:

$$\log(q_u) = C_0 + C_1\log(t_c) + C_2[\log(t_c)]^2 \tag{19.33}$$

where coefficients for C_0, C_1, and C_2 are given in Table 19.15 for I_a/P ratios. Figure 19.22 shows the graphical representation of Equation 19.33 for the NRCS type II rainfall distribution.

5. Determine the pond factor F_p, if applicable.
6. Use Equation 19.31 to find the peak discharge.

TABLE 19.13 Summary of Selected Computer Runoff Models

Attribute (1)	Model: DR3M-QUAL (2)	HSPF (3)	ILLUDAS (4)	Penn State (5)	Statistical (6)	STORM (7)	SWMM (8)	TR55 (9)	HEC-1 (10)
Sponsoring agency	USGS	EPA	Ill. State Water Survey	OWRT and City of Phil.[a]	EPA	HEC	EPA	SCS	HEC
Simulation type[b]	C, SE	C, SE	SE	SE	N/A	C	C, SE	SE	Se
No pollutants	4	10	None[c]	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[d]	N	N
Surcharge	Y[e]	N	Y[e]	N	N/A	N	Y[d]	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	
Storage analysis	Y	Y	Y	Y	Y[f]	Y	Y	Y	
Treatment analysis	Y	Y	N/A	N/A	Y[f]	Y	Y	N/A	
Suitable for planning (P), design (D)[g]	P, D	P, D	D	D	P	P	P, D	D	
Available on microcomputer	N	Y	Y	Y	Y[h]	N	Y	Y	
Data and personnel requirements	Medium	High	Low	Low	Medium	Low	High	Medium	
Overall model complexity[i]	Medium	High	Low	Low	Medium	Medium	High	Low	

Reset with permission from Water Environment Federation and American Society of Civil Engineers, *Design and Construction of Urban Stormwater Management Systems*, 1992.
[a]Currently supported by Penn State University.
[b]Y = yes, N = no, N/A = not applicable, C = continuous simulation, SE = single event simulation.
[c]Undocumented quality routines added during applications.
[d]Full dynamic equations and surcharge calculations only in Extran Block of SWMM.
[e]Surcharge simulated by storing excess inflow at upstream end of pipe. Pressure flow not simulated.
[f]Storage and treatment analyzed analytically. See references in Section 3.7.9.
[g]See Section 3.3.
[h]See Driscoll et al. 1989.
[i]General requirements for model installation, familiarization, data requirements, etc. To be interpreted only very generally.
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[e], illudas column = Y[e], available column = Y[f].

TABLE 19.14 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.

Storm Drainage Design

Following the development of the general site layout and hydrologic analysis of the site, the horizontal placement and sizing of the storm drainage system are performed. Conveyance systems may vary from nonstructural systems such as vegetated filter strips, rain gardens, and swales to structural systems comprising pipes, culverts, and inlets. The selection of what type of components will constitute a system may vary due to many factors, including overall development objectives, such as low-impact development (LID) and green building objectives, available site area for such components, topography, soils, and sensitive area concerns, as well as local jurisdictional requirements. More and more jurisdictions are requiring consideration and implementation of LID techniques, nonstructural stormwater management strategies, and widely accepted best management practices (BMPs). The design engineer must evaluate and give consideration to these objectives and incorporate these various design elements as appropriate.

Once the type of conveyance and collection system has been established, these systems are incorporated into the site

TABLE 19.15 Coefficients for Determining q_u

Rainfall Type	I_a/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	−0.11500
	0.50	2.17772	−0.36803	−0.09525

Source: USDA, TR-55.

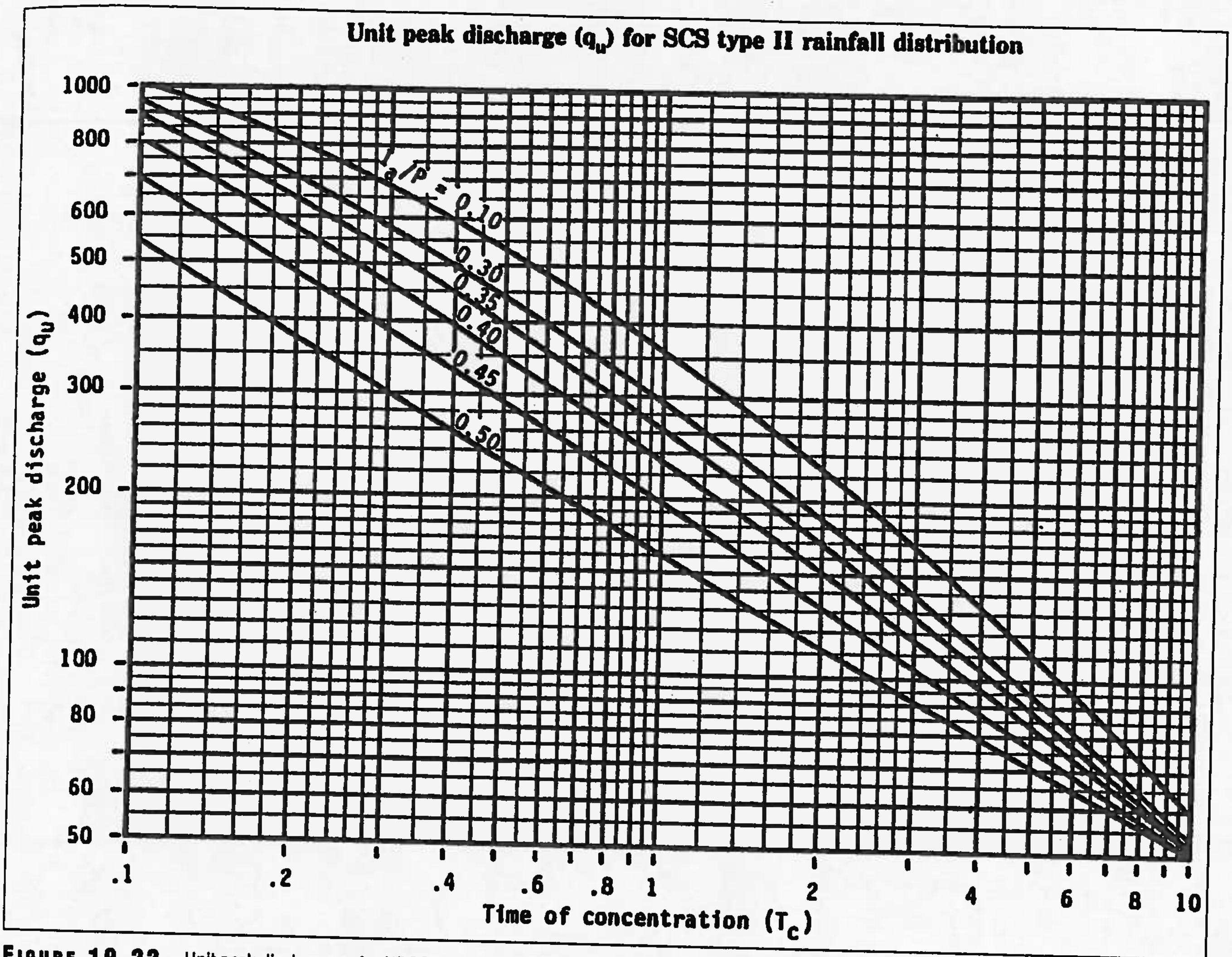


FIGURE 19.22 Unit peak discharge q_u for NRCS type II rainfall distribution. (*Source:* USDA, TR-55)

plan as the grading and drainage plan is developed. Preliminary sizing should be performed on each component to provide adequate conveyance of stormwater runoff to proposed stormwater management facilities or points of discharge (outfalls). It is important to verify the design storm and sizing requirements for these systems, with local authorities having jurisdiction. However, the designer must also account for safe runoff conveyance either via overland flow, via the collection system, or via other secondary means of conveyance for storm events exceeding the required design storm for the collection system. This is necessary to ensure the stormwater management facilities will actually receive the runoff from the higher-intensity storm events they are designed to attenuate. 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 (natural or man-made) should obtained.

Stormwater Management Facility Design

Designing stormwater management (SWM) facilities is part of the preliminary engineering stage of the design process. In order to size the facilities, it is important to determine the specific performance requirements for the facility. These may include water quality or pollution removal, ground water recharge, and quantity control (detention/retention) requirements for the proposed development. These requirements vary greatly from region to region within the country and even at the state and local levels. Although the exact configurations and grading of such facilities are subject to change during final engineering, it is important to ensure that adequate provisions have been made to accommodate and locate these facilities to achieve the required stormwater management objectives.

Stormwater management facilities are commonly grouped into several facility types, which are selected based on site considerations, performance requirements, aesthetics, and other factors (cost). These facilities usually fall into one of the following types:

- Detention basin (i.e., dry pond for the temporary impoundment of surface water runoff)
- Retention basin (i.e., wet pond that maintains a permanent pool of water with additional storage volume above the permanent pool for detaining runoff)
- Infiltration facilities
- Structural facilities

Exact design parameters for each of these facility types may vary depending on local requirements, and often combinations of these generalized facility types are used together to achieve performance goals. For instance, a stormwater management facility may employ infiltration of runoff as a means to provide ground water recharge and water quality functions, while providing controlled outflow for larger storms. Similarly, a wet pond may be used for water quality treatment in conjunction with extended detention to reduce peak runoff rates. If runoff volume control is required, the postdevelopment stormwater management program will have to include permanent storage facilities (retention), infiltration facilities, stormwater harvesting/reuse facilities (typically structural), or some combination of the three in order to minimize the total volume of runoff leaving the site. The designer must evaluate and determine the most appropriate facility or combination of facilities to achieve the applicable performance requirements.

Locating SWM Facilities. The location of SWM facilities should be integrated with the site design, either to minimize the impact on the development project or to enhance the development in terms of function and aesthetics. If possible, an SWM facility should be located so that runoff drains into it naturally, without requiring additional engineering measures such as storm sewers or channels to artificially force drainage divides. The most economical design for any SWM facility is one that requires the least earthwork and fewest structural components. Further, facilities should be located with respect to outfalls in order to maintain natural (existing) drainage divides as well as protect and improve the condition of outfalls through adequate capacity analysis and erosion reduction.

Preliminary Sizing of SWM Facilities. The basic steps for sizing and designing a stormwater management facility for water quantity control purposes are:

1. Determine facility location and type (e.g., retention or detention).

2. Develop inflow hydrographs for the design storms (predevelopment and postdevelopment). Localities may mandate that the facility must attenuate postdevelopment peak discharges for several design storms.

3. Determine maximum postdevelopment release rates based on local design standards.

4. Estimate the amount of storage required for each of the design storms (see "Estimating the Volume of Storage Required").

5. Grade earthen facilities or size (volumetrically) structural facilities to accommodate the necessary storage and develop stage-storage and stage discharge curves (see Chapter 22 for additional guidance).

6. Estimate water surface elevations (WSEs) for design storms (based on results from steps 4 and 5); determine freeboard (clear space above highest design storm WSE) requirements and design facility embankment (top of dam) or structural rooftop; assess feasibility of fully developed facility with grading design.

7. Select preliminary size and configuration of principal outlet control structure (risers, spillways, and/or clearwell arrangement).

8. Perform hydrologic routing computations to verify preliminary design features.

9. Based on results obtained in step 8, modify basin grading (or volumetric size) and outlet structure to optimize releases for various design storms.

10. Determine other design requirements (trash racks, antiseep collars, embankment seepage control elements, bypass structures, etc.) and the need for emergency overflow relief and design accordingly.

When sizing a water quality facility, the process can be simplified, as most water quality or BMP facilities are sized/designed based on a specific water quality volume (WQv). Depending on geography, climate, and jurisdictional requirements, the WQv usually ranges from 0.5 inch to 1 inch of rainfall per impervious acre. Once the facility location, type, and WQv are determined, design of the facility can proceed through steps 5, 7, and 10. Certain technology-based BMPs are sized/designed based on water quality storm flow rate as opposed to a WQv; these facilities should be sized according to manufacturer recommendations and any additional jurisdictional guidelines that specifically address the use of these BMPs. Additional guidance on SWM facility design is provided in Chapter 22.

Special consideration must be given to the spillway capacity of the stormwater management facilities, particularly in the case of dam embankments for surface facilities. Oftentimes the spillway capacity, which is related to the hazard classification of the facility, must be increased if the facility is found to be a hazard to downstream insurable structures. A hazard classification should be determined and, if necessary, a dam break analysis performed to analyze potential impacts downstream from the facility. Such analysis may determine that off-site easements are required downstream for a breach

These easements will need to be pursued by the owner or developer.

Estimating the Volume of Storage Required. When initially sizing an SWM facility, the required storage volume to meet detention requirements 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 19.23).

Once an initial estimate of the required volume has been made and the location determined, a preliminary grading plan (for earthen or surface facilities) or volumetric design (for structural or infiltration-based facilities) is performed. In preparing the grading plan, the objective is to obtain the preliminary storage volume while keeping in mind such things as minimizing earthwork, nominal height requirements of the embankment, depth and clear height (confined space) limitations for safety, sediment storage, aquatic vegetation, depth to ground water table, cost, and aesthetic considerations.

A graph of the stage-storage relationship is then developed. The stage-versus-storage relationship is derived by measuring the area within the pond or structure at selected elevations and estimating the storage volume based on the area and depth (and often, in the case of underground structural infiltration facilities, the void ratio of the media in conjunction with the structural storage). The areas can be measured with a planimeter on a contour map, with CADD software area measurement tools, or by other engineering methods for determining volume (see Chapter 23 for more information). Two different equations are commonly used to compute the volume: the average end area

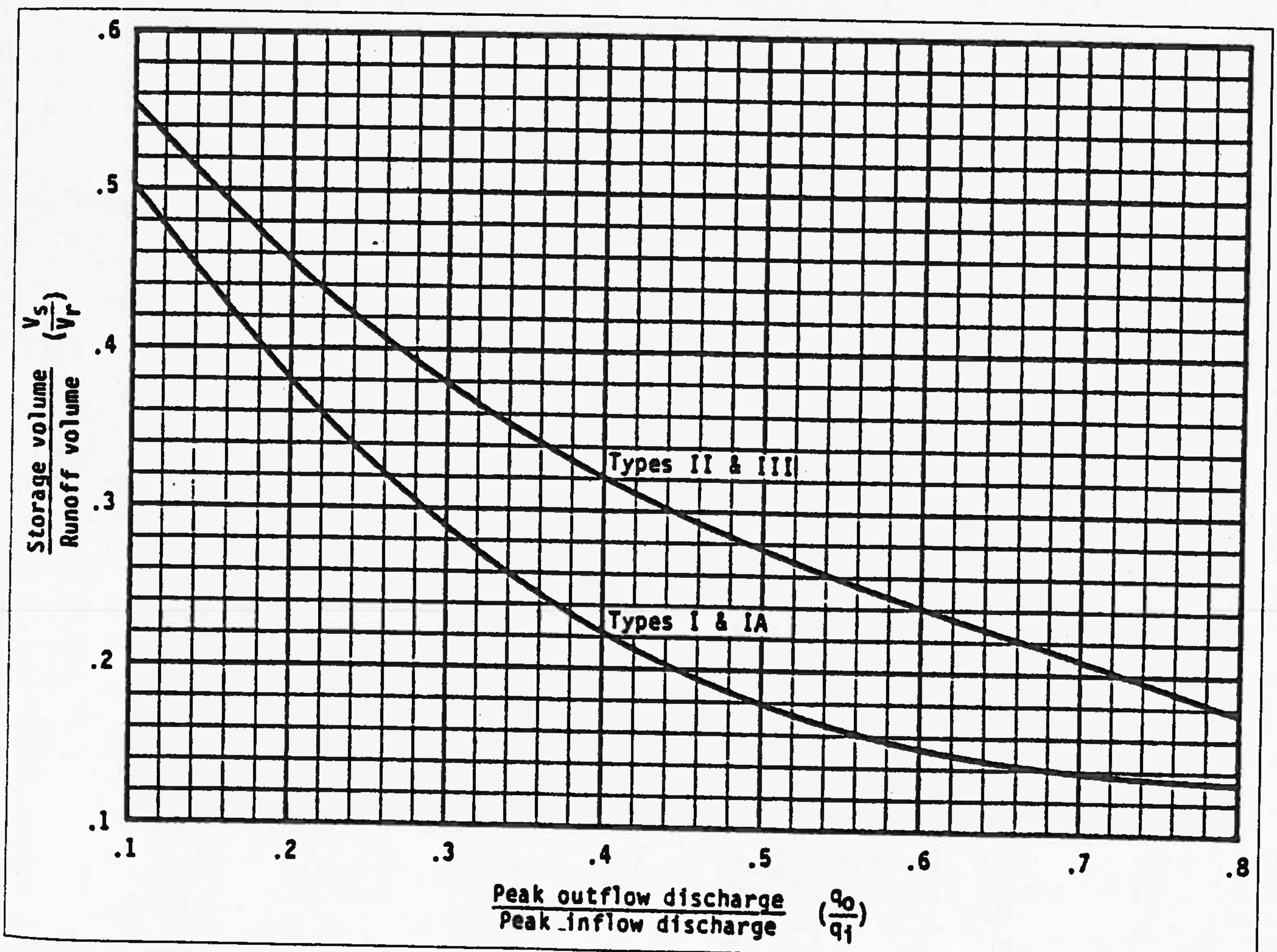


FIGURE 19.23 Approximate detention basin routing for synthetic rainfall distribution. (Source: USDA, TR-55)

method (Equation 22.19) and the conic approximation method (Equation 22.20). The conic approximation method is used in some computer methods such as HEC-1.

Average End Area Method:

$$V_{inc} = \frac{A_n + A_{n+1}}{2} \times h \tag{19.34}$$

Conic Approximation Method:

$$V_{inc} = \frac{A_n + A_{n+1} + \sqrt{A_n \times A_{n+1}}}{3} \times h \tag{19.35}$$

where V_{inc} = incremental volume in pond between contour lines representing A_n and A_{n+1}, A_n = measured area of nth contour line, A_{n+1} = measured area of the successive contour line, and h = elevation difference between the two contour lines.

Many stormwater management system design software packages are capable of performing preliminary sizing estimates for detention basins, utilizing these methods. They can be helpful in streamlining the evaluation of various basin and outlet device configurations.

Floodplain Studies

Existing floodplains that were approximated during the feasibility and site analysis stage of design should be analyzed and modeled using field-run topography to determine the extent of the floodplain. These floodplains should be brought in to the base map to more accurately identify their expanse. The specific details of floodplain analysis are presented in Chapter 18.

Grading and Earthwork

Site 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 whether any requirements for off-site construction are needed. 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.

Prior to initiating work on the grading plan, the design engineer should review soils mapping and/or available geotechnical reports to understand and evaluate existing site and soil conditions. These investigations should include a review of information regarding problem soils, ground water levels, rock formations, suitability of on-site soils for reuse, and other conditions that might affect the grading design.

Grading Strategy. Once site and street geometrics have been refined and gravity utilities tentatively arranged, it is time to develop a grading plan. The grading design can be broken down into various areas of the site including roadways, stormwater management areas, building pads, open-site areas, areas to remain undisturbed, buffer areas, and parking areas. Each area presents its own specific requirements that need to be evaluated against the existing site conditions. Grading experience makes most considerations second nature and enables the designer to concentrate on an overall strategy that produces the best possible plan.

Perhaps the most important aspect of grading is to ensure proper drainage of the site. If water is trapped, flows toward undesirable locations, or causes erosion, the grading has failed in its most basic requirement. The design engineer must take into account site-generated runoff as well as 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. 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 nonerosive 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.

Both experience and imagination play a role in developing the 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. 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 nonerosive. However, it is difficult to maintain sheet flow for large areas or over long distances, due to water's tendency to concentrate. Left on its own, water quickly gathers into swales and ditches. The designer's job is to artfully manipulate this transition from sheet flow to shallow concentrated flow using sound engineering principals that accomplish the goal without cluttering the site with drainage system components. Although it may be possible to direct all runoff to its outfall points via overland flow on a small site (i.e., a single residential lot or small commercial site), it is usually necessary to collect and pipe it underground on larger development sites. Detailed design methodologies for storm sewer conveyance systems are presented in Chapter 21 and should be referred to for additional clarity needed beyond the schematic design phase.

Guiding the designer are general rules of thumb that help simplify the process. A basic premise of grading is that a minimum slope of 0.5 percent needs be maintained to drain runoff across paved surfaces and 2 percent needs to be maintained to drain runoff across nonpaved areas. Decreasing the slope below these recommendations may result in sluggish drainage and standing water. Construction delays could also occur: 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 period of time, which may cause construction delays following rain. Yet site constraints often require slopes less than those recommended; thus, they are not entirely uncommon. The use of flat slopes should be carefully considered and simply avoided if possible.

In some instances, the minimum desirable slope exceeds 2 percent, 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.[3] Dropping the proposed elevation at least 6 inches below the parge grade within the first 10 feet of the building (5 percent) is ideal. Figure 19.24 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 percent 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: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, which makes a working knowledge of pertinent ordinances a valuable tool.

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 as well as pedestrian and bicycle facilities, usually closely tied to road grades, is explained in Chapter 20. Pedestrian facilities, especially those subject to accessible design criteria, require special attention. Accessible routes should be identified and preliminarily graded at this stage in the design process to ensure compliance with applicable federal regulations, and where conflicts become evident, alternative options should be investigated.

Grading and Aesthetics. Drainage considerations form the basis of functionally sound grading; however, function 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 enhances the consumer's experience and creates a higher demand for the property.

Figure 19.25 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 (19.25*a*). Grading can also be used to influence what we see, by hiding a visually undesirable element, as shown in 19.25*b*, or opening up a view, shown in 19.25*c*. 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 be used to direct wind away from buildings or outdoor use areas.

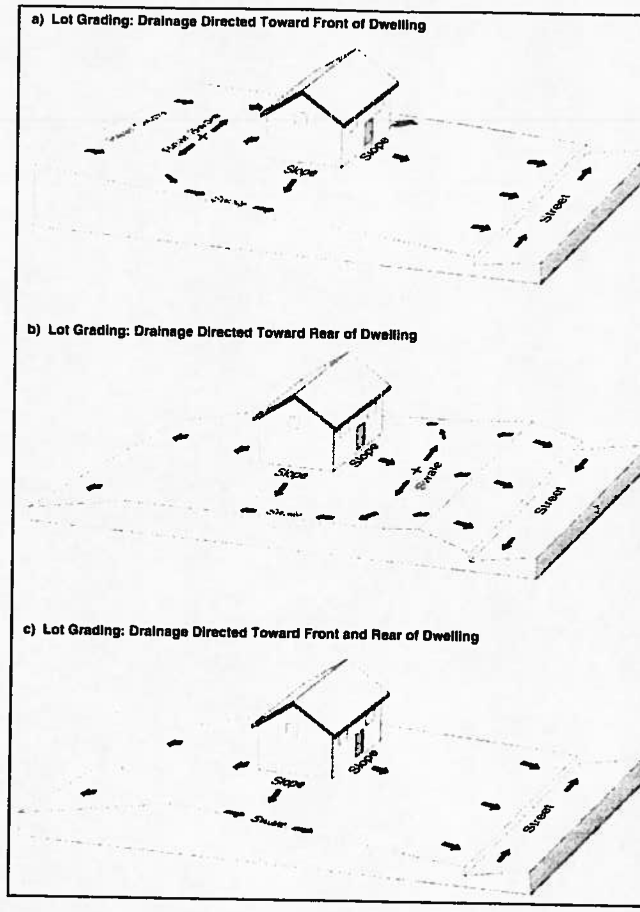


FIGURE 19.24 Three basic grading schemes for drainage around a building.

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-foot 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 or she must be prepared with lower-cost solutions if necessary.

Schematic Grading Analysis. Before detailed grading plans are under way, the designer should develop a schematic or preliminary grading scheme to determine any problem areas and to identify grading opportunities and constraints that will need to be addressed through the remainder of the

[3] A coat of masonry cement (parge) is applied to the part of the building walls below grade as a deterrent for moisture penetration. The parge grade is the elevation of the ground around the building sufficient to cover the parge coating.

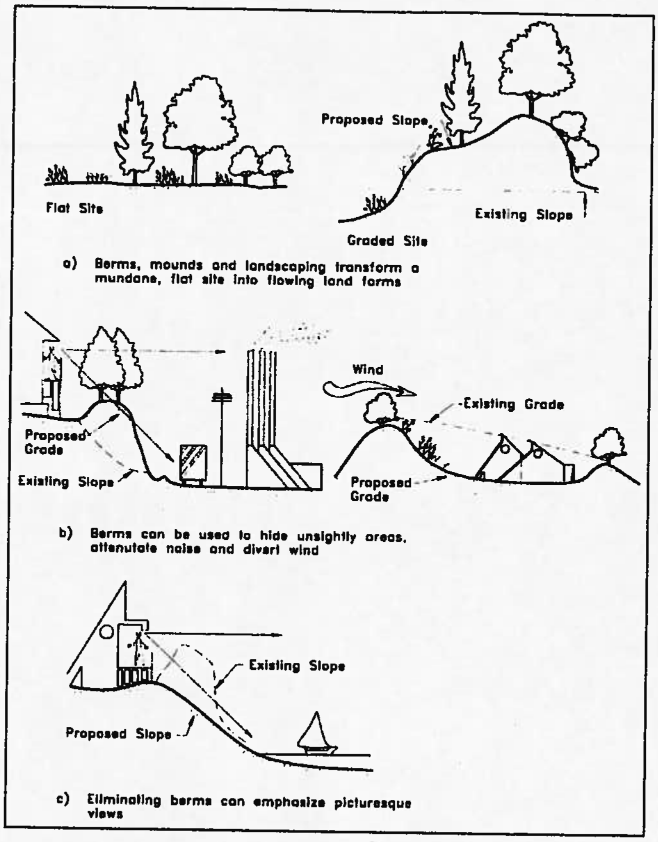


FIGURE 19.25 Grading to improve aesthetics. (Courtesy of Nelischer, Maurice, ed. *Handbook of Landscape Architectural Construction*, vol. 1, 2nd ed. Washington, DC: Landscape Architecture Foundation, 1985)

design progresses. Developing a schematic grading plan consists of utilizing the schematic layout to confirm the placement of building footprints and set the building elevation(s), road grades, and parking area grades. Spot elevations and sketching the 5-foot, or sometimes 2-foot, contour lines helps to determine the feasibility of the building elevations. Often the initial grading plan or general approach to the site grading scheme is flushed out by hand; it is then further refined, detailed, developed, and digitized using CADD software applications to establish geometrically correct alignments and digital terrain models (DTMs) for further analysis.

The following checks should be performed on the preliminary grading scheme in order to gauge functionality and aesthetic performance:

- Drainage patterns are analyzed.
- Rough estimates of cut and fill quantities are calculated (see Chapter 23 for detailed methodology).
- The need for steep slopes and retaining walls is determined.
- Tree-save and other sensitive (environmentally or historic) areas are identified.
- Limits of clearing and grading are defined.

During this process, several grading schemes may result, each with its own advantages. Conferring with the client may help in establishing the preferred grading scheme to pursue in final design.

Wastewater Collection and Treatment

Generally there are two common types of wastewater collection and treatment methods implemented on land development projects: individual subsurface disposal systems (septic systems) and public collection and treatment systems. A less common third type of system is a private community system that may utilize subsurface disposal or small, oftentimes packaged, treatment plants. Ordinarily the decision as to which type of system will be utilized to serve the development is made during the feasibility stage along with a determination as to whether sufficient capacity is available if utilizing public collection and treatment systems.

During the preliminary engineering phase, the design of the collection system begins with a projection of anticipated sewage flow. These calculations may be based on regulatory requirements for projected flows, historical data obtained from similar sizes and types of facilities, or other accepted reference sources (see Chapter 24 for additional information on sewage generation estimating). It is important for the design engineer to recognize that the system must be designed according to anticipated capacity requirements as well as regulatory requirements, either of which may end up governing the design. It is preferable for sanitary sewer collection systems to be gravity systems to the maximum extent possible, to eliminate the increased cost and maintenance of pumping facilities. It is also important to evaluate and assess the potential need for future expansion of the new system and make appropriate accommodations if this is anticipated to occur. If the collection system will be tied into an existing system, as-built information on the downstream system should be evaluated as part of the design. 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. Many jurisdictions require minimum clearances, cover, separation, and, occasionally, encasement of sanitary collection systems from other utilities and in areas such as stream crossings.

If a pump station is required to convey sanitary flow, a detailed analysis of the various options for placement of the station and the route of the gravity collection piping as well as the forcemain should be performed in the feasibility stage. In the preliminary engineering phase, the pumping station needs to be sized based on anticipated flows, allowable residence time, cycle length, and required wet well depth. Careful consideration must be given to construction feasibility and cost associated with each element of the pumping station.

In the feasibility phase, 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 need to be explored in the preliminary engineering phase. As an example, if septic systems will be used for sewage treatment, they should be preliminarily sized and configured. In order to do this, percolation tests in the area of the disposal fields must be performed to determine the rate at which the soil can absorb the effluent. The tests (including number and location) need to be coordinated with the geotechnical engineer and must be performed per the applicable regulatory requirements.

Water Supply and Distribution

Water supply is ordinarily facilitated by either connection of the development to a public water distribution system or installation of an on-site ground water supply well. In most cases, it is preferable to provide a connection to public water supply if it is available to the site or can be extended to serve the site at a reasonable cost. Again, the decision as to which type of system will be utilized to serve the development is made during the feasibility stage along with a determination as to whether sufficient capacity for fire and domestic flow is available from the existing water system.

The preliminary water supply system design begins with a projection of anticipated domestic and fire flow demands. These calculations may be based on regulatory requirements for projected flows, historical data obtained from similar sizes and types of facilities, or other accepted reference sources. The potential need for future expansion of the new system should be considered and appropriate accommodations made if this is anticipated to occur. The horizontal configuration of the water supply system should be determined. Water system pressures in the immediate vicinity of the proposed development should be checked as well as available fire flows.

If the site will be served by an onsite water supply well, borings and pumping tests must be performed to verify available capacity.

Erosion and Sediment Control

During any site development project, it is critical to develop a comprehensive soil erosion and sediment control plan to be implemented to prevent erosion and the off-site migration of sediments. Most states require the preparation of a Storm Water Pollution Prevention Plan (SWPPP) as part of every construction project that involves land-disturbing activities. 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 review and approve rough grading/erosion and sediment control plans so the developer/builder can begin site construction while preliminary and/or final engineering efforts are still under way.

Typical elements of most erosion and sediment control plans include stabilized construction entrances; temporary sediment barriers such as silt fence, stormwater inlet filters, temporary sediment ponds, runoff diversions and check dams; dust control measures; temporary stockpile and soil stabilization; and permanent measures such as permanent vegetative cover, conduit outlet protection, slope stabilization, or armoring. Many nonstructural design elements should also be incorporated into these plans, including minimizing site disturbance during construction, minimizing soil compaction in vegetated areas to reduce runoff, and protecting environmentally sensitive areas, such as wetlands, by establishing adequate buffers from construction activities. Careful consideration of erosion and sediment (E&S) controls at this phase of the design is critical to understanding the construction process for the proposed development program and developing a construction sequence or project phasing that respects the constraints of the site while still providing sufficient space and reasonable time frames for requisite construction activity. Planning for and performing the preliminary design of E&S controls will help to minimize erosion and construction runoff pollution as well as clarify critical areas requiring greater attention during final engineering.

Dry Utility Design

Most development projects require various dry utility services such as natural gas, electrical distribution, and telephone and data communications services. Availability of these services should be determined in the feasibility phase. It may be necessary to contact the utility providers during the preliminary engineering phase to determine the limits of design responsibility, upgrades that may be necessary, and load information that will be required to perform final design and engineering of the these systems. Additional information on designing these facilities is included in Chapter 26.

Cost Estimating

Preliminary cost estimates of the proposed site improvements, including the permitting or regulatory fees associated with the development program, can be determined at this stage of the project once the preliminary engineering design is complete. Such costs can be approximated using nationally published reference manuals or the local governing agency pricing guidelines. Oftentimes developers/builders have a database of cost guidelines they may wish to utilize or they use previously designed and built projects for unit price costs. These preliminary estimates are important for the developer to review in order to ensure the project costs are in line with expectations. At this stage of design, it is appropriate for the design engineer to include some level of design and construction contingencies in these estimates, to account for additional costs that may not be clarified until after the final design is complete. See Chapter 29 for additional guidance on estimate preparation.

Regulatory Permitting

Engineers, planners, and most members of the land design team appreciate the complexity involved in the design and construction of even a simple land development project. Countless hours of planning, evaluation, drafting, reviewing, calculation, and design effort go into each project. It is critically important that the land development engineer be cognizant of the myriad layers of governing regulations, agency review, scrutiny, and overall red tape that will be brought to bear on the project once it is designed. The regulatory approval process can often be the place where a client's great, exciting project ultimately meets an untimely death, if not planned for (in terms of schedule and cost) and diligently, tactfully pursued. Every design engineer should work with the client to develop a regulatory permitting strategy that takes into account all the various regulations and layers of jurisdictional oversight related to the project. This permit strategy should be incorporated into the project schedule to set reasonable time expectations and to identify potential roadblocks to commencing project construction. This may also facilitate the identification of ways to fast-track or expedite the project approval process, which is often a primary goal of many developers.

PRELIMINARY ENGINEERING STUDY DELIVERABLES

At the conclusion of the preliminary engineering effort, the finalized work product typically consists of a set of preliminary drawings, a design checklist, outline specifications and product data, preliminary reports, preliminary construction cost estimates, and value engineering recommendations.

Preliminary Engineering Checklist

Checklists and reports are often prepared during the site analysis, site selection, and feasibility 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 in order to document the design progression—in particular, any noteworthy changes to the development program. An example of a preliminary engineering checklist is provided in Figure 19.26.

Waiver Preparation. 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 might be required. Obtaining waivers or modifications can be critical in meeting the objectives of the proposed development 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.

Green Building Design and Sustainable Site Evaluation. Green building and sustainable site design concepts implemented as part of the project should be reviewed, evaluated, and refined at this stage of the design process with the entire design team and client. From a land development consulting perspective, this step includes a review of the individual green building metrics—in particular, those related to sustainable site development—and an evaluation of whether the site components and design are evolving in a manner conducive to meeting the established project goals. With reference to USGBC's LEED-NC Sustainable Sites category referenced throughout this text, the preliminary engineering evaluation should look at all of the site credits and reevaluate the design elements based on the following considerations:

- Prerequisite: Development of a Construction Activity Pollution Prevention Plan or E&S Plan commences during the schematic design phase.
- Credits 1–3: review and confirm compliance as required.
  - Credit 1, Site Location and Credit 3 Brownfield Redevelopment are a function of the initial site selection and should be confirmed and documented if applicable. If a brownfield site, any remediation and cleanup efforts should be incorporated into the project and permit schedule at this time (see Chapter 17 for additional details).
  - Credit 2, Development Density and Community Connectivity should be reviewed in terms of both compliance paths—site and community FAR of 1.377 or proximity (½ mile) to both a residential community of density 10 dwelling units per acre (du/ac) and 10 basic services that are pedestrian accessible. Regardless of the chosen compliance path, the specific metrics should be verified or checked; specifically, if the second compliance path is chosen, pedestrian facilities between the site and the basic services should be reviewed and preliminary designs developed where needed.
- Credits 4.1–4.4: The Alternative Transportation credits require a complete evaluation of the site parking plan in order to determine a comprehensive approach to meeting these credits in keeping with the project green building goals as well as the owner requirements for parking accommodations. A parking master plan is recommended in order to organize and account for the various combinations of preferred parking, required handicap parking, and other facilities such as bike racks or fueling stations. This master plan should be developed concurrently with any Transportation Demand Management (TDM) strategy that might be warranted as a result of proffers or as a component of the development program in general.
  - Credit 4.1, Public Transportation Access: This credit is predominantly a function of the site selection/location—the site is either within a ½ mile of mass transit or ¼ mile of two bus lines or it isn't; however, if this credit is pursued, the requisite maps and documentation should be prepared at this stage to ensure a thorough understanding of the transit options/routes/stops and that pedestrian routes exist between the site and the qualifying facilities. If

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 19.26 Example of a preliminary engineering checklist.

D. Stormwater Management

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? ____ ____ ____

E. Floodplain

1.0 Is there a floodplain present? ____ ____ ____

2.0 If so, has it been modeled using field-run topography? ____ ____ ____

F. Grading / Earthwork

1.0 Has a grading study been performed? ____ ____ ____

2.0 Has an earthwork analysis been performed? ____ ____ ____

3.0 Have the soils been evaluated? ____ ____ ____

G. Wastewater Collection / Wastewater Treatment

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? ____ ____ ____

H. Water Distribution / Water Treatment

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? ____ ____ ____

I. Erosion and Sediment Control

1.0 Has a general erosion and sediment control program been determined for this project? ____ ____ ____

J. Cost Estimates

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 19.26 *(Continued)*

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 19.26 *(Continued)*

pedestrian improvements are required, they should be incorporated into the preliminary engineering design.

□ Credit 4.2, Bicycle Storage and Changing Rooms: These should be provided in close proximity to a building entrance. Computations should be performed to determine the number of bike stalls/racks required, and they should be located on-site in the preliminary plan. Coordination with the architect and/or MEP is important in commercial applications in order to ensure the shower provision is also met and bike and shower facilities are located accordingly.

□ Credit 4.3, Low Emitting and Alternative Fueled Vehicles: There are three compliance paths for this credit; options 1 and 2 relate to the provision of and preferred parking for Low-E and alternative vehicles, while option 3 requires alternative fuel refueling stations to be provided. In terms of the preliminary engineering plan, preferred parking should be accommodated and denoted in the overall parking scheme; a detail for signage needs to be developed as part of the final engineering effort. If refueling stations are the preferred compliance path, appropriate siting/location of the facilities should take place, and the engineer should begin to understand any utility, screening, or special design requirements related to this facility.

□ Credit 4.4, Parking Capacity: This is a critical credit to assess during the preliminary engineering phase, as parking counts are often set or committed to during the rezoning/entitlement process. This credit emphasizes meeting but not exceeding local zoning ordinance parking requirements, providing preferred parking for ride-sharing scenarios, and developing transportation demand management (TDM) programs in order to minimize on-site parking requirements and encourage alternative forms of transportation. Preferred parking spaces for car pools or van pools should be designated on the parking master plan and a signage detail developed.

■ Credits 5.1–5.2, Site Development: This includes evaluation of design measures to protect and restore natural habitat and to maximize open space on the site. As discussed in Chapter 15, consideration of this credit is important during the schematic design phase as limits of clearing grade begin to take shape. These limits should be carefully considered, refined, and modified during the course of preliminary engineering to accommodate infrastructure needs (including construction staging and installation requirements) while balancing the target open-space goals for the site.

■ Credits 6.1–6.2, Stormwater Design: This includes reviewing provisions to reduce quantity of site runoff and inclusion of design strategies to provide water quality treatment of stormwater runoff. As indicated in the preceding discussion of preliminary engineering of stormwater management facilities, the first step is determining the applicable requirements. It should be noted that often the local jurisdictional SWM requirements differ from the specific metrics of Credits 6.1 and 6.2, in which case the engineer must determine the goals of the SWM program in consultation with the client and the rest of the design team. Once a clear SWM goal is determined, facilities can be considered and preliminarily designed to meet the requisite criteria (see Chapter 22).

■ Credits 7.1–7.2, Heat Island Effects: This includes review of design provisions to reduce rooftop and non-rooftop heat island effects, most often resulting from the creation of new paved surfaces. Implementation of design features such as highly reflective (white) roofing or green roofs and incorporation of shade features, such as shade trees in parking areas, highly reflective or open grid paving, and/or covered parking, are tactics for reducing the heat island effect. During preliminary engineering it is appropriate to start looking at the material selection for site-impervious features such as sidewalks, trails, and plazas as well as developing a landscape or streetscape concept to address provision of shade. Although these details are more typical of final engineering design, coordination of this effort with the design team is important, as many credits are purposely linked to produce synergy in design—for instance, a green roof most definitely would help achieve Credit 7.1 but might also help to achieve the stormwater credits (6.1 and 6.2) and open-space credits (5.1 and 5.2), depending on the site location and the development program as a whole, not to mention the internal building benefits that would be examined under the Energy and Atmosphere category. It is important for the design team to work collaboratively at this point to optimize site and building features from both a functional and a cost perspective. Understanding the rating system, the project goals, and the owner's requirements are key to developing a sustainable site plan.

- Credit 8, Light Pollution Reduction: This includes review of proposed site lighting to ensure that lighting is maintained at a minimal level to the extent possible and light sources are shielded to minimize night glow from the site. As indicated in Chapter 26, a lighting consultant is likely required in order to provide the most efficient lighting system in terms of meeting safety and comfort standards while minimizing light pollution. That consultant should be brought onto the design team, if not already included, and should begin to lay out a lighting scheme and research fixtures in concert with refinement of the preliminary plan.

- Water Efficiency Credit 1 (Water Efficient Landscaping) and Credit 2 (Innovative Wastewater Treatment Technologies) should also be considered and reviewed during the schematic design. If required, an irrigation consultant should be brought onto the team. The irrigation system design should be developed concurrently with the landscape and stormwater management plans, especially if water reuse is a desirable strategy. If on-site wastewater treatment is a site requirement as determined during the feasibility study and further refined during the course of preliminary engineering, treatment strategies should be examined and refined with respect to this credit if possible.

If the targeted green building goals—specifically, the site components—are not being attained, an assessment of the specific site metrics as outlined here should be performed to determine other measures or design principles that should be incorporated into the design during the course of final engineering. Changes to the design program are bound to occur; savvy land development consultants should be aware of and attentive to the various green building strategies employed on the project as a whole so that they can adapt, refine, and innovate throughout the design process to optimize the site's potential, developing strategies that are buildable and sustainable.

VALUE ENGINEERING

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 value engineering is to identify cost-saving opportunities to the developer/builder. This effort may include members of the project team, such as the architect and planner, public agency reviewers, the project attorney, the construction manager, and the developer/builder.

During a typical value engineering review, it might become apparent that the site elevation should be raised or lowered to balance the earth materials, based on the preliminary grading and earthwork calculations. Reconfiguring lots may reduce street lengths and utility requirements. Perhaps regrading or adding retaining walls might preserve more open space and trees.

Performing a value engineering study during the preliminary engineering phase allows design and cost issues to be reviewed and potential options to be identified prior to final design. This provides an opportunity to make design adjustments with less impact to the schedule and the effort required to complete the design.

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