NASHVILLE STORMWATER MANAGEMENT MANUAL

VOLUME 3—THEORY

CHAPTER 2
Hydrology

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Chapter 2
HYDROLOGY

SYNOPSIS

Surface hydrology is a key component of stormwater management evaluations. Ideally, observed streamflow data should be used to evaluate the surface hydrology of a watershed. However, since streamflow data are usually not available, synthetic procedures based on theoretical and empirical relationships are often used in practice. Although this chapter provides information on the fundamentals of streamflow data analysis, the emphasis is on synthetic procedures for converting precipitation to runoff.

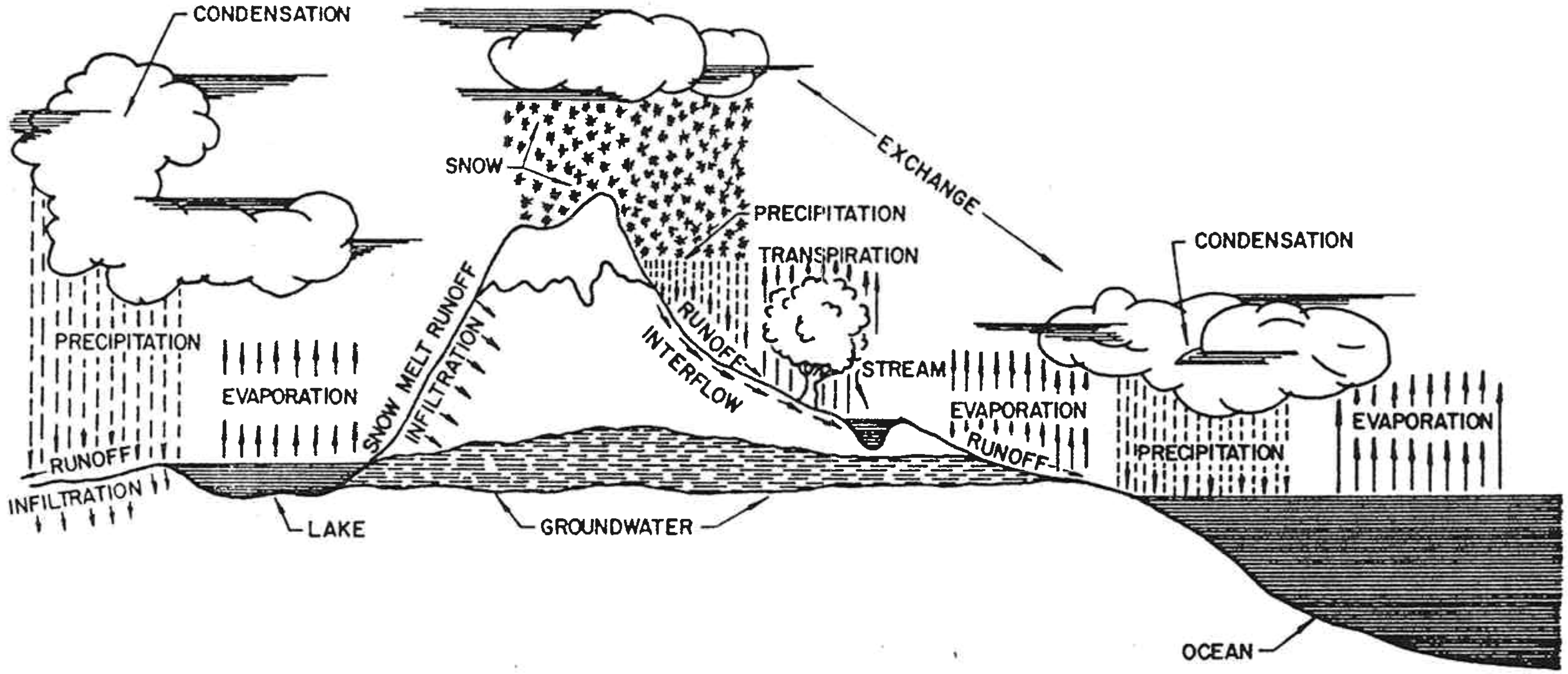
A general description of the hydrologic cycle and precipitation forms is followed by a discussion of relationships for evaluating rainfall excess. Fundamentals of synthetic procedures for the development of flood hydrographs and peak flow rates are followed by a discussion of hydrologic routing .

2.1 HYDROLOGIC CYCLE

The movement of water between the atmosphere and the earth's surface, and its transformation during this movement from one physical state to another, is called the hydrologic cycle. A simplified illustration of this process is provided in Figure 2-1.

During the hydrologic cycle, moisture in the warm air condenses and becomes precipitation in the form of rain, hail, sleet, or snow. The disposition of that precipitation as it reaches the earth's surface and eventually returns to the atmosphere completes the cycle.

The various processes during the hydrologic cycle that remove precipitation before it can become surface runoff are called abstractions. The relative magnitude of each of the following abstractions should be considered during stormwater management calculations.



Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-1
The Hydrologic Cycle

2.1.1 EVAPORATION

When air is in an unsaturated condition (i.e., has not reached its maximum capacity of moisture for the given temperature and pressure), water is returned to it through evaporation. This can occur both before and after precipitation reaches the ground and is a continuous process from standing waterbodies. Evaporation can usually be ignored, although it plays a major role in the long-term water balance of a watershed and may be important if long periods are required to percolate stormwater.

2.1.2 INFILTRATION

The flow of water into the ground is termed infiltration, and its disposition depends on the soil's field capacity, which is the moisture held by the soil after excess gravitational drainage occurs. If the moisture content is greater than field capacity, water percolates into the groundwater, from which it may move as seepage into streams and lakes or, in capillary movement, back to the root zone.

The process of infiltration is influenced by many factors, including soil type, vegetal cover, time elapsed between precipitation events, precipitation intensity, and temperature. Although infiltration is often an important element in determining the response of a watershed to a given rainfall event, there is as yet no universally acceptable model for predicting infiltration rates. Empirical methods discussed later in this chapter are often used in practice.

2.1.3 TRANSPIRATION

Transpiration is the process by which vegetation takes water from the soil and transmits it as vapor to the air through foliage. When the moisture content of the soil is less than the field capacity, transpiration can be a significant pathway for returning water to the atmosphere. As with evaporation, transpiration is only significant over time and can usually be ignored for stormwater management calculations.

2.1.4 INTERCEPTION

Interception is the process in which water falling as precipitation is trapped by objects above ground, such as buildings, trees, and vegetation, from which it subsequently evaporates. The quantity of water removed in this manner for a single storm is generally small.

2.1.5 DEPRESSION STORAGE

When water accumulates in puddles or depressions, with no possibility of release through overland flow or runoff, it is termed depression storage. The amount of water detained in depression storage varies greatly with land use, e.g., a paved surface has less storage than a furrowed field. The relative importance of depression storage to runoff rates for a given storm depends on the amount and intensity of precipitation.

2.1.6 DETENTION STORAGE

The storage incurred during a storm event by the volume of water forming the flow path that allows overland flow to enter drainage systems is called detention storage. The storage is temporary, since the water will continue to run off after precipitation ceases. The amount of water thus stored depends on land use, vegetal cover, slope, and rainfall intensity.

2.1.7 RUNOFF COMPONENTS

The following four main components of runoff are generally considered:

1. Overland flow
2. Channel precipitation
3. Interflow
4. Groundwater flow

Overland flow travels over the ground surface to the stream channel. Channel precipitation is direct rainfall on the water surface. Runoff that enters a stream channel by traveling laterally through the upper layer of soil is called interflow, while streamflow generated by the occurrence of water table conditions above the channel bottom is called groundwater or base flow.

The distinctions drawn between these four components are arbitrary. For convenience, a common practice is to divide the total surface runoff volume into two parts, the storm (or direct) runoff and base flow. Direct runoff includes overland flow, subsurface flow, and channel precipitation, while base flow consists of groundwater flow. Since base flow may not always be significant, a direct runoff hydrograph may be adequate for hydraulic calculations.

If base flow is a significant factor for a given watershed, an appropriate base flow component should be considered. Fundamentals of base flow separation are presented in books by Chow (1964), Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), and USDOT, FHWA (HEC-19, 1984).

2.2 PRECIPITATION

Precipitation occurs in a liquid state as rain, in a mixture of liquid and frozen water as sleet (melted snow), and frozen in crystalline (snow) or massive (hail) form. It results when air cools, causing moisture to condense. The most common cause of cooling is dynamic, or adiabatic, lifting of the air, in which a given parcel of air rises and cools, and moisture condenses into very small cloud droplets. If these droplets coalesce and become large enough to overcome air resistance, precipitation occurs.

2.2.1 TYPES

Precipitation can be classified by the origin of the lifting motion that induces it. Each type is characterized by different spatial and temporal rainfall regimes. The three major types of storms are:

1. Convective
2. Orographic
3. Cyclonic

A fourth classification often added is the hurricane or tropical cyclone, although it is basically a subtype of the cyclonic storm.

Convective Storms

Precipitation occurs during convective storms when warm, moist air rises from lower elevations into cooler overlying air, as shown in Figure 2-2. The characteristic form of convective precipitation is the summer thunderstorm. The surface of the earth, warmed considerably by afternoon sun, imparts heat to the adjacent air. The warmed air rises through cooler, overlying air, and if sufficient moisture content conditions are met, rapid condensation occurs. A single convective storm can produce extremely high rainfall rates in a short period of time, which is typically represented by the 8- to 60-minute portion of the appropriate rainfall intensity-duration-frequency (IDF) curves.

Orographic Storms

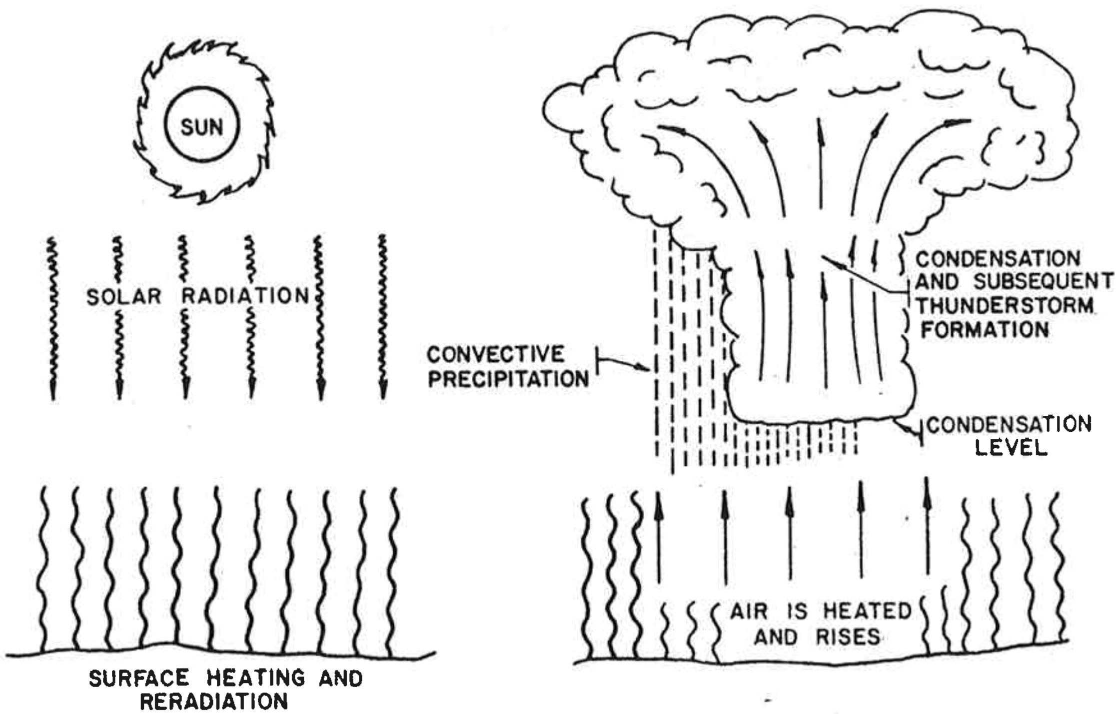
Orographic precipitation occurs when air condenses as it is forced to rise over a fixed-position geographic feature, such as is shown in Figure 2-3. Because of the relatively mild relief around Nashville and Davidson County, this type of storm is uncommon. When it does occur in more mountainous areas, however, it can generate high intensity rainfall.

Cyclonic Storms

Cyclonic precipitation results when air rises as it moves from an area of high pressure toward an area of low pressure. In the middle latitudes, cyclonic storms generally move from west to east, and are sometimes called extra-tropical cyclones or continental storms. Continental storms occur at the boundary between air masses of significantly different temperatures. A disturbance in the boundary can grow, appearing as a wave as it travels from west to east along the boundary.

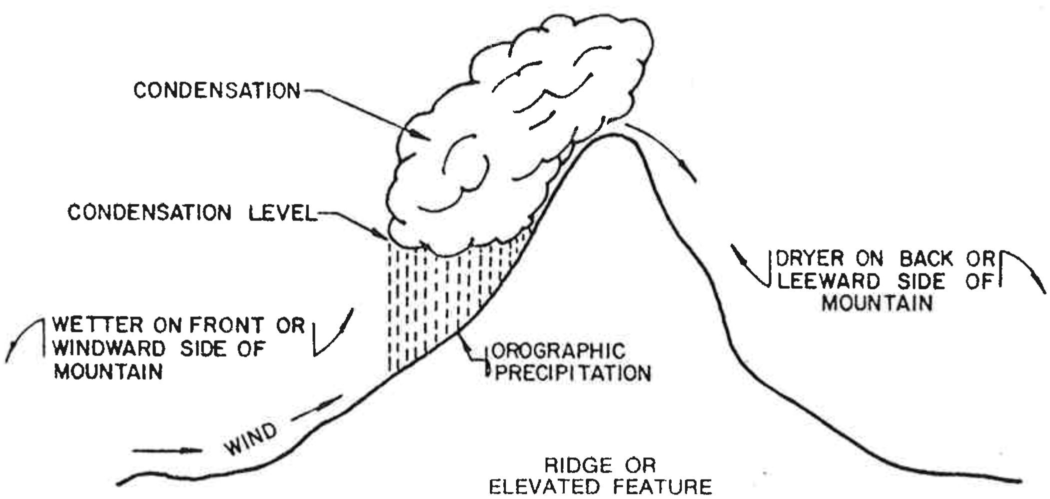
On a weather map, the cyclonic storm will generally appear as shown in Figure 2-4, with two boundaries or fronts. Warm air pushing into an area of cool air is called a warm front; when cold air is the aggressor, a cold front occurs (see Figure 2-5). The precipitation associated with a cold front is usually heavy and covers a relatively small area; warm front precipitation is more passive and lighter, but covers a much larger area. Tornadoes and other violent weather phenomena are associated with cold fronts.

Hurricanes. Hurricanes or tropical cyclones develop over tropical oceans where surface water temperatures are greater than 85°F. These storms can produce tremendous amounts of



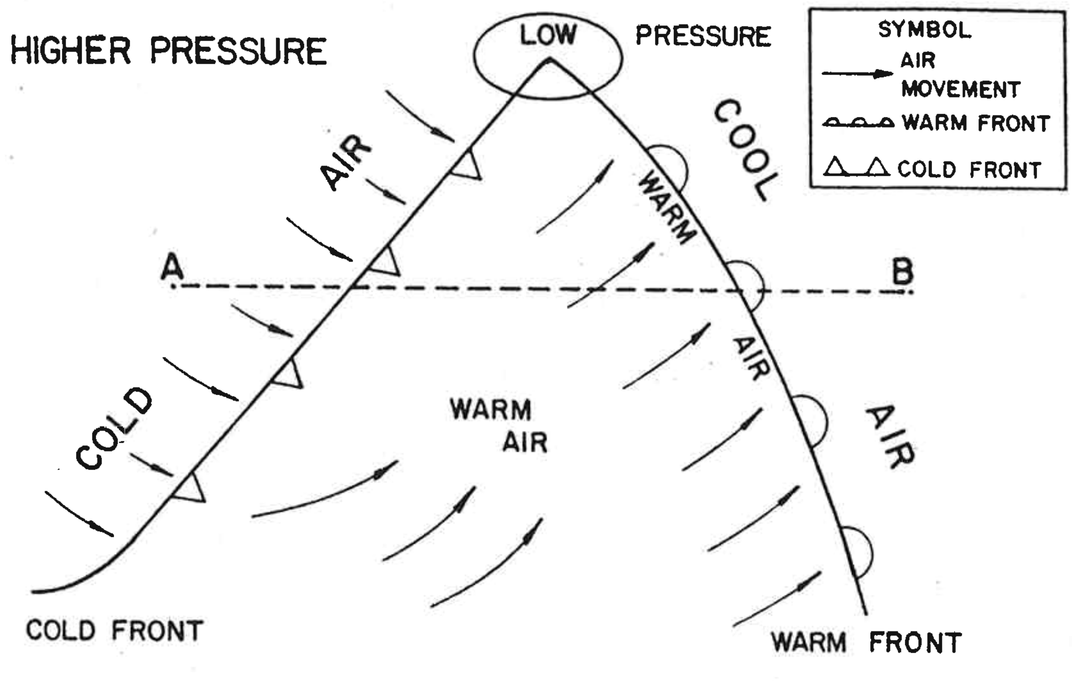
Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-2
Convective Storm



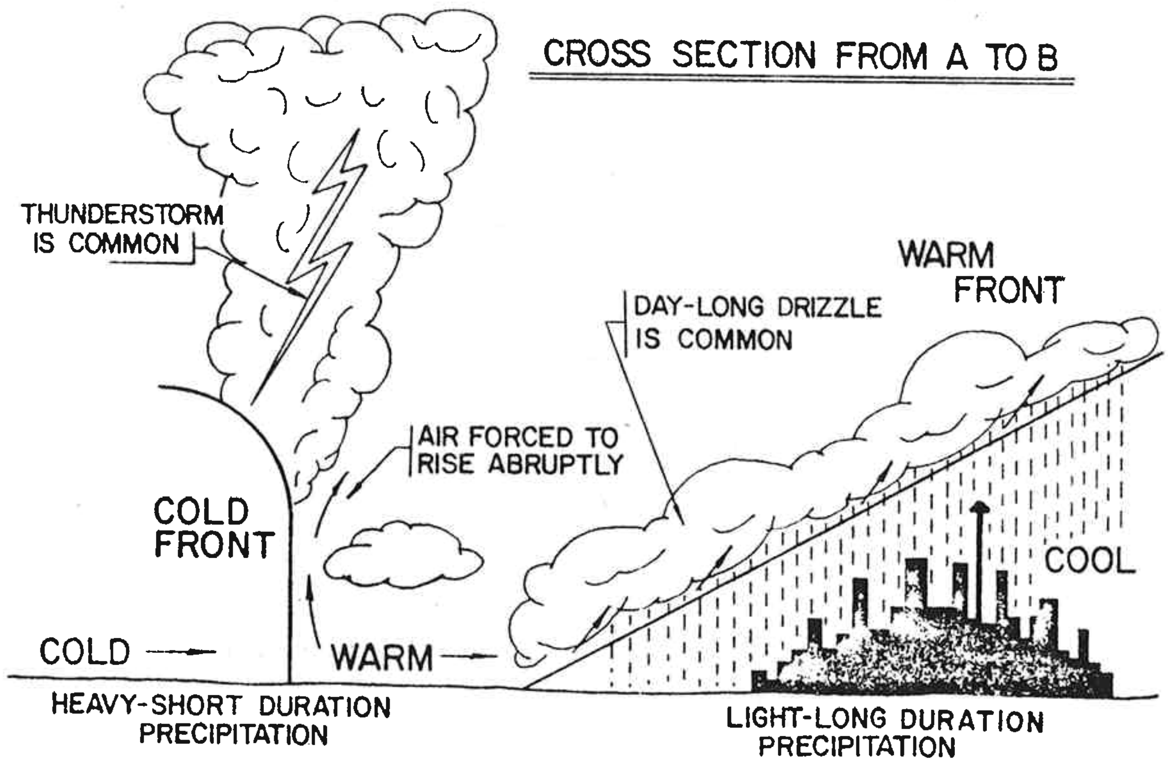
Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-3
Orographic Storm



Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-4
Cyclonic Storm



Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-5
Frontal Action of a Cyclonic Storm

precipitation in a relatively short time. Rainfall amounts of 15 to 20 inches in less than 24 hours are common in well developed tropical storms.

Hurricanes are very unlikely in the Metropolitan Nashville area, because hurricane forces decrease rapidly as they move across land. However, significant rainfall amounts can be expected from hurricane remnants as they pass nearby.

Northeaster. Extratropical storms that occur along the northern part of the east coast of the United States, accompanied by strong winds blowing from the northeast, are called northeasters. A typical northeaster consists of a single center of low pressure about which the winds revolve. Wind patterns are less symmetrical than those associated with hurricanes.

2.2.2 MEASUREMENT

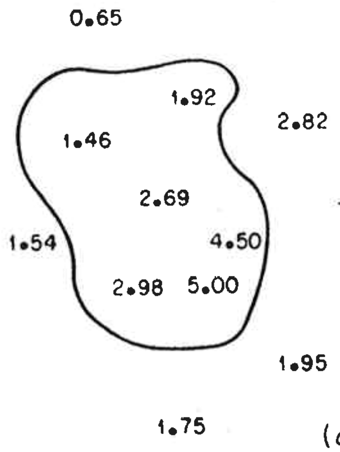
Precipitation can be measured using a variety of instruments and techniques, all of which are based on the vertical depth of water that would accumulate on a level surface if the precipitation remained where it fell.

Point Precipitation

Point precipitation is measured in either a recording or nonrecording rain gage. The standard non-recording gage used by the U.S. National Weather Service is an 8-inch diameter collector with a measuring tube that has a cross-sectional area one-tenth that of the collector. Recording gages in common use are the tipping bucket, the weighing gage, and the float gage. Recording gages provide information on timing, intensity, and the depth of rainfall.

Areal Precipitation

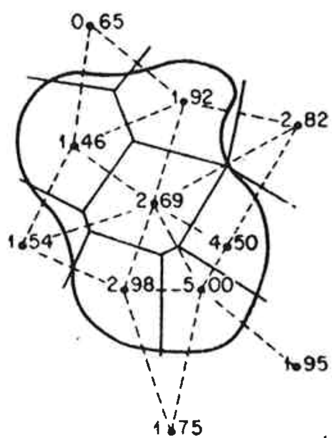
The spatial variability of precipitation can be evaluated by averaging the point measurements from several locations. Figure 2-6 provides a comparison of three averaging techniques, which are briefly described below. Point measurements can be adjusted directly to areal values by applying the areal reduction factors from Hershfield (1961) presented in Volume 2.



Arithmetic mean:

$$\frac{1.46 + 1.92 + 2.69 + 4.50 + 2.98 + 5.00}{6} = 3.09 \text{ in.}$$

(a)



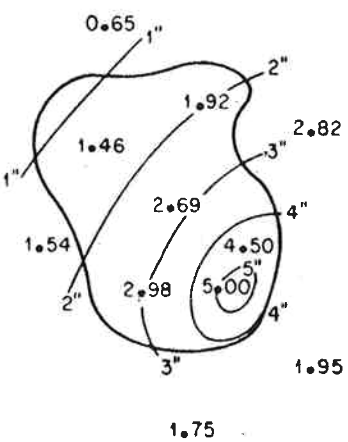
Thiessen method:

Observed precip. (in.)	Area* (sq mi)	Per cent total area	Weighted precipitation (in.) (col. 1 × col. 3)
0.65	7	1	0.01
1.46	120	19	0.28
1.92	109	18	0.35
2.69	120	19	0.51
1.54	20	3	0.05
2.98	92	15	0.45
5.00	82	13	0.65
4.50	76	12	0.54
	626	100	2.84

Average = 2.84 in.

*Area of corresponding polygon within basin boundary

(b)



Isohyetal method:

Isohyet (in.)	Area* enclosed (sq mi)	Net area (sq mi)	Avg. precip. (in.)	Precipitation volume (col. 3 × col. 4)
5	13	13	5.3	69
4	90	77	4.6	354
3	206	116	3.5	406
2	402	196	2.5	490
1	595	193	1.5	290
<1	626	31	0.8	25
				1634

Average = 1634 ÷ 626 = 2.61 in.

*Within basin boundary

(c)

Reference: Linsley, Kohler, and Paulhus (1982).

FIGURE 2-6

Areal Averaging of Precipitation by (a) Arithmetic Method, (b) Thiessen Method, and (c) Isohyetal Method

Arithmetic Average. This simple method adds the observed depths for each point precipitation measurement within a watershed and divides by the number of gages. The results are satisfactory if the gages are uniformly distributed and individual measurements do not vary widely from the average. Part A of Figure 2-6 illustrates this technique.

Thiessen Polygon. A weighting factor is developed for each point precipitation measurement. A perpendicular bisector is drawn for each line that connects gage locations. The effective area for each gage is then determined by the polygon bracketing each gage location. Multiplying the precipitation depth measured at each gage by the respective effective areas, expressed as a percentage of the total area, provides the weighted average precipitation for each gage. These weighted depths are then summed to give the precipitation depth for the total area. This technique is illustrated in Part B of Figure 2-6.

Isohyetal Method. Contours of equal precipitation are drawn to reflect the spatial variability observed over the area. The area between contours is multiplied by the average precipitation over that contour interval to obtain a weighted precipitation value. The values are then summed and divided by the total area to give the precipitation depth for the total area. Part C of Figure 2-6 illustrates this method.

2.2.3 CHARACTERISTICS

The following characteristics of precipitation are typically considered in drainage calculations:

1. Intensity (rate of precipitation)
2. Duration
3. Time distribution of rainfall (hyetograph)
4. Storm shape, size, and movement
5. Frequency

Brief descriptions of these characteristics are given below.

Intensity

Intensity is the rate of precipitation, commonly given in units of inches per hour. In any given storm, the instantaneous intensity is the slope of the mass rainfall curve at a particular time. For stormwater management, it is desirable to divide the storm into convenient time increments and to determine the average intensity over each of the selected periods. The results are plotted as rainfall hyetographs.

For drainage calculations, intensity is perhaps the most important of the rainfall characteristics, since discharge from a given watershed will increase as the intensity rises. Intensity varies from misting conditions, in which only a trace (<0.005 inch) of precipitation may fall, to cloudbursts, during which several inches per hour are common.

Duration

The duration of observed precipitation events is related to selection of a minimum interevent period during which no rainfall occurs. Statistical procedures discussed in textbooks by Benjamin and Cornell (1970) or Haan (1977) can provide a basis for selecting an appropriate minimum interevent time when observed data are evaluated.

Time Distribution

A hyetograph provides a distribution of incremental precipitation versus time (see Figure 2-7). In practice, depth-duration data for a particular design frequency are generally used to develop a synthetic design storm event. Theoretically, it is more accurate to use actual precipitation data and continuous real time simulation rather than a single design storm event. Current computer technology makes this approach possible (McPherson, 1978, or Marsalek, 1978), but it is often infeasible because of cost and data limitations.

Because depth-duration data do not represent a logical, chronological sequence of precipitation during a storm, the precipitation depth increments must be rearranged into a sequence that might actually occur. A balanced storm procedure can accomplish this rearrangement or, if a 24-hour duration storm is desired, a Soil Conservation Service (SCS) standard distribution could be used. The 24-hour rainfall hyetograph presented in Volume 2 should be used for Nashville and Davidson County.

A synthetic design storm developed using either the balanced storm or an SCS distribution will generally be more extreme

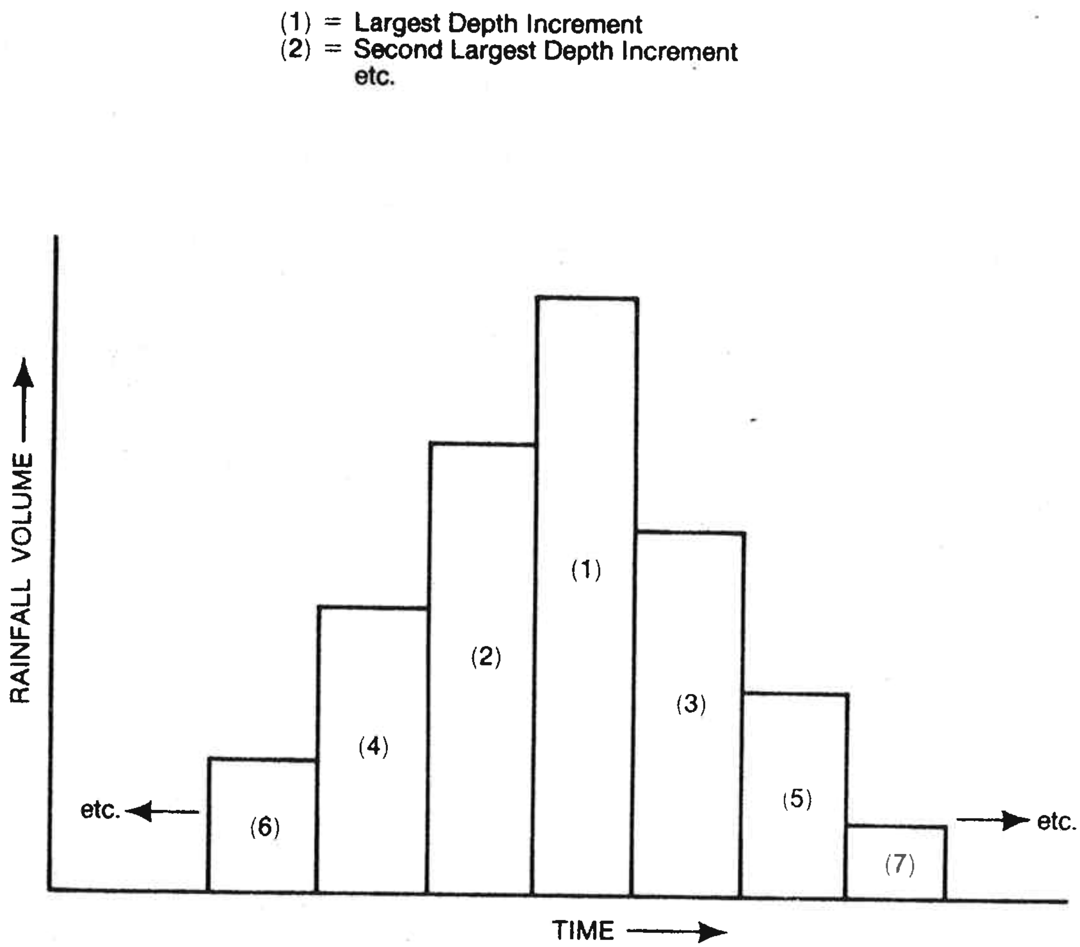


FIGURE 2-7
Balanced Storm Approach for Developing a Design Storm Hyetograph

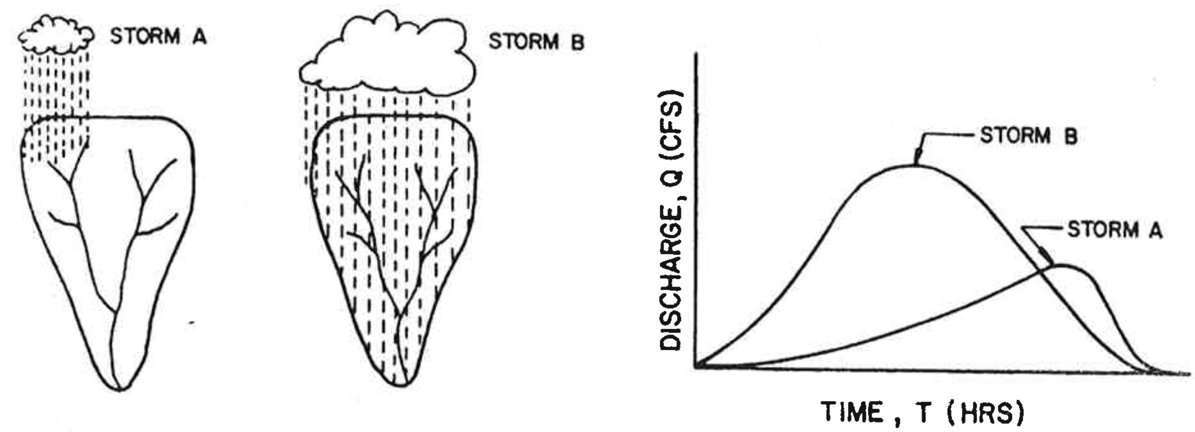
than most actual storms of the same total depth and duration, because the short-duration events are assumed to be nested within the long-duration storm. This is accepted practice, because it ensures that critical events for the small headwater areas of a watershed will not be overlooked. In addition, if the design storm duration is long enough, the critical duration for sizing a storage basin is ensured.

Balanced Storm. The balanced storm approach provides a precipitation hyetograph that has the same return period for each time interval within the total storm duration. This approach places the largest increment of rainfall for the selected time step at the midpoint of the total storm duration. For example, the midpoint for a 24-hour storm is 12 hours. Smaller depth increments are arranged symmetrically about the largest value, with the second largest value placed before and the third largest value placed after the largest. This "before and after" process continues until the entire hyetograph is developed. Figure 2-7 depicts the balanced storm approach for developing a design storm hyetograph.

SCS Distributions. Hyetographs developed by the SCS using published data were demonstrated through experience to have similar characteristics. For areas subject to both short-duration summer thunderstorms and long-duration frontal storms, the rainfall distribution versus time is termed a Type II distribution by the SCS. For areas in which intense short-duration storms are not prevalent, the rainfall distribution versus time is termed a Type I distribution.

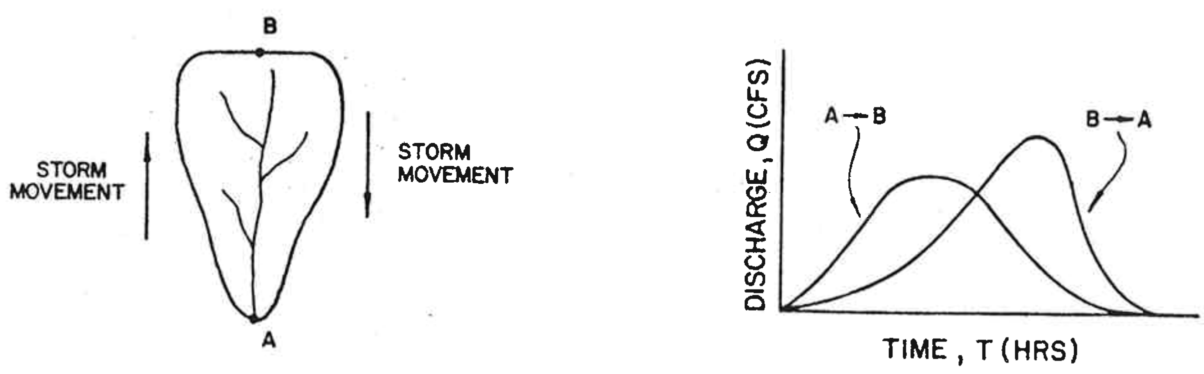
Shape, Size, and Movement

Storm shape, size, and movement are normally determined by the type of storm. For example, storms associated with cold fronts tend to be more localized, faster moving, and of shorter duration than storms produced by warm fronts. All three factors determine the areal extent of precipitation and the size of the drainage area that contributes over time to the surface runoff. As illustrated in Figure 2-8, a small localized storm of a given intensity and duration over just a part of the drainage area will result in much less flow than if the same storm covered the entire watershed.



Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-8
Effect of Storm Size on Surface Runoff



Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-9
Effect of Storm Movement on Surface Runoff

The location of a localized storm in the drainage basin also affects the time distribution of the surface runoff. A storm near the outlet of the watershed will cause peak flow to occur very quickly and the flood to pass rapidly. If the same storm occurred in a remote part of the basin, the runoff at the outlet would last longer and the peak flow would be lowered by storage in the channel.

Storm movement has a similar effect on the runoff distribution, particularly if the basin is long and narrow. Figure 2-9 shows that a storm moving up a basin from its outlet gives a distribution of runoff that is relatively symmetrical with respect to the peak flow. The same storm moving down the basin will usually result in a higher peak flow and an unsymmetrical distribution, with the peak flow occurring later. This phenomenon is more critical for large watersheds than for small ones.

Frequency

Frequency quantifies the likelihood of the recurrence of precipitation with a given duration and average intensity. The frequency of occurrence for the surface runoff resulting from precipitation is of primary concern and, in particular, the frequency of the peak discharge. Although the assumption that a given frequency storm always produces a flood of the same frequency should be used with caution, there are a number of analytical techniques based on it, particularly for ungaged watersheds.

2.3 RAINFALL EXCESS

After precipitation is reduced by the various abstraction processes, the amount which is left is often called rainfall excess. In practice, rainfall excess is usually estimated using empirical methods such as a runoff coefficient or SCS curve number. Theoretical fundamentals for evaluating rainfall excess, which are not presented in this manual, may be obtained from USDOT, FHWA (HEC-19, 1984), Chow (1964), Linsley, Kohler, and Paulhus (1982), Eagleson (1970), and Viessman et al. (1977).

2.3.1 RUNOFF COEFFICIENT

The calculation of rainfall excess as a fixed percentage of precipitation is accomplished using a parameter called the runoff coefficient. Because the runoff coefficient must account for interception, surface storage, and infiltration, the suitability of this empirical approach decreases as the complexity of the watershed increases.

Selection of a runoff coefficient should include a consideration of soil type, land use, antecedent moisture conditions, precipitation duration, and intensity. The tables presented in Volume 2 facilitate the selection process, but their use should be based on sound engineering judgment. A comparison of the actual performance of drainage facilities to design calculations can provide a basis for making judgment decisions. In general, the best use of a runoff coefficient is for homogeneous watersheds with high percentages of impervious area and under circumstances when the areal distribution of rainfall can be assumed to be relatively uniform.

2.3.2 SCS RELATIONSHIP

The SCS has developed an empirical relationship for estimating rainfall excess that accounts for infiltration losses and initial abstractions by using a site-specific runoff parameter called the curve number (CN). The watershed CN is a dimensionless coefficient that reflects watershed cover conditions, hydrologic soil group, land uses, and antecedent moisture conditions. Procedures for determining curve numbers are discussed in Volume 2.

The maximum soil storage and a CN value for a watershed can be related by the following expression:

$$S = \frac{1,000}{CN} - 10 \qquad (2-1)$$

where:

S = Maximum soil storage, in inches

CN = Watershed curve number, dimensionless

When the maximum soil storage is known, the rainfall excess can be calculated using the following SCS relationship:

$$R = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (2\text{-}2)$$

where:

R = Accumulated rainfall excess (or runoff), in inches

P = Accumulated rainfall, in inches

S = Maximum soil storage, in inches

Three levels of antecedent moisture conditions are considered by the SCS relationship. Antecedent Moisture Condition I (AMC-I) is the lower limit of antecedent rainfall or the upper limit of the maximum soil storage, S. Antecedent Moisture Condition II (AMC-II) represents average antecedent rainfall conditions, and Antecedent Moisture Condition III (AMC-III) is the upper limit of antecedent rainfall or the lower limit of S. For design purposes, AMC-II is generally selected.

Additional information on the SCS relationship can be found in USDA, SCS publications TP-149(1973) and NEH-4(1972).

2.4 STATISTICAL ANALYSIS

Statistical analysis is a systematic way of quantifying data for purposes of generalization. Methods of statistical analysis can also be used to predict future events, based on the characteristics of past data.

Statistical analysis is based on the concepts of populations and samples. A population is defined as the entire collection of all possible occurrences of a given quantity. It may be either finite or infinite. A sample is part of a population and should have the same characteristics, or be representative, of that population.

To facilitate a typical statistical analysis, such as the probability of a certain flood event from a sample of infinite population, the concept of frequency distributions is used. A frequency distribution is an arrangement of data by classes or categories, with associated frequencies for each class. The frequency distribution is used to determine the magnitude of past events, as well as how often events of a specified magnitude have occurred.

A frequency distribution is constructed by first examining the range of magnitudes, i.e., the difference between the largest and the smallest floods, and dividing this range into a number of conveniently sized groups, usually between 10 and 20. These groups are called class intervals. The size of the class interval is the range divided by the number of class intervals selected. The following guidelines may be helpful in selecting the number of class intervals:

1. The class intervals should not overlap; for example, use 0-99, 100-199, rather than 0-100, 100-200.

2. The number of class intervals that have no events should be limited.

3. The class intervals should be of uniform size.

Guidelines for determining floodflow frequencies from observed streamflow data may be obtained from Bulletin 17B of the U.S. Water Resources Council (revised 1981). A literature review and evaluation of urban floodflow frequency procedures is presented by Rawls et al. (1980). The U.S. Geological Survey (USGS) is the primary government agency responsible for collection of streamflow data and maintenance of systematic peak discharge information. These data are reported in USGS Water Supply Papers, Annual Surface Water Records, and computer files.

2.4.1 TYPES OF DATA SERIES

The following types of sample data are most common:

1. Complete duration series
2. Annual series
3. Partial duration series
4. Extreme value series

The complete duration series includes all available data; the annual series consists of one value per year (typically the maximum peak flow). The partial duration series includes all peak flows above a selected base value. The extreme value series consists of the largest observation in a given time interval. Note that the annual series is a special type of extreme value series, with a time interval of 1 year.

The annual and partial duration series give similar results for return periods beyond 10 years. For return periods less than 10 years, the annual series gives higher return periods for the same peak flow rates. A comparison of the frequency curves developed using the annual and partial duration series is presented in Figure 2-10.

2.4.2 PLOTTING POSITION

Assigning a frequency to a data point is commonly referred to as determining the plotting position. This determination for sample data is never conclusive, as there is no assurance that a sample contains the smallest and largest values of the total population.

Several plotting position relationships are discussed by Chow (1964) and general criteria for plotting position relationships are presented by Gumbel (1958). In a comparative study of plotting position relationships, Benson (1962) found that the Weibull relationship provided estimates that were consistent with experience. The Weibull relationship is expressed as:

$$P = \frac{m}{n + 1} \tag{2-3}$$

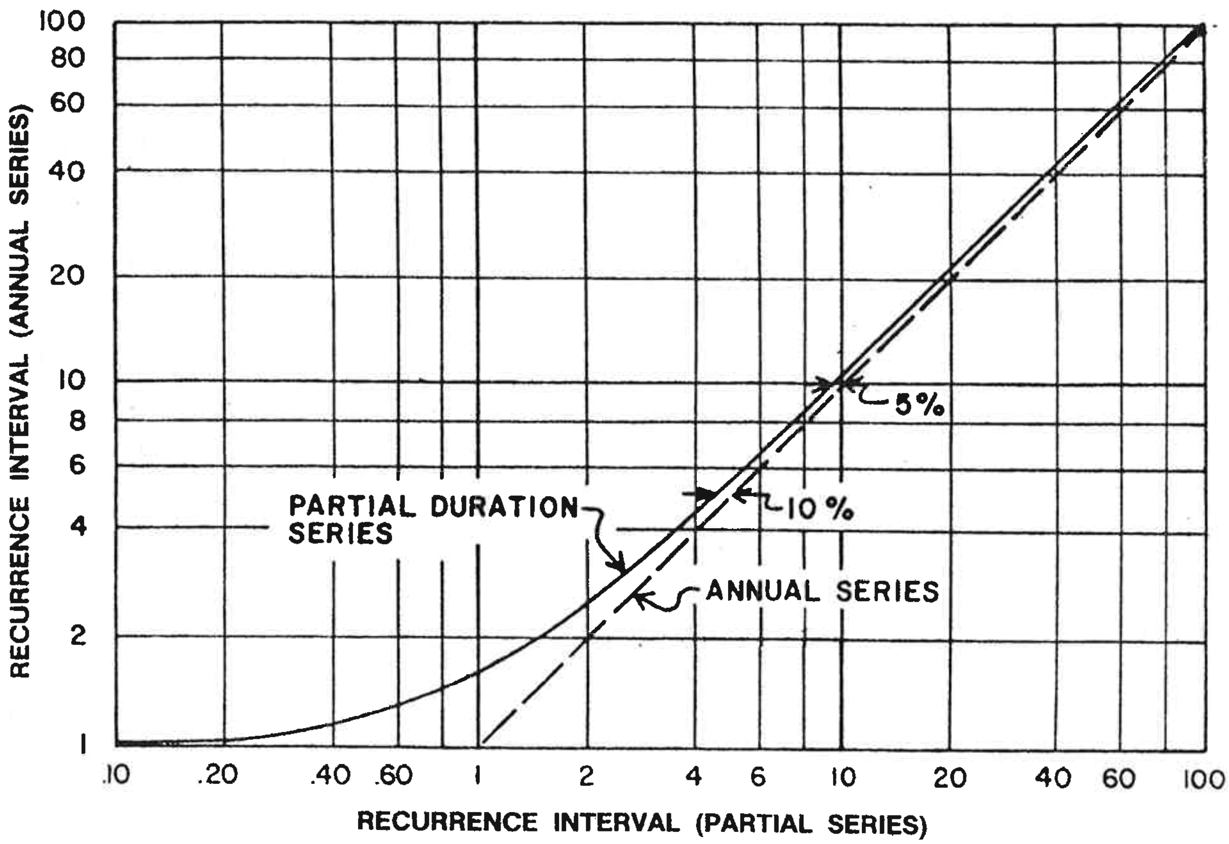
where:

P = Plotting position

m = Order number of the event from highest to lowest

n = Number of years of record

For partial duration series in which the number of floods exceeds the number of years of record, Beard (1962) recommends the expression:



Reference: USDOT, FHWA, HEC-19 (1984).

FIGURE 2-10
Relationship Between Annual and Partial Duration Series

$$P = \frac{2m - 1}{2n} \qquad (2\text{-}4)$$

where:

P = Plotting position

m = Order number of the event from highest to lowest

n = Number of years of record

2.4.3 STANDARD DISTRIBUTIONS

Several frequency distributions are commonly applied to the analysis of streamflow data and, as a result, have been extensively studied and standardized. The following standard frequency distributions are most useful:

1. Normal distribution
2. Log-normal distribution
3. Gumbel extreme value distribution
4. Log-Pearson Type III distribution

Mathematical fundamentals for these standard frequency distributions can be found in publications by Haan (1977), Riggs (1968a and b), Benjamin and Cornell (1970), and USDOT, FHWA (HEC-19, 1984).

Normal Distribution

The normal, or Gaussian, distribution is a classical mathematical distribution occurring in the analysis of natural phenomena. The normal distribution is a symmetrical, unbounded, bell-shaped curve with the maximum value at the central point and extending from $-\infty$ to $+\infty$.

For the normal distribution, the maximum central value occurs at the mean flow. Because of absolute symmetry, half of the flows are below the mean and half are above. The median, therefore, corresponds to the mean value. Another characteristic of the normal distribution curve is that 68.3 percent of the events will fall between ± one standard deviation, 95 percent will fall within ± two standard deviations, and 99.7 percent will fall within ± three standard deviations.

A disadvantage of the normal distribution is that it is unbounded in the negative direction, whereas most hydrologic variables are bounded and can never be less than zero. For this reason, and because many hydrologic variables exhibit a pronounced skew, the normal distribution usually has limited application. Because the logarithms of hydrologic variables are often normally distributed, a log transformation of the data can sometimes overcome this limitation.

Log-Normal Distribution

The log-normal distribution has the same characteristics as the normal distribution except that the independent variable is replaced with its logarithm. The log-normal distribution is bounded on the left by zero and has a pronounced positive skew. These characteristics are common to many frequency distributions obtained from analysis of hydrologic data.

Gumbel Extreme Value Distribution

The Gumbel extreme value distribution, sometimes called the double exponential distribution of extreme values, can also be used to describe the distribution of hydrologic variables, especially peak discharges. The characteristics of the Gumbel extreme value distribution are occurrence of mean flow at a return period of 2.33 years and a positive skew, i.e., toward the high flows or extreme values.

Log-Pearson Type III Distribution

Another distribution that has found wide application in hydrologic analysis is the log-Pearson Type III distribution, which is a three-parameter gamma distribution with a logarithmic transform of the independent variable. It is one of a number of standard distributions that have been developed, more or less empirically, for application to statistical problems. Its advantages are its ability to conform with available data and its flexibility, which allows its use in a variety of distributions. This flexibility has led the U.S. Water Resources Council to recommend its use as the standard distribution for flood frequency studies by all U.S. government agencies.

Mean flow, standard deviation, and coefficient of skew are necessary to describe the log-Pearson Type III distribution. Judicious selection of these three parameters makes it possible to fit almost any shape of distribution. An extensive description of the use of this distribution in determining flood frequency distributions is presented in Bulletin 17B by the U.S. Water Resources Council (revised 1981).

2.4.4 STANDARD ERROR AND CONFIDENCE LIMITS

Since more than one distribution may fit given streamflow data, quantitative measures should be used to decide which distribution is most suitable. Two of the most common measures are the standard error of estimate and confidence limits. Mathematical relationships for these parameters can be obtained in publications by Haan (1977), Benjamin and Cornell (1970), or USDOT, FHWA (HEC-19, 1984).

Standard Error of Estimate

The standard error of estimate is inversely proportional to the square root of the period of record; that is, the shorter the period of record, the larger the standard error. For example, standard errors for a short record will be approximately twice as large as those for a record four times as long.

Confidence Limits

Confidence limits are used to estimate the uncertainties associated with the determination of floods of specified return periods from frequency distributions. Since a given frequency distribution is only an estimated determinant from one sample, it is probable that another sample of equal length but taken at a different time from the same stream would yield a different frequency curve. Confidence limits, or more correctly, confidence intervals, define the range within which these frequency curves could be expected to fall, with specified confidence or levels of significance (see U.S. Water Resources Council, revised 1981).

2.5 TERMINOLOGY

2.5.1 HYDROGRAPHS

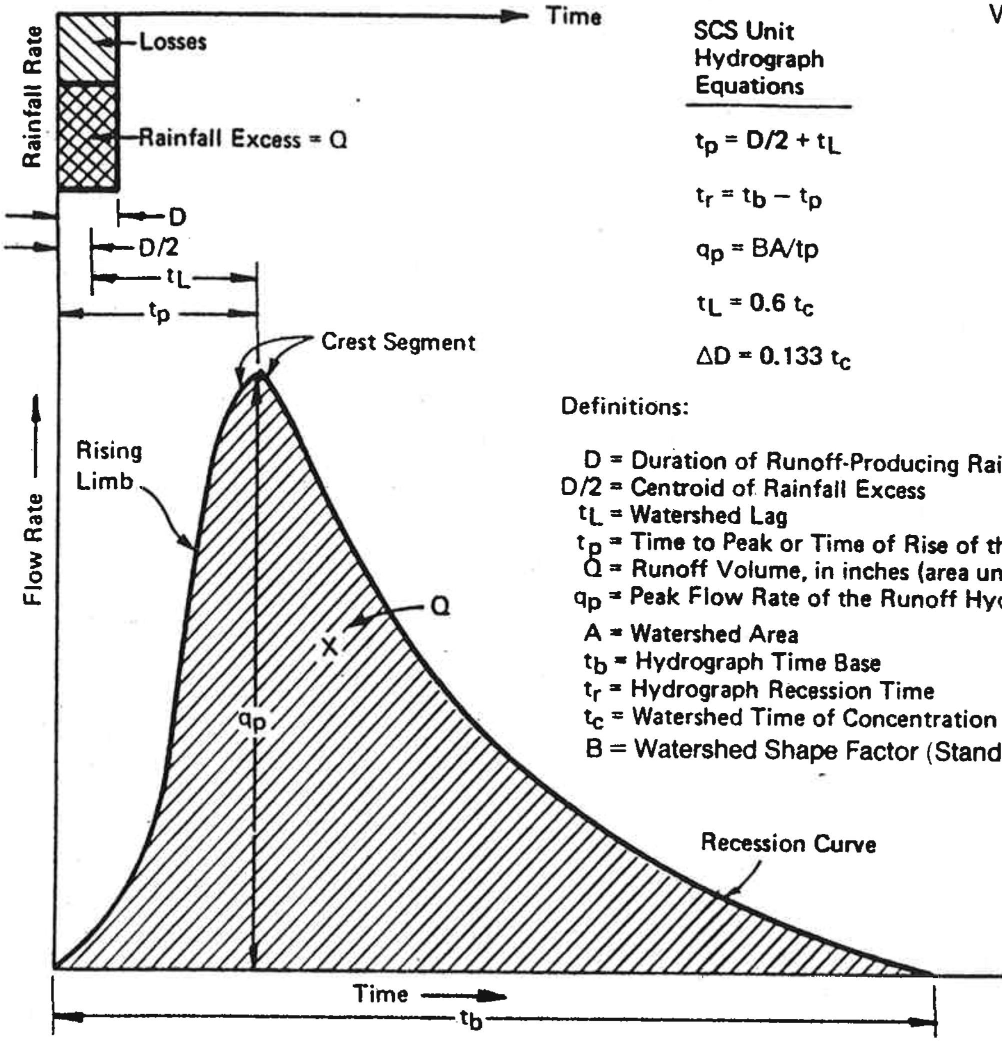
A flood hydrograph is a continuous plot of the surface runoff flow rate versus time. It is produced when a particular watershed is subjected to an input of rainfall excess. By definition, the volume of water contained in the flood hydrograph is equal to the volume of water contained in the rainfall excess hyetograph. Although historical streamflow data are preferred, they are not generally collected for the typical watershed; thus, synthetic methods for developing flood hydrographs are often required. If historical data are not available for a given basin, then data from a similar basin may be appropriate.

A typical hydrograph resulting from an isolated period of rainfall consists of a rising limb, a crest segment, and a falling or recession limb. These components are illustrated in Figure 2-11. The shape of the rising limb is influenced primarily by the characteristics that produced the surface runoff. The point of inflection on the recession limb of the hydrograph is commonly assumed to mark the time at which surface inflow to the channel system ceases (see Figure 2-12). Thereafter, the recession curve represents the withdrawal of water from storage within the watershed. As a result, the recession limb is largely independent of the storm and is influenced instead by watershed characteristics, such as channel slope and storage availability.

Typical hydrograph components are presented in Figure 2-11, along with appropriate SCS hydrograph equations. A rainfall excess hyetograph, which in this case is a single block of rainfall excess over duration, D, is shown in the upper part of Figure 2-11. The runoff hydrograph is presented directly below the rainfall excess hyetograph. The area enclosed by the hyetograph and by the runoff hydrograph represents the same volume, Q, of direct runoff. The maximum flow rate of the hydrograph is the peak flow, q_p.

2.5.2 TIME PARAMETERS

The time from the start of the hydrograph to q_p is the time to peak, t_p. The total time duration of the hydrograph is known as the time base, t_b. The watershed lag time, t_L, is



SCS Unit Hydrograph Equations

$t_p = D/2 + t_L$

$t_r = t_b - t_p$

$q_p = BA/tp$

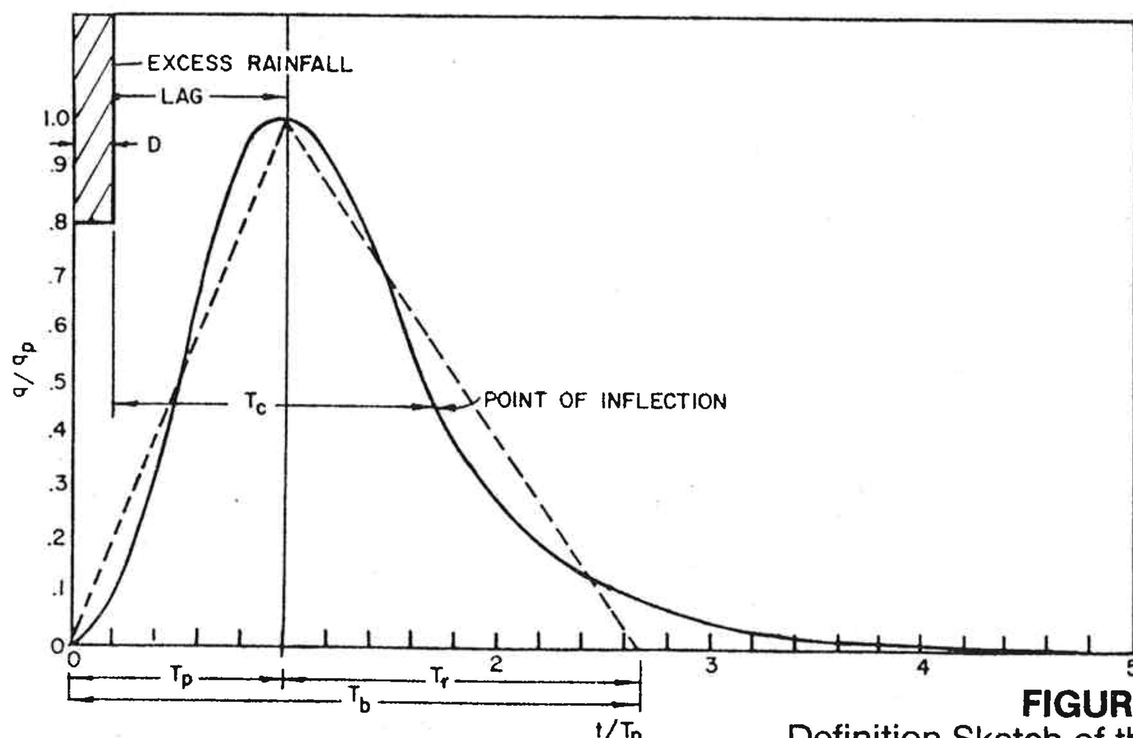
$t_L = 0.6\ t_c$

$\Delta D = 0.133\ t_c$

Definitions:

- D = Duration of Runoff-Producing Rainfall in Hours
- $D/2$ = Centroid of Rainfall Excess
- t_L = Watershed Lag
- t_p = Time to Peak or Time of Rise of the Runoff Hydrograph
- Q = Runoff Volume, in inches (area under the curve)
- q_p = Peak Flow Rate of the Runoff Hydrrograph in cfs
- A = Watershed Area
- t_b = Hydrograph Time Base
- t_r = Hydrograph Recession Time
- t_c = Watershed Time of Concentration
- B = Watershed Shape Factor (Standard SCS B = 484)

FIGURE 2-11
General Hydrograph Terminology



Reference: USDA, SCS, NEH-4 (1972).

FIGURE 2-12
Definition Sketch of the SCS Dimensionless Unit Hydrographs

defined as the time from the center of mass of the rainfall excess to the runoff hydrograph peak. The following equation summarizes the relationship between the time parameters for the rising limb of a direct runoff hydrograph:

$$t_p = \frac{D}{2} + t_L \qquad (2\text{-}5)$$

where:

t_p = Time to peak or time of rise of the runoff hydrograph

D = Duration of runoff-producing rainfall

t_L = Watershed lag time

The recession time for a hydrograph is the difference between the time base and the time to peak and can be expressed as:

$$t_r = t_b - t_p \qquad (2\text{-}6)$$

where:

t_r = Hydrograph recession time

t_b = Hydrograph time base

t_p = Time to peak or time of rise of the runoff hydrograph

The time of concentration, t_c, is the time required for a hydraulic wave to travel across the watershed. It is often approximated as the time required for runoff to travel from the hydraulically most remote part of the watershed to the point of reference.

Many empirical relationships are available for estimating the value of t_c from site-specific watershed data. Since these relationships can be developed to estimate different components of the total watershed time of concentration, an understanding of the conditions for which they are applicable is essential before putting them to use. Empirical methods should not be used unless it can be demonstrated that they are suitable for actual site conditions.

A segmental approach, commonly known as the velocity method, typically requires evaluation of the following three flow components:

1. Overland flow
2. Rill, shallow channel, and street gutter flow
3. Open channel flow

Overland flow is sheet flow over plane surfaces, usually limited to a maximum length of 300 feet. After 300 feet, overland flow generally becomes concentrated into rills, small channels, or gutters. Open channel flow is appropriate when the main conveyance system is encountered.

The travel time for any flow segment can be estimated as the ratio of flow length to the velocity of flow. The number of flow segments or flow components considered should best represent the actual flow path of the watershed being evaluated. The relationship for calculating the time of concentration is expressed as:

$$t_c = t_1 + t_2 + t_3 + \ldots t_i \qquad (2\text{-}7)$$

where:

t_c = Watershed time of concentration

t_1 = Overland flow travel time

t_2 = Rill, shallow channel, and street gutter flow travel time

t_3 = Open channel flow travel time

t_i = Travel time for the i^{th} segment

Procedures for estimating values for t_1, t_2, and t_3 are presented in Volume 2. In general, the Manning kinematic wave equation (Ragan, 1971) should be used for overland flow. Data presented in SCS TR-55 (1984) or Manning's Equation can be used to evaluate rill, shallow channel, and street gutter flow travel times. Manning's Equation is generally used to evaluate the open channel flow travel time for the main conveyance system.

2.6 PEAK RUNOFF RATES

Peak runoff rates are automatically obtained when a flood hydrograph is developed. In some situations, however, a peak runoff rate can be obtained without developing a complete flood hydrograph, through use of the Rational Method, regression equations, or USDA, SCS, TR-55 (1986).

2.6.1 RATIONAL METHOD

According to the Rational Method, the peak runoff rate can be estimated as the product of a runoff coefficient, a rainfall intensity, and the drainage area. The Rational Method is frequently expressed mathematically as:

$$Q = CIA \tag{2-8}$$

where:

Q = Peak runoff rate, in cubic feet per second (cfs)

C = Rational Method runoff coefficient, dimensionless

I = Average rainfall intensity for the design storm, in inches/hour

A = Watershed drainage area, in acres

To aid in the analysis of parameters used in the Rational Method, the following form is used in this manual:

$$Q_T = C_T \, I_{t_c} \, A \tag{2-9}$$

where:

Q_T = Peak runoff rate for return period T, in cfs

C_T = Rational Method runoff coefficient, expressed as the dimensionless ratio of rainfall excess to total rainfall, for the design storm return period T

I_{t_c} = Average rainfall intensity, in inches/hour, during a period of time equal to t_c for the design storm return period T (i.e., I_{t_c} is a function of t_c)

A = Watershed drainage area, in acres, tributary to the design point

t_c = Watershed time of concentration, in minutes, defined as the time required for a wave to travel from the hydraulically most distant point of a watershed to the design point

To be dimensionally correct, a conversion factor of 1.008 should be used to convert acre-inches per hour to cfs. In practice, this factor is usually neglected.

The basic assumptions behind the Rational Method include the following made by Rossmiller (1980):

1. Runoff is linearly related to rainfall.
2. The rainfall occurs uniformly over a given watershed.
3. The peak runoff rate occurs when the entire area is contributing flow.
4. The rainfall excess hyetograph is one of constant intensity for a duration equal to t_c.
5. The frequency of the peak runoff rate is the same as the frequency of the average rainfall intensity.

Because these assumptions are generally valid for only small watersheds (less than 200 acres), extra effort is warranted if larger areas are concerned. Comparison of results from the Rational Method to results from other procedures and historical observations is often appropriate.

2.6.2 REGRESSION EQUATIONS

A regression equation is a regional method that correlates peak discharge with physical factors such as watershed area and stream slope. The development of regional regression equations requires considerable field data and a major work effort. Streamflow data must be subjected to a statistical analysis (as discussed in this chapter); then the discharge-frequency curves must be correlated with watershed characteristics. Procedures for developing regression equations are presented in publications by Haan (1977) and Benjamin and Cornell (1970). Regression equations that have been developed for use in Metropolitan Nashville and Davidson County are presented in Volume 2.

2.6.3 SCS TR-55

The SCS has prepared TR-55 (1986) to provide simplified procedures for developing peak runoff rates in urban areas. These procedures are largely based on results of hydrologic calculations obtained using the SCS computer program TR-20 (1978). The methods provided in TR-55 are intended to be used only for overland and open channel flow conditions. If storm sewer systems are involved, TR-55 is not valid.

2.7 UNIT HYDROGRAPH THEORY

The concept of a unit hydrograph published by L. K. Sherman in 1932 provides a widely accepted basis for converting rainfall excess from a watershed to a runoff hydrograph. Although the tools and data available for developing unit hydrographs are more extensive than when Sherman first proposed unit hydrograph theory, the concept has not changed.

A unit hydrograph is defined as a runoff hydrograph that is produced by 1 inch of rainfall excess distributed uniformly over a watershed and occurring at a uniform rate during a specified period of time. Sherman originally used "unit" to denote the specified duration of rainfall excess for a particular unit hydrograph. The word "unit" is often misinterpreted as 1 inch or a "unit depth" of effective rainfall excess rather than as a "unit of time" for rainfall excess as originally intended.

The following assumptions constitute the basis of unit hydrograph theory:

1. The rainfall excess is uniformly distributed within its unit duration or specified period of time.

2. The rainfall excess is uniformly distributed in space over a particular watershed.

3. The time base for a direct runoff hydrograph due to a rainfall excess of unit duration is constant.

4. The ordinates of the direct runoff hydrographs, when a common unit duration is considered, are directly proportional to the total volume of direct runoff represented by each hydrograph (principle of linearity or superposition).

5. For a given drainage basin, the hydrograph of runoff caused by a unit duration and volume of rainfall excess is invariable (principle of time invariance).

These assumptions cannot be precisely applied to natural precipitation and drainage basin characteristics. However, experience has shown that the unit hydrograph method gives results that are sufficiently accurate for most drainage problems.

The two fundamental assumptions that must always be considered when applying unit hydrograph theory are the principle of linearity (Assumption 4) and the principle of time invariance (Assumption 5). Theoretically, each increment of rainfall excess can be routed through the subject watershed in accordance with the principle of linearity. In practice, this means that the product of a rainfall excess volume and the sequence of unit hydrograph ordinates (i.e., runoff rates in cfs per inch of rainfall excess) produces an estimate of the runoff hydrograph for that volume of rainfall excess. In addition, the principle of linearity allows individual runoff hydrographs developed from a sequence of individual rainfall excess volumes (i.e., a design storm of rainfall excess increments arranged in units of time equal to the unit duration) to be superimposed and added when estimating a total runoff hydrograph. The principle of time invariance requires that the hydrologic characteristics of the drainage basin be fixed or specified for a particular unit hydrograph. Land development and channel improvements are typical activities that violate the principal of time invariance.

The unit hydrograph method was originally devised for large drainage basins. However, Brater (1940) showed that unit hydrograph theory was also applicable to small drainage basins varying in size from 4 acres to 10 square miles. Dooge (1973) provides a good discussion of unit hydrograph theory, including the definition and application of the instantaneous unit hydrograph.

Unit hydrographs can be developed either from observed precipitation and streamflow records or from synthetic unit hydrograph procedures using site-specific watershed characteristics. Theoretical aspects of several procedures are presented below.

2.7.1 DERIVATION FROM OBSERVED DATA

If available, observed precipitation and streamflow data should be used to derive a unit hydrograph. Streamflow data for a reasonably uniform intensity storm of the desired duration with a relatively large runoff volume are best suited for this derivation. Base flow, if significant, must be separated from direct runoff. Additional details on the derivation of a unit hydrograph from observed streamflow data can be found in Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), Wisler and Brater (1959), Chow (1964), Dooge (1973), and USDOT, FHWA (HEC-19, 1984).

2.7.2 SCS DIMENSIONLESS UNIT HYDROGRAPH

The SCS has derived two dimensionless unit hydrographs from a large number of observed unit hydrographs for watersheds of various sizes and geographic locations. One type has a curvilinear shape; the second is a triangular approximation to that curvilinear shape. In both cases, once the time to peak and peak flow for a particular unit hydrograph have been defined, the entire shape of the unit hydrograph can be estimated with the appropriate dimensionless unit hydrograph.

Figure 2-12 presents the SCS dimensionless curvilinear unit hydrograph, which has 37.5 percent of its total volume on the rising side, corresponding to one unit of time and one unit of discharge. The triangular approximation to this SCS curvilinear hydrograph has the same percent of volume on the rising side of the triangle. Therefore, one unit of time, t_p, equals 0.375 unit of the discharge volume, and the time base of this triangle is 2.67 units of time (1.0/0.375), while the recession limb is 1.67 units of time ($t_b - t_p$).

Having established the shape and the time axis for the dimensionless unit hydrograph, only one value for the peak flow, q_p, will make the volume under the unit hydrograph equal to 1 inch of rainfall excess. This value can be determined by the equation:

$$q_p = 60 \frac{BA}{t_p} \qquad (2\text{-}10)$$

where:

q_p = Unit hydrograph peak discharge, in cfs

B = Hydrograph shape factor (Note: The SCS dimensionless unit hydrograph B value of 484 should be used in Nashville and Davidson County)

A = Drainage area, in square miles

t_p = Time to peak, in minutes (time from the beginning of rainfall excess to the peak flow rate)

The hydrograph shape factor, B, is generally considered to be a constant characteristic of a watershed. The SCS dimensionless unit hydrographs shown in Figure 2-12 are based on a B value of 484. However, since the value of B can be expected to range from 600 in steep terrain to 300 in flat swampy areas, adjustments to the unit hydrograph shape may be warranted. These adjustments are accomplished by changing the percent of volume under the rising and recession limbs of the unit hydrograph to reflect the corresponding change in the hydrograph shape factor. The standard SCS value of 484 should be used in Nashville and Davidson County unless another value is approved by MDPW.

The dimensionless unit hydrographs shown in Figure 2-12 have a time to peak at 1 unit of time and the point of inflection at approximately 1.7 units of time. Using the SCS average relationship of lag time to time of concentration, given as $t_L = 0.6\ t_c$, and the point of inflection, given as $1.7\ t_p$, the following relationships are obtained:

$$t_c + \Delta D = 1.7\ t_p \qquad (2\text{-}11)$$

$$t_p = \frac{\Delta D}{2} + 0.6\ t_c \qquad (2\text{-}12)$$

where:

t_c = Watershed time of concentration

ΔD = Duration of rainfall excess increment

t_p = Time to peak (time from the beginning of rainfall excess to the peak flow rate)

which can be solved simultaneously to give:

$$\Delta D = 0.133 t_c \qquad (2\text{-}13)$$

2.7.3 CLARK UNIT HYDROGRAPH

The Clark method (1945) requires three parameters to calculate a unit hydrograph: t_c, the time of concentration for the basin, R, a storage coefficient, and a time-area curve. A time-area curve defines the cumulative area of the watershed contributing runoff to the subbasin outlet as a function of time (expressed as a proportion of t_c). The Clark unit hydrograph is most applicable in areas where observed streamflow indicates that the SCS shape factor of

484 is not appropriate, or in subbasins where a specific time-area curve is needed to describe an unusual shape.

A time-area curve can be developed using a dimensionless time-area curve as presented by the U.S. Army Corps of Engineers (1982) in the HEC-1 computer program and expressed as:

$$AI = 1.414T^{1.5} \qquad 0 \leq T < .5 \qquad (2\text{-}14)$$

$$1 - AI = 1.414\ (1-T)^{1.5} \qquad .5 \leq T \leq 1 \qquad (2\text{-}15)$$

where:

AI = Cumulative drainage area as a fraction of total subbasin area

T = Fraction of time of concentration

The ordinates of the time-area curve are converted to volume of runoff per second for unit excess and interpolated to the given time interval. The resulting translation hydrograph is then routed through a linear reservoir to simulate the storage effects of the basin, and the resulting unit hydrograph for instantaneous excess is averaged to produce the hydrograph for unit excess occurring in the given time interval.

The linear reservoir routing uses the general equation:

$$Q(2) = (CA)\ (I) + (CB)\ Q(1) \qquad (2\text{-}16)$$

and the routing coefficients are calculated from:

$$CA = \Delta t/(R + .5\ \Delta t) \qquad (2\text{-}17)$$

$$CB = 1 - CA \qquad (2\text{-}18)$$

$$QUNGR = .5[Q(1) + Q(2)] \qquad (2\text{-}19)$$

where:

Q(2) = Instantaneous flow at the end of period, in cfs

Q(1) = Instantaneous flow at the beginning of period, in cfs

I = Ordinate of the translation hydrograph

Δt = Computation time interval, in hours

QUNGR = Unit hydrograph ordinate at the end of the computation interval

The computation of unit hydrograph ordinates is terminated for the HEC-1 computer program when its volume exceeds 0.995 inch or 150 ordinates, whichever occurs first.

2.7.4 URBAN REGRESSION EQUATIONS

Observed streamflow data for small urban watersheds have been analyzed to develop regression equations for deriving synthetic unit hydrographs. Equations for unit durations of 10 minutes (Espey et al., 1978) and 30 minutes (Hamm et al., 1973) are available. Site-specific parameters required to use the equations include main channel length, L, and slope, S, impervious cover, I, drainage area, A, and the watershed conveyance factor, Φ. The watershed conveyance factor is intended to account for the reduction in the time of rise caused by channel improvements or storm sewers. The two components of this factor account for channel characteristics, Φ_1, and vegetation characteristics, Φ_2. The conveyance factor is calculated as the sum of these two components.

The unit hydrograph regression equations provide a basis for estimating the time of rise, t_r, peak runoff rate, q_p, time base, t_b, and unit hydrograph widths at one-half and three-fourths of the peak discharge, W_{50} and W_{75}. Using these shape parameters, a unit hydrograph with 1 inch of runoff volume can be constructed.

2.8 HYDROLOGIC ROUTING

After a flood hydrograph has been developed for a particular watershed, it may be necessary to route that hydrograph to another point in the drainage system without allowing for additional flow. This process is commonly known as flood routing.

Hydrologic and hydraulic routing techniques are available to quantify the peak flow attenuation and time lag likely to occur when a flood travels through a drainage system. However, hydrologic routing considers only the conservation of mass, whereas hydraulic routing considers both the conservation of mass and the equations of motion. Since, in practice, hydrologic routing techniques are usually adequate for stormwater design purposes, the scope of this section will be limited to their use. For information on hydraulic routing techniques, references by Henderson (1966), Viessman et al. (1977), and Chow (1959, 1964) should be consulted.

2.8.1 INPUT DATA

Any flood routing technique requires the following three types of input data:

1. An inflow hydrograph
2. A stage-storage relationship
3. A stage-discharge relationship

An inflow hydrograph can be developed using the fundamentals discussed earlier. The stage-storage and stage-discharge relationships are developed to account for the characteristics of the channel system in question. In practice, it may be convenient to combine these two relationships into a single storage-discharge relationship. The storage-discharge relationships for channels are usually quite different from those for reservoirs, because storage in a channel may depend on both inflow and outflow.

2.8.2 CONTINUITY EQUATION

Hydrologic flood routing techniques are all based upon the continuity equation, which requires that the rate of change to storage in a drainage system must account for all mass flow into and out of that system. Mathematically, the continuity equation is expressed as:

$$I - O = \frac{\Delta S}{\Delta t} \qquad (2\text{-}20)$$

where:

I = Inflow rate to the drainage system, in cfs

O = Outflow rate from the drainage system, in cfs

$\frac{\Delta S}{\Delta t}$ = The rate of change to storage in a drainage system, in cfs

As noted above, the storage within a particular channel segment will generally depend on both the inflow to and the outflow from that channel segment. In practice, it is usually acceptable to approximate this dual characteristic of channel storage by dividing the total storage volume into two components. The first component depends only on the outflow rate and is commonly known as prism storage. The second component is wedge storage, which depends on the difference between inflow and outflow rates. This two-component approximation of the channel storage relationship is illustrated in Figure 2-13.

A general mathematical relationship for expressing this two-component channel storage volume is presented by Chow (1959) as:

$$S = \frac{b}{a} \quad XI^{m/n} + (1-X)O^{m/n} \qquad (2\text{-}21)$$

where:

S = Channel storage volume, in cubic feet

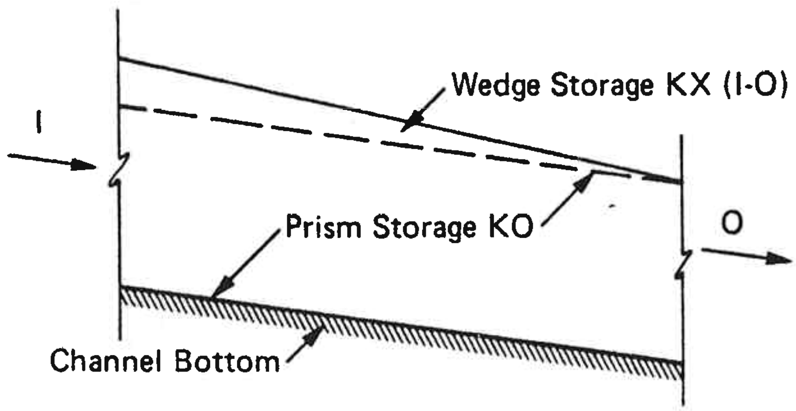
I = Inflow rate to the channel, in cfs

O = Outflow rate from the channel, in cfs

X = Dimensionless factor that determines the relative weights of I and O on the channel storage volume

a and n = Constants that reflect the stage-discharge characteristics of the channel segment

b and m = Constants that reflect the stage-storage characteristics of the channel segment



Note: Channel Storage, S = KO + KX (I-O)

FIGURE 2-13
Two-Component Channel Storage Approximation