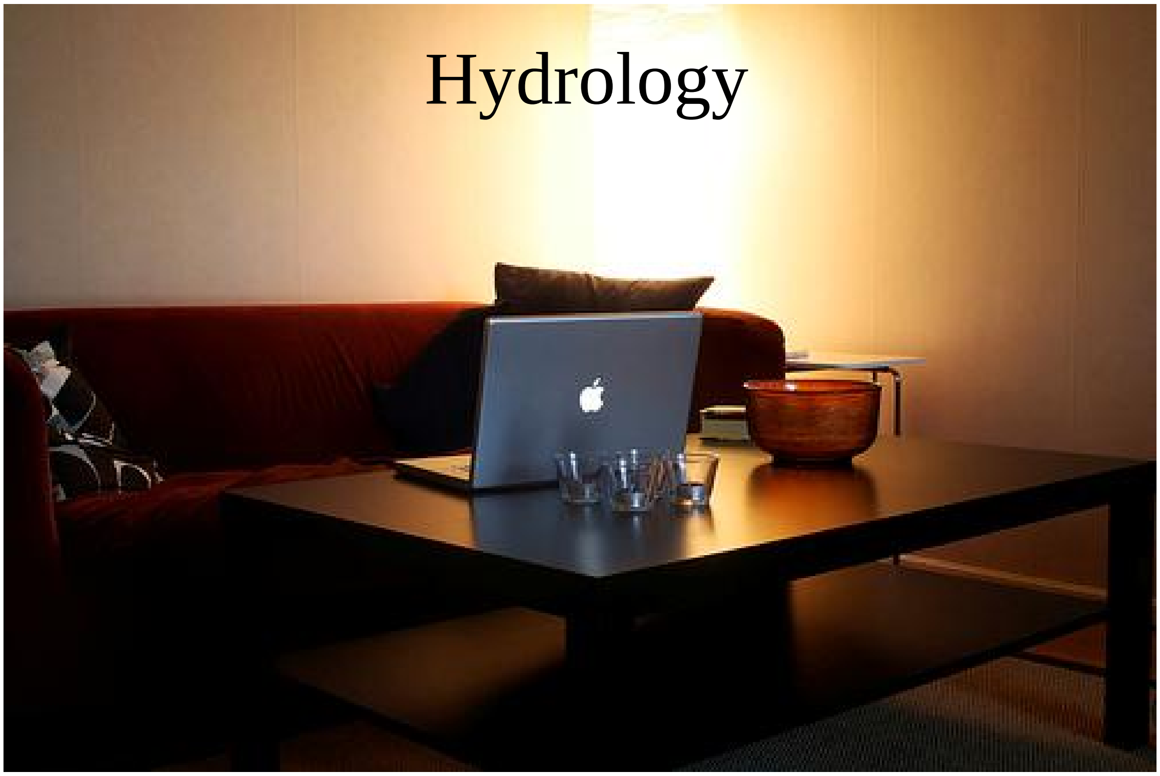

Hydrology

Part I - Rainfall/Runoff Processes
Part II - Peak Flow for Ungaged Sites

By

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PDHLibrary Course No 2019003
7 PDH HOURS

HDROLOGY PART I – RAINFALL/RUNOFF PROCESSES

RAINFALL/RUNOFF PROCESSES

The runoff process can be defined as that collection of interrelated natural processes by which water, as precipitation, enters a watershed and then leaves as runoff. In other words, surface runoff is the portion of the total precipitation that has not been removed by processes in the hydrologic cycle. The amount of precipitation that runs off from the watershed is called the "rainfall excess", and "hydrologic abstractions" is the commonly used term that groups all of the processes that extract water from the original precipitation. It follows then that the volume of surface runoff is equal to the volume of rainfall excess, or, in the case of the typical highway problem, the runoff is the original precipitation less infiltration and storage.

The primary purpose of this course is to describe more fully the runoff process. An understanding of the process is necessary to properly apply hydrologic design methods. Pertinent aspects of precipitation are identified and each of the hydrologic abstractions is discussed in some detail. The important characteristics of runoff are then defined together with how they are influenced by different features of the drainage basin. The course includes a qualitative discussion of the runoff process, beginning with precipitation and illustrating how this input is modified by each of the hydrologic abstractions. Because the time characteristics of runoff are important in design, a discussion of runoff travel time parameters is included.

PRECIPITATION

Precipitation is the water that falls from the atmosphere in either liquid or solid form. It results from the condensation of moisture in the atmosphere due to the cooling of a parcel of air. The most common cause of cooling is dynamic or adiabatic lifting of the air. Adiabatic lifting means that a given parcel of air is caused to rise with resultant cooling and possible condensation into very small cloud droplets. If these droplets coalesce and become of sufficient size to overcome the air resistance, precipitation in some form results.

Forms of Precipitation

Precipitation occurs in various forms. Rain is precipitation that is in the liquid state when it reaches the earth. Snow is frozen water in a crystalline state, while hail is frozen water in a 'massive' state. Sleet is melted snow that is an intermixture of rain and snow. Of course, precipitation that falls to earth in the frozen state cannot become part of the runoff process until melting occurs. Much of the precipitation that falls in mountainous areas and in the northerly latitudes falls in the frozen form and is stored as snowpack or ice until warmer temperatures prevail.

Types of Precipitation (by Origin)

Precipitation can be classified by the origin of the lifting motion that causes the precipitation. Each type is characterized by different spatial and temporal rainfall regimens. The three major types of storms are classified as convective storms, orographic storms, and cyclonic storms. A fourth type of storm is often added, the hurricane or tropical cyclone, although it is a special case of the cyclonic storm.

Convective Storms

Precipitation from convective storms results as warm moist air rises from lower elevations into cooler overlying air as shown in Figure 1. The characteristic form of convective precipitation is the summer thunderstorm. The surface of the earth is warmed considerably by mid- to late afternoon of a summer day, the surface imparting its heat to the adjacent air. The warmed air begins rising through the overlying air, and if proper moisture content conditions are met (condensation level), large quantities of moisture will be condensed from the rapidly rising, rapidly cooling air. The rapid condensation may often result in huge quantities of rain from a single thunderstorm spawned by convective action, and very large rainfall rates and depths are quite common beneath slowly moving thunderstorms.

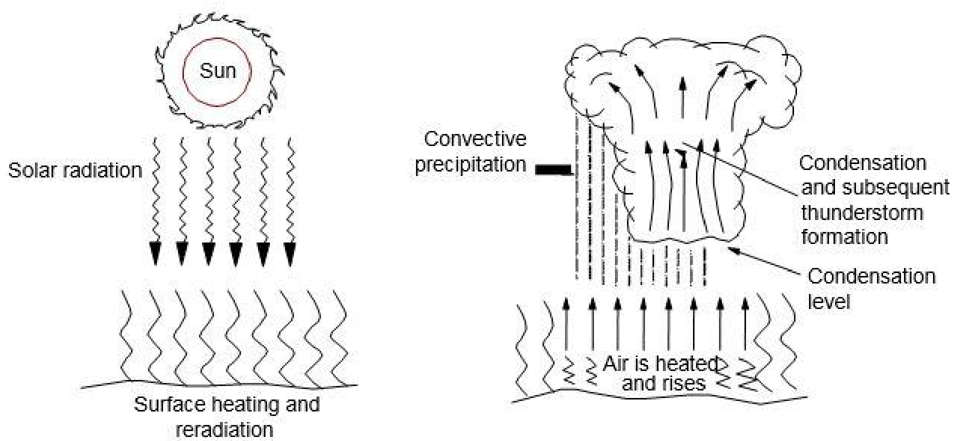

Figure 1. Convective storm

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Orographic Storms

Orographic precipitation results as air is forced to rise over a fixed-position geographic feature such as a range of mountains (see Figure 2). The characteristic precipitation patterns of the Pacific coastal states are the result of significant orographic influences. Mountain slopes that face the wind (windward) are much wetter than the opposite (leeward) slopes. In the Cascade Range in Washington and Oregon, the west-facing slopes may receive upwards of 100 inches of precipitation annually, while the east-facing slopes, only a short distance away over the crest of the mountains, receive on the order of 20 inches of precipitation annually.

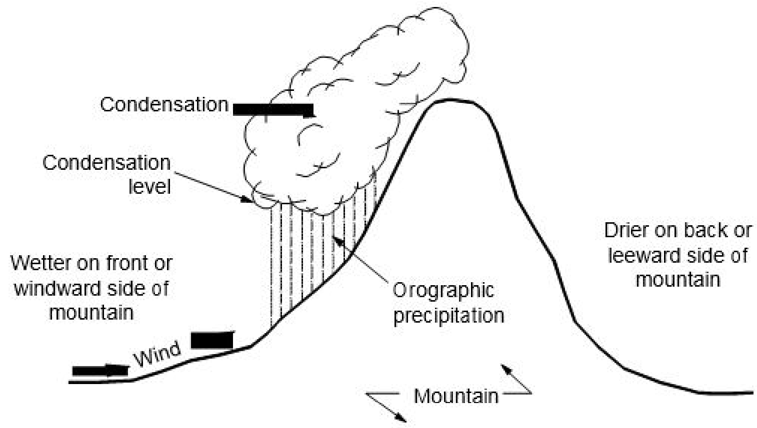

Figure 2. Orographic storm

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Cyclonic Storms

Cyclonic precipitation is caused by the rising or lifting of air as it converges on an area of low pressure. Air moves from areas of higher pressure toward areas of lower pressure. In the middle latitudes, cyclonic storms generally move from west to east and have both cold and warm air associated with them. These mid-latitude cyclones are sometimes called extra-tropical cyclones or continental storms.

Continental storms occur at the boundaries of air of significantly different temperatures. A disturbance in the boundary between the two air parcels can grow, appearing as a wave as it travels from west to east along the boundary. Generally, on a weather map, the cyclonic storm will appear as shown in Figure 3, with two boundaries or fronts developed. One has warm air being pushed into an area of cool air, while the other has cool air pushed into an area of warmer air. This type of air movement is called a front; where warm air is the aggressor, it is a warm front, and where cold air is the aggressor, it is a cold front (see Figure 4). The precipitation associated with a cold front is usually heavy and covers a relatively small area, whereas the precipitation associated with a warm front is more passive, smaller in quantity, but covers a much larger area. Tornadoes and other violent weather phenomena are associated with cold fronts.

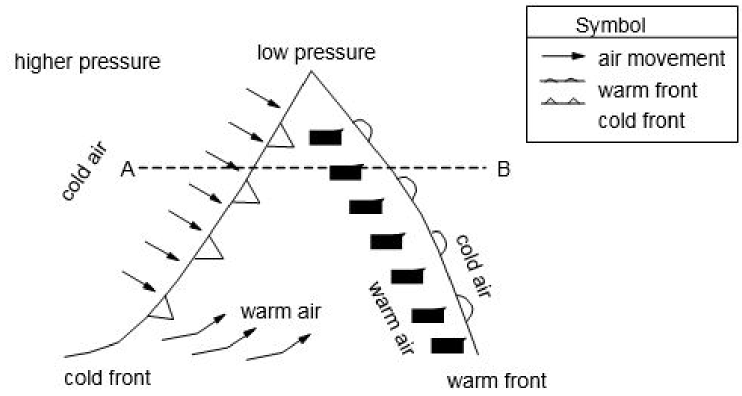

Figure 3. Storm as it appears on weather map in the northern hemisphere

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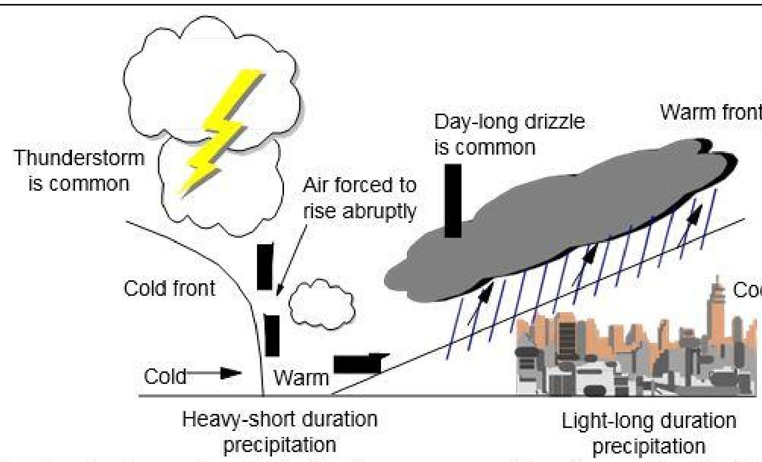

Figure 4. Cyclonic storms in mid-latitude; cross-section from A to B of Figure 3

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Hurricanes and Typhoons

Hurricanes, typhoons, or tropical cyclones develop over tropical oceans that have a surface-water temperature greater than 84°F. A hurricane has no trailing fronts, as the air is uniformly warm since the ocean surface from which it was spawned is uniformly warm. Hurricanes can drop tremendous amounts of moisture on an area in a relatively short time. Rainfall amounts of 14 to 20 inches in less than 24 hours are common in well-developed hurricanes, where winds are often sustained in excess of 75 miles/hour.

<u>Characteristics of Rainfall Events</u>

The characteristics of precipitation that are important to highway drainage are the intensity (rate of rainfall); the duration; the time distribution of rainfall; the storm shape, size, and movement; and the frequency.

Intensity is defined as the time rate of rainfall depth and is commonly given in the units of inches per hour. All precipitation is measured as the vertical depth of water (or water equivalent in the case of snow) that would accumulate on a flat level surface if all the precipitation remained where it fell. A variety of rain gauges have been devised to measure precipitation. All first-order weather stations use gauges that provide nearly continuous records of accumulated rainfall with time. These data are typically reported in either tabular form or as cumulative mass rainfall curves (see Figure 5).

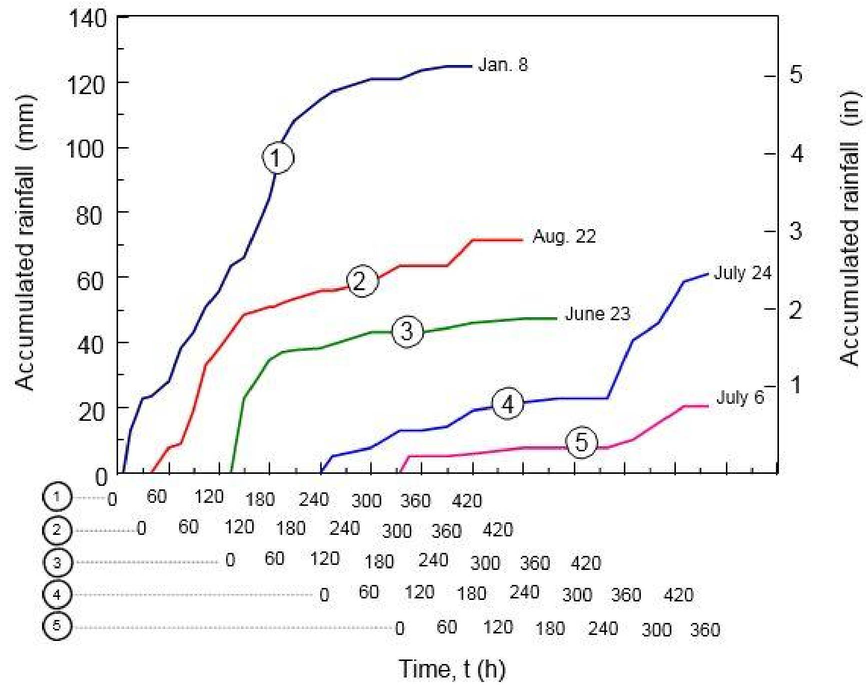

Figure 5. Typical mass rainfall curves: Kickapoo Station recording gauge

Figure 5. Typical mass rainfall curves: Kickapoo Station recording gauge

In any given storm, the instantaneous intensity is the slope of the mass rainfall curve at a particular time. For hydrologic analysis, it is desirable to divide the storm into convenient time increments and to determine the average intensity over each of the selected periods. These results are then plotted as rainfall hyetographs, two examples of which are shown in Figure 6 for the Kickapoo Station.

While the above illustrations use a 1-hour time increment to determine the average intensity, any time increment compatible with the time scale of the hydrologic event to be analyzed can be used. Figure 6 shows the irregular and complex nature of different storms measured at the same station.

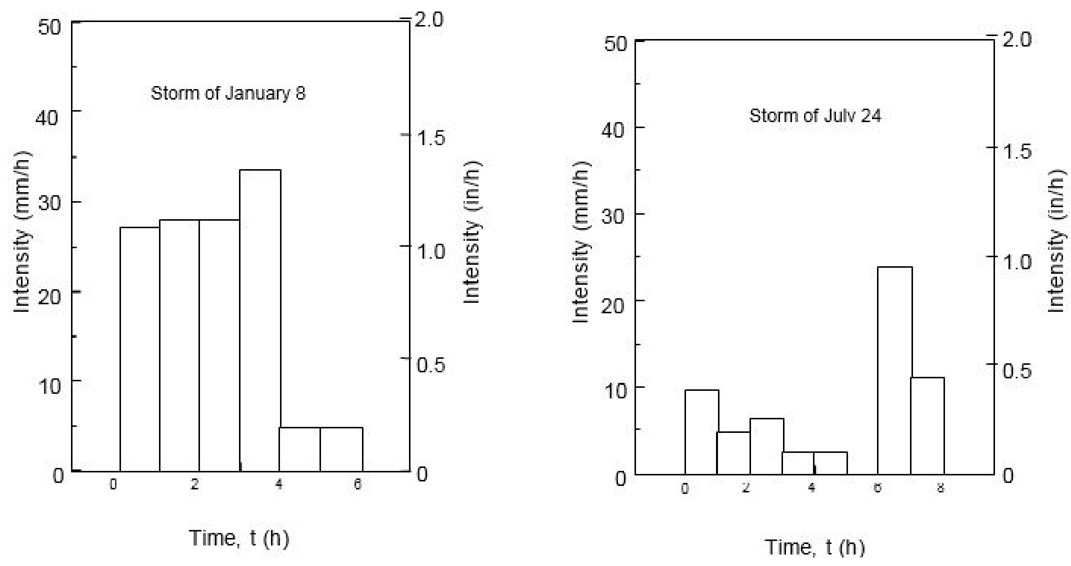

Figure 6. Rainfall hyetographs for Kickapoo Station

Figure 6. Rainfall hyetographs for Kickapoo Station

In spite of this complexity, intensity is the most important of the rainfall characteristics. All other factors being equal, the more intense the rainfall, the larger will be the discharge rate from a given watershed. Intensities can vary from misting conditions where a trace of precipitation may fall to cloudbursts. Figure 7 summarizes some of the maximum observed rainfalls in the United States. The events given in Figure 7 are depth-duration values at a point and can only be interpreted for average intensities over the reported durations. Still some of these storms were very intense, with average intensities on the order of 6 to 20 inches/hour for the shorter durations (<1 hour) and from 2 to 10 inches/hour for the longer durations (>1 hour). Since these are only averages, it is probable that intensities in excess of these values occurred during the various storms.

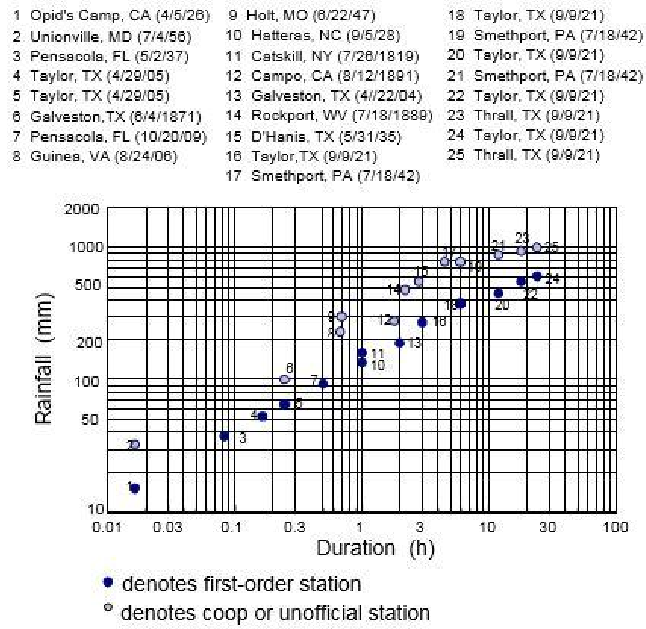

Figure 7. Maximum observed rainfalls (U.S.) from USWB, 1947; ECAFE, U.N., 1967

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The storm duration or time of rainfall can be determined from either Figure 5 or 6. In the case of Figure 5, the duration is the time from the beginning of rainfall to the point where the mass curve becomes horizontal, indicating no further accumulation of precipitation. In Figure 6, the storm duration is simply the width (time base) of the hyetograph. The most direct effect of storm duration is on the volume of surface runoff, with longer storms producing more runoff than shorter duration storms of the same intensity.

The time distribution of the rainfall is normally given in the form of intensity hyetographs similar to those shown in Figure 6. This time variation directly determines the corresponding distribution of the surface runoff. As illustrated in Figure 8, high intensity rainfall at the beginning of a storm, such as the January 8 storm in Figure 6, will usually result in a rapid rise in the runoff, followed by a long recession of the flow. Conversely, if the more intense rainfall occurs toward the end of the duration, as in the July 24 storm of Figure 6, the time to peak will be longer, followed by a rapidly falling recession.

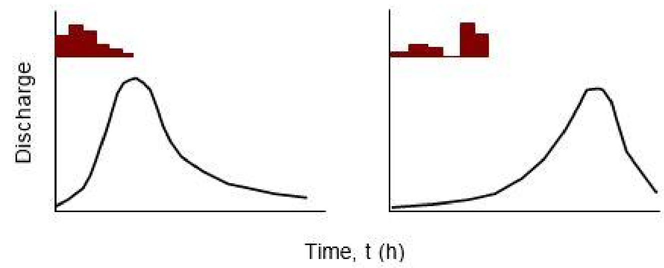

Figure 8. Effect of time variation of rainfall intensity on the surface runoff

Figure 8. Effect of time variation of rainfall intensity on the surface runoff

Storm pattern, areal extent, and movement are normally determined by the type of storm. For example, storms associated with cold fronts (thunderstorms) tend to be more localized, faster moving, and of shorter duration, whereas warm fronts tend to produce slowly moving storms of broad areal extent and longer durations. All three of these factors determine the areal extent of precipitation and how large a portion of the drainage area contributes over time to the surface runoff. As illustrated in Figure 9, a small localized storm of a given intensity and duration, occurring over a part of the drainage area, will result in much less runoff than if the same storm covered the entire watershed.

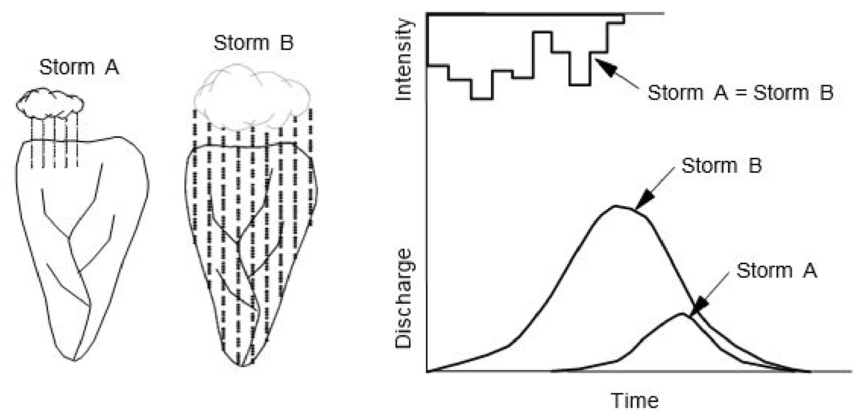

Figure 9. Effect of storm size on runoff hydrograph

Figure 9. Effect of storm size on runoff hydrograph

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 result in the peak flow occurring very quickly and a rapid passage of the flood. If the same storm occurred in a remote part of the basin, the runoff at the outlet due to the storm would be longer and the peak flow lower due to storage in the channel.

Storm movement has a similar effect on the runoff distribution particularly if the basin is long and narrow. Figure 10 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 in time.

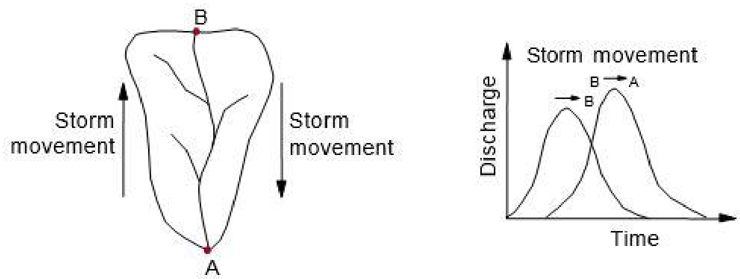

Figure 10. Effect of storm movement on runoff hydrograph

Figure 10. Effect of storm movement on runoff hydrograph

Frequency is also an important characteristic because it establishes the frame of reference for how often precipitation with given characteristics is likely to occur. From the standpoint of highway design, a primary concern is with the frequency of occurrence of the resulting surface runoff, and in particular, the frequency of the peak discharge. While the designer is cautioned about assuming that a storm of a given frequency always produces a flood of the same frequency, there are a number of analytical techniques that are based on this assumption, particularly for ungauged watersheds. Some of the factors that determine how closely the frequencies of precipitation and peak discharge correlate with one another are discussed further below.

Precipitation is not easily characterized although there have been many attempts to do so. References and data sources are available that provide general information on the character of precipitation at specified geographic locations. It is important, however, to understand the highly variable and erratic nature of precipitation. Highway designers should become familiar with the different types of storms and the characteristics of precipitation that are indigenous to their regions of concern. They should also understand the seasonal variations that are prevalent in many areas. In addition, it is very beneficial to study reports that have been prepared on historic storms and floods in a region. Such reports can provide information on past storms and the consequences that they may have had on drainage structures.

Intensity-Duration-Frequency Curves

Three rainfall characteristics are important and interact with each other in many hydrologic design problems. Rainfall intensity, duration, and frequency were defined and discussed in the previous section. For use in design, the three characteristics are combined, usually graphically into the intensity-duration-frequency (IDF) curve. Rainfall intensity is graphed as the ordinate and duration as the abscissa. One curve of intensity versus duration is given for each exceedence frequency. IDF curves are location dependent. IDF curves have been developed for many jurisdictions throughout the United States through frequency analysis of rainfall events. Figure 10A illustrates an example IDF curve. To interpret an IDF curve, find the rainfall duration along the X-axis, go vertically up the graph until reaching the proper return period, then go horizontally to the left and read the intensity from the Y-axis. Regional IDF curves are available in some highway agency and local drainage manuals. If the IDF curves are not available, the designer needs to develop them on a project by project basis.

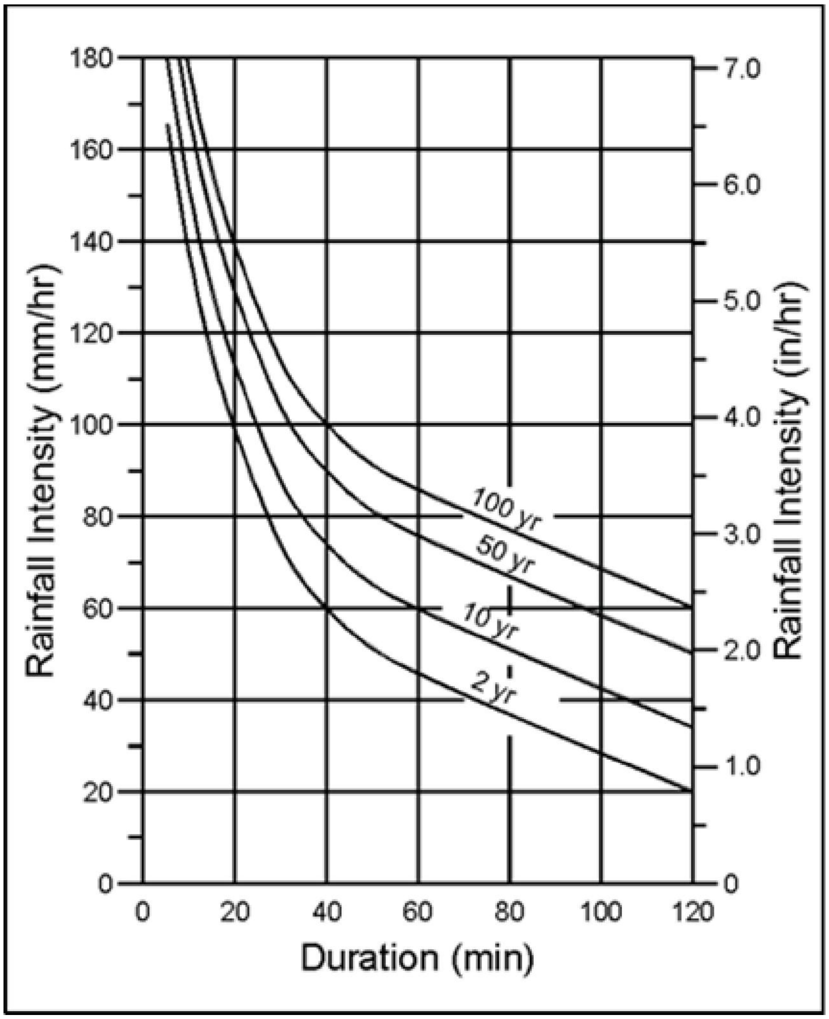

Figure 10A. Sample IDF curves

IDF curves are plotted on a log-log scale and have a characteristic shape. Typically, the IDF curve for a specific exceedence frequency is characteristically curved for small durations usually 2 hours and shorter, and straight for the longer durations.

Volume-duration-frequency (VDF) curves are sometimes provided in hydrologic design manuals. The VDF curve is similar to the IDF curve except the depth of rainfall is graphed as the ordinate. The IDF curve is preferred because many design methods use rainfall intensities rather than rainfall depths.

HYDROLOGIC ABSTRACTIONS

The collective term given to the various processes that act to remove water from the incoming precipitation before it leaves the watershed as runoff is abstractions. These processes are evaporation, transpiration, interception, infiltration, depression storage, and detention storage. The most important abstractions in determining the surface runoff from a given precipitation event are infiltration, depression storage, and detention storage.

Evaporation

Evaporation is the process by which water from the land and water surfaces is converted into water vapor and returned to the atmosphere. It occurs continually whenever the air is unsaturated and temperatures are sufficiently high. Air is 'saturated' when it holds its maximum capacity of moisture at the given temperature. Saturated air has a relative humidity of 100 percent. Evaporation plays a major role in determining the long-term water balance in a watershed. However, evaporation is usually insignificant in small watersheds for single storm events and can be discounted when calculating the discharge from a given rainfall event.

Transpiration

Transpiration is the physical removal of water from the watershed by the life actions associated with the growth of vegetation. In the process of respiration, green plants consume water from the ground and transpire water vapor to the air through their foliage. As was the case with evaporation, this abstraction is only significant when taken over a long period of time and has minimal effect upon the runoff resulting from a single storm event for a watershed.

Interception

Interception is the removal of water that wets and adheres to objects above ground such as buildings, trees, and vegetation. This water is subsequently removed from the surface through evaporation. Interception can be as high as 0.08 inches during a single rainfall event, but usually is nearer 0.02 inches. The quantity of water removed through interception is usually not significant for an isolated storm, but, when added over a period of time, it can be significant.

It is thought that as much as 25 percent of the total annual precipitation for certain heavily forested areas of the Pacific Northwest of the United States is lost through interception during the course of a year.

Infiltration

Infiltration is the flow of water into the ground by percolation through the earth's surface. The process of infiltration is complex and depends upon many factors such as soil type, vegetal cover, antecedent moisture conditions or the amount of time elapsed since the last precipitation event, precipitation intensity, and temperature. Infiltration is usually the single most important abstraction in determining the response of a watershed to a given rainfall event. As important as it is, no generally acceptable model has been developed to accurately predict infiltration rates or total infiltration volumes for a given watershed.

Depression Storage

Depression storage is the term applied to water that is lost because it becomes trapped in the numerous small depressions that are characteristic of any natural surface. When water temporarily accumulates in a low point with no possibility for escape as runoff, the accumulation is referred to as depression storage. The amount of water that is lost due to depression storage varies greatly with the land use. A paved surface will not detain as much water as a recently furrowed field. The relative importance of depression storage in determining the runoff from a given storm depends on the amount and intensity of precipitation in the storm. Typical values for depression storage range from 0.04 to 0.3 inches with some values as high as 0.6 inches per event. As with evaporation and transpiration, depression storage is generally not directly calculated in highway design.

Detention Storage

Detention storage is water that is temporarily stored in the depth of water necessary for overland flow to occur. The volume of water in motion over the land constitutes the detention storage. The amount of water that will be stored is dependent on a number of factors such as land use, vegetal cover, slope, and rainfall intensity. Typical values for detention storage range from 0.08 to 0.4 inches, but values as high as 2 inches have been reported.

Total Abstraction Methods

While the volumes of the individual abstractions may be small, their sum can be hydrologically significant. Therefore hydrologic methods commonly lump all abstractions together and compute a single value. The SCS curve number method lumps all abstractions together, with the volume equal to the difference between the volumes of rainfall and runoff. The phi-index method assumes a constant rate of abstraction over the duration of the storm. These total abstraction methods simplify the calculation of storm runoff rates.

CHARACTERISTICS OF RUNOFF

Water that has not been abstracted from the incoming precipitation leaves the watershed as surface runoff. While runoff occurs in several stages, the flow that becomes channelized is the main consideration to highway stream crossing design since it influences the size of a given drainage structure. The rate of flow or runoff at a given instant, in terms of volume per unit of time, is called discharge. Some characteristics of runoff that are important to drainage design are: (1) the peak discharge or peak rate of flow; (2) the discharge variation with time (hydrograph); (3) the stage-discharge relationship; (4) the total volume of runoff; and (5) the frequency with which discharges of specified magnitudes are likely to be equaled or exceeded (probability of exceedence).

Peak Discharge

The peak discharge, often called peak flow, is the maximum rate of runoff passing a given point during or after a rainfall event. Highway designers are interested in peak flows for storms in an area because it is the discharge that a given structure must be sized to handle. Of course, the peak flow varies for each different storm, and it becomes the designer's responsibility to size a given structure for the magnitude of storm that is determined to present an acceptable risk in a given situation. Peak flow rates can be affected by many factors in a watershed, including rainfall, basin size, and the physiographic features.

Time Variation (Hydrograph)

The flow in a stream varies from time to time, particularly during and in response to storm events. As precipitation falls and moves through the watershed, water levels in streams rise and may continue to do so (depending on position of the storm over the watershed) after the precipitation has ceased. The response of an affected stream through time during a storm event is characterized by the flood hydrograph. This response can be pictured by graphing the flow in a stream relative to time. The primary features of a typical hydrograph are illustrated in Figure 11 and include the rising and falling limbs, the peak flow, the time to peak, and the time base of the hydrograph. There are several types of hydrographs, such as flow per unit area and stage hydrographs, but all display the same typical variation through time.

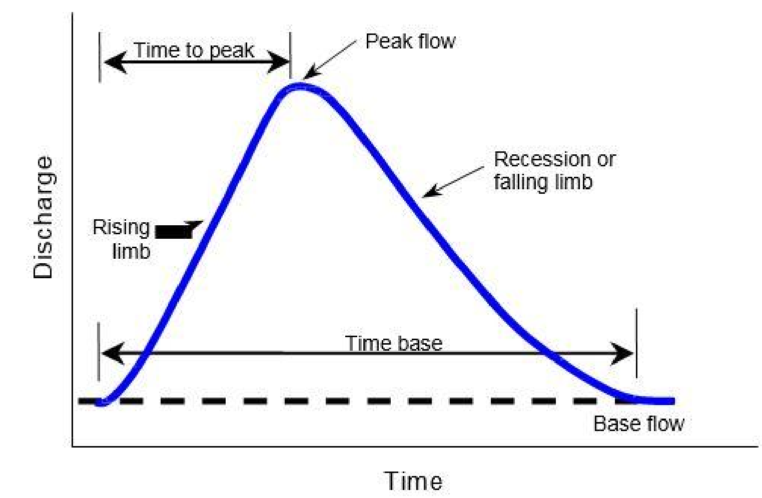

Figure 11. Elements of a flood hydrograph

Figure 11. Elements of a flood hydrograph

Stage-Discharge

The stage of a river is the elevation of the water surface above some arbitrary datum. The datum can be mean sea level but can also be set slightly below the point of zero flow in the given stream. The stage of a river is directly related to the discharge, which is the quantity of water passing a given point (see Figure 12). As the discharge increases, the stage rises and as the discharge decreases, the stage falls. Generally, discharge is related to stage at a particular point by using a variety of techniques and instrumentation to obtain field measurements of these (and related) parameters.

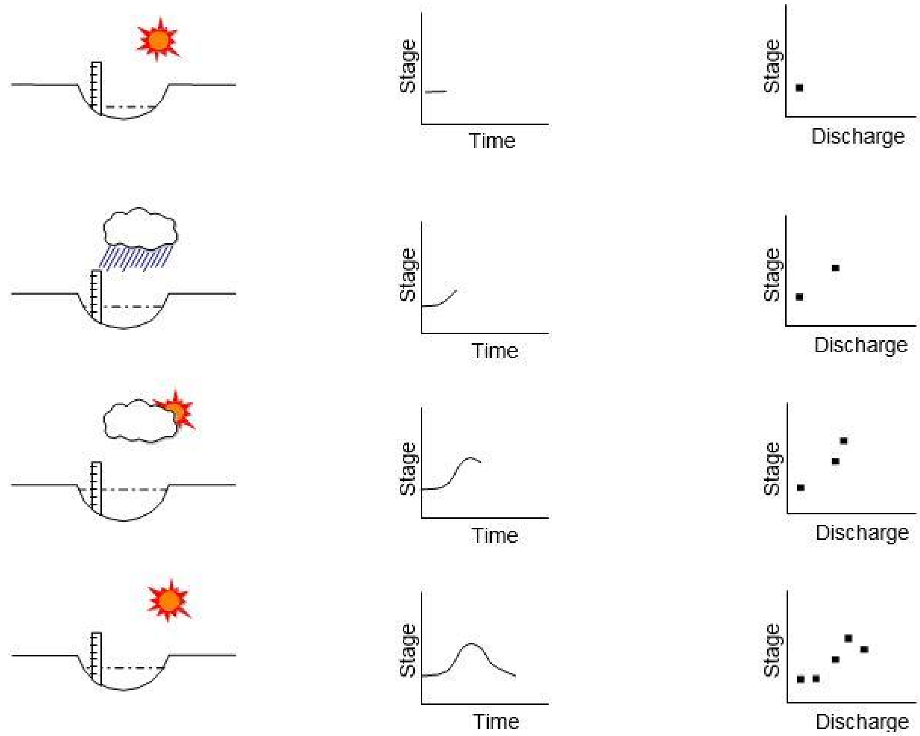

Figure 12. Relation between stage and discharge

Total Volume

The total volume of runoff from a given flood is of primary importance to the design of storage facilities and flood control works. Flood volume is not normally a consideration in the design of highway drainage crossing structures. However, flood volume is used in various analyses for other design parameters. Flood volume is most easily determined as the area under the flood hydrograph (Figure 11) and is commonly measured in units of cubic feet or acre-feet. The equivalent depth of net rain over the watershed is determined by dividing the volume of runoff by the watershed area.

Frequency

The exceedence frequency is the relative number of times a flood of a given magnitude can be expected to occur on the average over a long period of time. It is usually expressed as a ratio or a percentage. By its definition, frequency is a probabilistic concept and is the probability that a flood of a given magnitude may be equaled or exceeded in a specified period of time, usually 1 year. Exceedence frequency is an important design parameter in that it identifies the level of risk during a specified time interval acceptable for the design of a highway structure.

Return Period

Return period is a term commonly used in hydrology. In very simple terms, it is the average time interval between the occurrence of storms or floods of a given magnitude, but is often misinterpreted. The exceedence probability (p) and return period (T) are related by:

$$T = 1/p \qquad \text{(Equation 1)}$$

For example, a flood with an exceedence probability of 0.01 in any one year is referred to as the 100-year flood. The use of the term return period is sometimes discouraged because some people interpret it to mean that there will be exactly T years between occurrences of the event. Two 100-year floods can occur in successive years or they may occur 500 years apart. The return period is only the long-term average number of years between occurrences, and for most flood events, the meaning of long-term in millennia, not decades or centuries.

EFFECTS OF BASIN CHARACTERISTICS ON RUNOFF

The spatial and temporal variations of precipitation and the concurrent variations of the individual abstraction processes determine the characteristics of the runoff from a given storm. These are not the only factors involved, however. Once the local abstractions have been satisfied for a small area of the watershed, water begins to flow overland and eventually into a natural drainage channel such as a gully or a stream valley. At this point, the hydraulics of the natural drainage channels have a large influence on the character of the total runoff from the watershed.

A few of the many factors that determine the hydraulic character of the natural drainage system are drainage area, slope, hydraulic roughness, natural and channel storage, drainage density, channel length, antecedent moisture conditions, urbanization, and other factors. The effect that each of these factors has on the important characteristics of runoff is often difficult to quantify. The following paragraphs discuss some of the factors that affect the hydraulic character of a given drainage system.

Drainage Area

Drainage area is the most important watershed characteristic that affects runoff. The larger the contributing drainage area, the larger will be the flood runoff (see Figure 13a). Regardless of the method utilized to evaluate flood flows, peak flow is directly related to the drainage area.

Slope

Steep slopes tend to result in rapid runoff responses to local rainfall excess and consequently higher peak discharges (see Figure 13b). The runoff is quickly removed from the watershed, so the hydrograph is short with a high peak. The stage-discharge relationship is highly dependent upon the local characteristics of the cross-section of the drainage channel and, if the slope is sufficiently steep, supercritical flow may prevail. The total volume of runoff is also affected by slope. If the slope is very flat, the rainfall will not be removed as rapidly. The process of infiltration will have more time to affect the rainfall excess, thereby increasing the abstractions and resulting in a reduction of the total volume of rainfall that appears directly as runoff.

Slope is very important in how quickly a drainage channel will convey water and, therefore, it influences the sensitivity of a watershed to precipitation events of various time durations.

Watersheds with steep slopes will rapidly convey incoming rainfall and, if the rainfall is convective (characterized by high intensity and relatively short duration), the watershed will respond very quickly with the peak flow occurring shortly after the onset of precipitation. If these convective storms occur with a given frequency, the resulting runoff can be expected to occur with a similar frequency. On the other hand, for a watershed with a flat slope, the response to the same storm will not be as rapid and, depending on a number of other factors, the frequency of the resulting discharge may be dissimilar to the storm frequency.

Hydraulic Roughness

Hydraulic roughness is a composite of the physical characteristics that influence the depth and speed of water flowing across the surface, whether natural or channelized. It affects both the time response of a drainage channel and the channel storage characteristics. Hydraulic roughness has a marked effect on the characteristics of the runoff resulting from a given storm. The peak rate of discharge is usually inversely proportional to hydraulic roughness (i.e., the lower the roughness, the higher the peak discharge). Roughness affects the runoff hydrograph in a manner opposite of slope. The lower the roughness, the more peaked and shorter in time the resulting hydrograph will be for a given storm (see Figure 13c).

The stage-discharge relationship for a given section of drainage channel is also dependent on roughness (assuming normal flow conditions and the absence of artificial controls). A higher roughness results in a higher stage for a given discharge.

The total volume of runoff is virtually independent of hydraulic roughness. An indirect relationship does exist in that higher roughness slows the watershed response and allows some of the abstraction processes more time to affect runoff. Roughness also has an influence on the frequency of discharges of certain magnitudes by affecting the response time of the watershed to precipitation events of specified frequencies.

Storage

It is common for a watershed to have natural or manmade storage that greatly affects the response to a given precipitation event. Common features that contribute to storage within a watershed are lakes, marshes, heavily vegetated overbank areas, natural or manmade constrictions in the drainage channel that cause backwater, and the storage in the floodplains of large, wide rivers. Storage can have a significant effect in reducing the peak rate of discharge, although this reduction is not necessarily universal. There have been some instances where artificial storage redistributes the discharges very radically, resulting in higher peak discharges than would have occurred had the storage not been added. As shown in Figure 13d, storage generally spreads the hydrograph out in time, delays the time to peak, and alters the shape of the resulting hydrograph from a given storm.

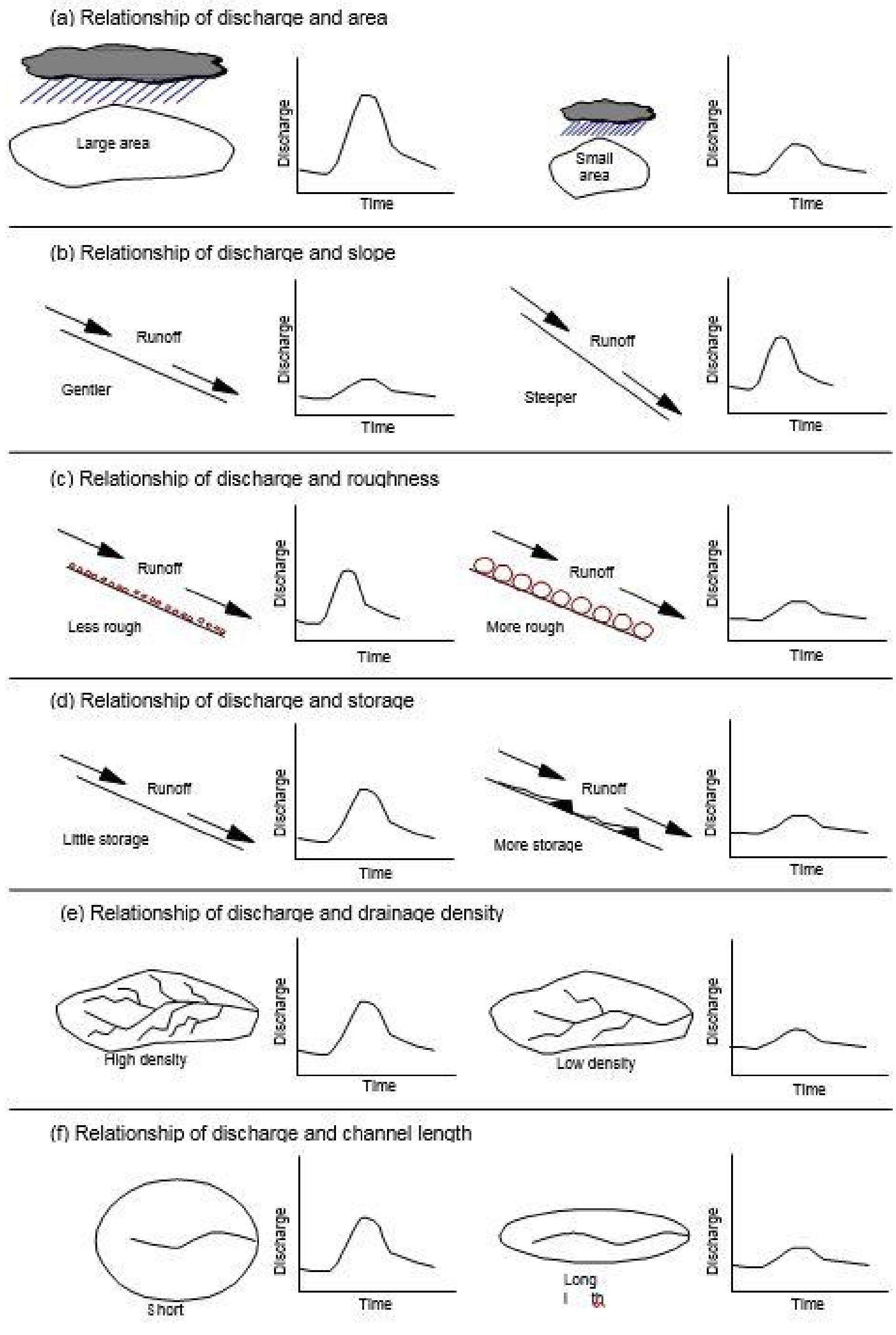

Figure 13. Effects of basin characteristics on the flood hydrograph

Figure 13. Effects of basin characteristics on the flood hydrograph

The stage-discharge relationship also can be influenced by storage within a watershed. If the section of a drainage channel is upstream of the storage and within the zone of backwater, the stage for a given discharge will be higher than if the storage were not present. If the section is downstream of the storage, the stage-discharge relationship may or may not be affected, depending upon the presence of channel controls.

The total volume of runoff is not directly influenced by the presence of storage. Storage will redistribute the volume over time but will not directly change the volume. By redistributing the runoff over time, storage may allow other abstraction processes to decrease the runoff (as was the case with slope and roughness).

Changes in storage have a definite effect upon the frequency of discharges of given magnitudes. Storage tends to dampen the response of a watershed to very short events and to accentuate the response to very long events. This alters the relationship between frequency of precipitation and the frequency of the resultant runoff.

Drainage Density

Drainage density can be defined as the ratio between the number of well-defined drainage channels and the total drainage area in a given watershed. Drainage density is usually assumed to equal the total length of continuously flowing streams divided by the drainage area. It is determined by the topography and the geography of the watershed.

Drainage density has a strong influence on both the spatial and temporal response of a watershed to a given precipitation event. If a watershed is well covered by a pattern of interconnected drainage channels, and the overland flow time is relatively short, the watershed will respond more rapidly than if it were sparsely drained and overland flow time was relatively long. The mean velocity of runoff is normally lower for overland flow than it is for flow in a well- defined natural channel. High drainage densities are associated with increased response of a watershed leading to higher peak discharges and shorter hydrographs for a given precipitation event (see Figure 13e).

Drainage density has a minimal effect on the stage-discharge relationship for a particular section of drainage channel. It does, however, have an effect on the total volume of runoff since some of the abstraction processes are directly related to how long the rainfall excess exists as overland flow. Therefore, the lower the density of drainage, the lower will be the volume of runoff from a given precipitation event.

Changes in drainage density such as with channel improvements in urbanizing watersheds can have an effect on the frequency of discharges of given magnitudes. By strongly influencing the response of a given watershed to any precipitation input, the drainage density determines in part the frequency of the response. The higher the drainage density, the more closely related the resultant runoff frequency would be to that of the corresponding precipitation event.

Channel Length

Channel length is an important watershed characteristic. The longer the channel, the more time it takes for water to be conveyed from the headwaters of the watershed to the outlet. Consequently, if all other factors are the same, a watershed with a longer channel length will usually have a slower response to a given precipitation input than a watershed with a shorter channel length. As the hydrograph travels along a channel, it is attenuated and extended in time due to the effects of channel storage and hydraulic roughness. As shown in Figure 13f, longer channels result in lower peak discharges and longer hydrographs.

The frequency of discharges of given magnitudes will also be influenced by channel length. As was the case for drainage density, channel length is an important parameter in determining the response time of a watershed to precipitation events of given frequency. However, channel length may not remain constant with discharges of various magnitudes. In the case of a wide floodplain where the main channel meanders appreciably, it is not unusual for the higher flood discharges to overtop the banks and essentially flow in a straight line in the floodplain, thus reducing the effective channel length.

The stage-discharge relationship and the total volume of runoff are practically independent of channel length. Volume, however, will be redistributed in time, similar in effect to storage but less pronounced.

Antecedent Moisture Conditions

As noted earlier, antecedent moisture conditions, which are the soil moisture conditions of the watershed at the beginning of a storm, affect the volume of runoff generated by a particular storm event. Runoff volumes are related directly to antecedent moisture levels. The smaller the moisture in the ground at the beginning of precipitation, the lower will be the runoff. Conversely, the larger the moisture content of the soil, the higher the runoff attributable to a particular storm.

Urbanization

As a watershed undergoes urbanization, the peak discharge typically increases and the hydrograph becomes shorter and rises more quickly. This is due mostly to the improved hydraulic efficiency of an urbanized area. In its natural state, a watershed will have developed a natural system of conveyances consisting of gullies, streams, ponds, marshes, etc., all in equilibrium with the naturally existing vegetation and physical watershed characteristics. As an area develops, typical changes made to the watershed include: (l) removal of existing vegetation and replacement with impervious pavement or buildings, (2) improvement to natural watercourses by channelization, and (3) augmentation of the natural drainage system by storm sewers and open channels. These changes tend to decrease depression storage, infiltration rates, and travel time. Consequently, peak discharges increase, with the time base of hydrographs becoming shorter and the rising limb rising more quickly.

Other Factors

There can be other factors within the watershed that determine the characteristics of runoff, including the extent and type of vegetation, the presence of channel modifications, and flood control structures. These factors modify the runoff by either augmenting or negating some of the basin characteristics described above. It is important to recognize that all of the factors discussed exist concurrently within a given watershed, and their combined effects are very difficult to model and quantify.

ILLUSTRATION OF THE RUNOFF PROCESS

Several key hydrologic abstractions were previously described in general terms. The method by which the runoff process can be analyzed and the results used to obtain a hydrograph are illustrated in this section. Figures 14a through 14f show the development of the flood hydrograph from a typical rainfall event.

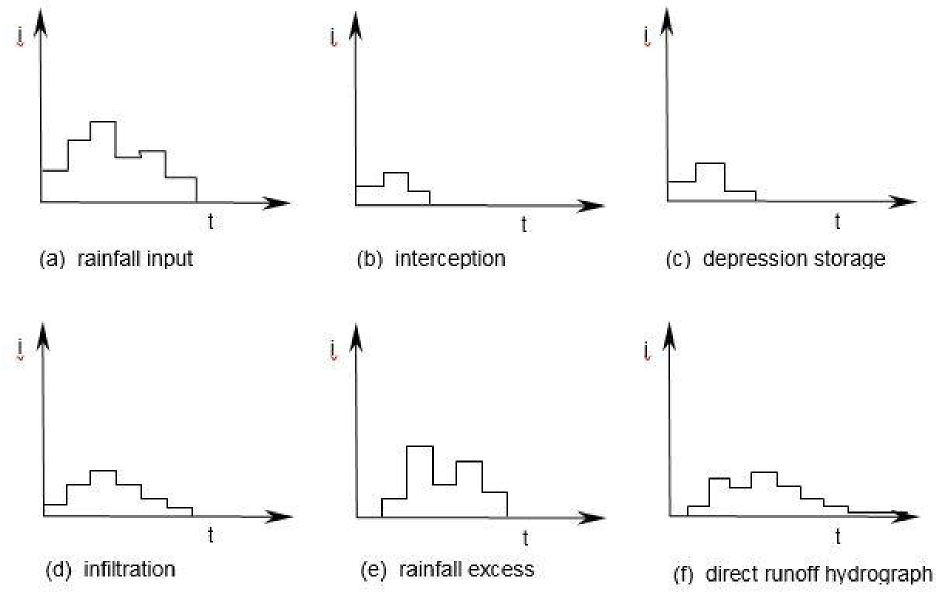

Figure 14. The runoff process

Figure 14. The runoff process

<u>Rainfall Input</u>

Rainfall is randomly distributed in time and space, and the rainfall experienced at a particular point can vary greatly. For simplification, consider the rainfall at only one point in space and assume that the variation of rainfall intensity with time can be approximated by discrete time periods of constant intensity. This simplification is illustrated in Figure 14a. The specific values of intensity and time are not important for this illustrative example since it shows only relative magnitudes and relationships. The rainfall, so arranged, is the input to the runoff process, and from this, the various abstractions must be deleted.

<u>Interception</u>

Figure 14b illustrates the relative magnitude and time relationship for interception. When the rainfall first begins, the foliage and other intercepting surfaces are dry. As water adheres to these surfaces, a large portion of the initial rainfall is abstracted. This occurs in a relatively short period of time and, once the initial wetting is complete, the interception losses quickly decrease to a lower, nearly constant value. The rainfall that has not been intercepted falls to the ground surface to continue in the runoff process.

<u>Depression Storage</u>

Figure 14c illustrates the relative magnitude of depression storage with time. Only the water that is in excess of that necessary to supply the interception is available for depression storage. This is the reason that the depression-storage curve begins at zero. The amount of water that goes into depression storage varies with differing land uses and soil types, but the curve shown is representative. The smallest depressions are filled first and then the larger depressions are filled as time and the rainfall supply continue. The slope of the depression-storage curve depends on the distribution of storage volume with respect to the size of depressions. There are usually many small depressions that fill rapidly and account for most of the total volume of depression storage. This results in a rapid peaking of storage with time as shown in Figure 14c. The large depressions take longer to fill and the curve gradually approaches zero when all of the depression storage has been filled. When the rainfall input equals the interception, infiltration, and depression storage, there is no surface runoff.

Infiltration

Infiltration is a complex process, and the rate of infiltration at any point in time depends on many factors. The important point to be illustrated in Figure 14d is the time dependence of the infiltration curve. It is also important to note the behavior of the infiltration curve after the period of relatively low rainfall intensity near the middle of the storm event. The infiltration rate increases over what it was prior to the period of lower intensity because the upper layers of the soil are drained at a rate that is independent of the rainfall intensity. Most deterministic models, including the phi-index method for estimating infiltration, do not model the infiltration process accurately in this respect.

Rainfall Excess

Only after interception, depression storage, and infiltration have been satisfied is there an excess of water available to run off from the land surface. As previously defined, this is the rainfall excess and is illustrated in Figure 14e. Note how this rainfall excess differs with the actual rainfall input, Figure 14a.

The concept of excess rainfall is very important in hydrologic analyses. It is the amount of water available to run off after the initial abstractions and other losses have been satisfied. Except for the losses that may occur during overland and channelized flow, it determines the volume of water that flows past the outlet of a drainage basin. When multiplied by the drainage area, it should be very nearly equal to the volume under the direct runoff hydrograph. The rainfall excess has a direct effect on the outflow hydrograph. It influences the magnitude of the peak flow, the duration of the flood hydrograph, and the shape of the hydrograph.

Detention Storage

A volume of water is detained in temporary (detention) storage. This volume is proportional to the local rainfall excess and is dependent on a number of other factors as noted previously. Although all water in detention storage eventually leaves the basin, this requirement must be met before runoff can occur.

Local Runoff

Local runoff is actually the residual of the rainfall input after all abstractions have been satisfied. It is similar in shape to the excess rainfall (see Figure 14e), but is extended in time as the detention storage acts on the local runoff.

Outflow Hydrograph

Figure 14f illustrates the final outflow hydrograph from the watershed due to the local runoff hydrograph. This final hydrograph is the cumulative effect of all the modifying factors that act on the water as it flows through drainage channels as discussed above. The total volume of water contained under the direct runoff hydrograph of Figure 14f and the rainfall excess of Figure 14e are the same, although the position of the outflow hydrograph in time is modified due to the smoothing of the surface runoff and the channel processes.

The processes that have been discussed in the previous sections all act simultaneously to transform the incoming rainfall from that shown in Figure 14a to the corresponding outflow hydrograph of Figure 14f. This example serves to illustrate the runoff process for a small local area. If the watershed is of appreciable size or if the storm is large, areal and time variations and other factors add a new level of complexity to the problem.

TRAVEL TIME

The travel time of runoff is very important in hydrologic design. In the design of inlets and pipe drainage systems, travel times of surface runoff must be estimated. Some peak discharge methods use the time of concentration as input to obtain rainfall intensities from the intensity-duration-frequency curves. Hydrograph times-to-peak, which are in some cases computed from times of concentration, are used with hydrograph methods. Channel routing methods use computed travel times in routing hydrographs through channel reaches. Thus, estimating travel times are central to a variety of hydrologic design problems.

Time of Concentration

The time of concentration, which is denoted as tc, is defined as the time required for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point. Note that the hydraulically most distant point is a function of travel time, not distance, so the topographically most distant point may not be the hydraulically most distant point. A long but steep flow path with a high velocity may actually have a shorter travel time than a short but relatively flat flow path. There may be multiple paths to consider in determining the longest travel time. The designer must identify the flow path along which the longest travel time is likely to occur. Factors that affect the time of concentration are the length of flow, the slope of the flow path, and the roughness of the flow path. For flow at the upper reaches of a watershed, rainfall characteristics, most notably the intensity, may also influence the velocity of the runoff.

Various methods can be used to estimate the time of concentration of a watershed. When selecting a method to use in design, it is important to select a method that is appropriate for the flow path. Some estimation methods were designed and can be classified as “lumped” in that they were designed and calibrated to be used for an entire watershed; the SCS lag formula is an example of this method. These methods have tc as the dependent variable. Other methods are intended for one segment of the principal flow path and produce a flow velocity that can be used with the length of that segment of the flow path to compute the travel time on that segment. With this method, the time of concentration equals the sum of the travel times on each segment of the principal flow path.

In classifying these methods so that the proper method can be selected, it is useful to describe the segments of flow paths. Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually 0.8 to 1.2 inches or less. Concentrated flow is runoff that occurs in rills and swales and has depths on the order of 1.5 to 4.0 inches. Part of the principal flow path may include pipes or small streams. The travel time through these segments would be computed separately. Velocities in open channels are usually determined assuming bank-full depths.

Velocity Method

The velocity method (sometimes referred to as the segment method) can be used to estimate travel times for sheet flow, shallow concentrated flow, pipe flow, or channel flow. It is based on estimating the travel time from the length and velocity:

$T_t = L/60V$ (Equation 2)

where,
T_t = travel time, min
L = flow length, ft
V = flow velocity, ft/s.

The travel time is computed for the principal flow path. When the principal flow path consists of segments that have different slopes or land covers, the principal flow path should be divided into segments and Equation 2 used for each flow segment. The time of concentration is then the sum of travel times.

Velocity is a function of the type of flow (overland, sheet, rill and gully flow, channel flow, pipe flow), the roughness of the flow path, and the slope of the flow path. Some methods also include a rainfall index such as the 2-year, 24-hour rainfall depth. A number of methods have been developed for estimating the velocity.

*Sheet-Flow Travel Tim*e

Sheet flow is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. Typically flow depths will not exceed 2 in. Such flow occurs over relatively short distances, rarely more than about 100 ft, but most likely less than 80 ft. Sheet flow rates are commonly estimated using a version of the kinematic wave equation. The original form of the kinematic wave time of concentration is:

$$t_c = \frac{\alpha}{i^{0.4}} \left(\frac{nL}{\sqrt{S}}\right)^{0.6}$$

Equation 3

n = roughness coefficient (see Table 1)
L = flow length, ft
i = rainfall intensity, in/h, for a storm that has a return period T and duration of tc minutes
S = slope of the surface, ft/ft
α = unit conversion constant equal to 0.93 in English units.

Some hydrologic design methods, such as the rational equation, assume that the storm duration equals the time of concentration. Thus, the time of concentration is entered into the IDF curve to find the design intensity. However, for Equation 3, i depends on tc and tc is not initially known. Therefore, the computation of tc is an iterative process. An initial estimate of tc is assumed and used to obtain i from the intensity-duration-frequency curve for the locality. The tc is computed from Equation 3 and used to check the initial value of i. If they are not the same, the process is repeated until two successive tc estimates are the same.

Table 1. Manning's Roughness Coefficient (n) for Overland and Sheet Flow
(SCS, 1986; McCuen, 1989)

n	Surface Description
0.011	Smooth asphalt
0.012	Smooth concrete
0.013	Concrete lining
0.014	Good wood
0.014	Brick with cement mortar
0.015	Vitrified clay
0.015	Cast iron
0.024	Corrugated metal pipe
0.024	Cement rubble surface
0.050	Fallow (no residue)
	Cultivated soils
0.060	Residue cover δ 20%
0.170	Residue cover > 20%
0.130	Range (natural)
	Grass
0.150	Short grass prairie
0.240	Dense grasses
0.410	Bermuda grass
	Woods*
0.400	Light underbrush
0.800	Dense underbrush

*When selecting n for woody underbrush, consider cover to a height of about 30 mm (0.1 ft). This is the only part of the plant cover that will obstruct sheet flow.

To avoid the necessity to solve for tc iteratively, the SCS TR-55 (1986) uses the following variation of the kinematic wave equation:

$$t_c = \frac{\alpha}{P_2^{0.5}} \left(\frac{nL}{\sqrt{S}}\right)^{0.8}$$

Equation 4

P_2 = 2-year, 24-hour rainfall depth, in
α = unit conversion constant equal to 0.42 in English units.

The other variables are as previously defined. Equation 4 is based on an assumed IDF relationship. SCS TR-55 (1986) recommends an upper limit of L = 100 ft for using this equation.

Shallow Concentrated Flow

After short distances, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using an empirical relationship between the velocity and the slope:

$V = \alpha\, k\, S^{0.5}$ Equation 5

where,

V = velocity, ft/s
S = slope, ft/ft
k = dimensionless function of land cover (see Table 2)
α = unit conversion constant equal to 33 in English units.

Table 2. Intercept Coefficients for Velocity vs. Slope Relationship (McCuen, 1989)

k	Land Cover/Flow Regime
0.076	Forest with heavy ground litter; hay meadow (overland flow)
0.152	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)
0.213	Short grass pasture (overland flow)
0.274	Cultivated straight row (overland flow)
0.305	Nearly bare and untilled (overland flow); alluvial fans in western mountain regions
0.457	Grassed waterway (shallow concentrated flow)
0.491	Unpaved (shallow concentrated flow)
0.619	Paved area (shallow concentrated flow); small upland gullies

Pipe and Channel Flow

Flow in gullies empties into channels or pipes. In many cases, the transition between shallow concentrated flow and open channels may be assumed to occur where either the blue-line stream is depicted on USGS quadrangle sheets (scale equals 1:24000) or when the channel is visible on aerial photographs. Channel lengths may be measured directly from the map or scale photograph. However, depending on the scale of the map and the sinuosity of the channel, a map-derived channel length may be an underestimate. Pipe lengths should be taken from as- built drawings for existing systems and design plans for future systems.

Cross-section information (i.e., depth-area and roughness) can be obtained for any channel reach in the watershed. Manning's equation can be used to estimate average flow velocities in pipes and open channels:

$V = (1.49/n)\ R^{2/3}\ S^{1/2}$

V = velocity, ft/s
n = Manning's roughness coefficient
R = hydraulic radius, ft
S = slope, ft/ft

The hydraulic radius equals the cross-sectional area divided by the wetted perimeter. For a circular pipe flowing full, the hydraulic radius equals one-fourth of the diameter: R = D/4. For flow in a wide rectangular channel, the hydraulic radius is approximately equal to the depth of flow (d): R = d.

Example 1: Estimating Time of Concentration with the Velocity Method. Two watershed conditions are indicated, pre- and post-development, and summarized in Table 3. In the pre-development condition, the 4-acre drainage area is primarily forested, with a natural channel having a good stand of high grass. In the post-development condition, the channel has been eliminated and replaced with a 15-inch diameter pipe. The solution is as follows.

Table 3. Characteristics of Principal Flow Path for Example 1

Watershed Condition	Flow Segment	Length (ft)	Slope (ft/ft)	Type of Flow
Existing	1	100	0.010	Overland (forest)
	2	300	0.008	Grassed waterway
	3	480	0.008	Roadside channel (high grass, good stand)
Developed	1	50	0.010	Overland (short grass)
	2	50	0.010	Paved
	3	300	0.008	Grassed waterway
	4	420	0.009	Pipe – concrete (15 inch)

For the existing condition, the velocities of flow for the overland and grassed waterway segments can be obtained with Equation 5 and Table 2. For the slopes given in Table 3, the velocities for the first two segments are:

$V_1 = \alpha\ k\ S^{0.5} \quad = 33 * 0.076 * (0.010)^{0.5} = 0.25$ ft/sec

$V_2 = \alpha\ k\ S^{0.5} \quad = 33 * 0.457 * (0.008)^{0.5} = 1.35$ ft/sec

For the roadside channel, the velocity can be estimated using Manning's equation; a value for Manning's n of 0.15 is obtained from Table 1 and a hydraulic radius of 1.0 ft is estimated using conditions at the site:

$V_3 = (1.49/0.15)\ (1.0)^{2/3}\ (0.008)^{1/2} = 0.89$ ft/sec

Thus the time of concentration can be computed by summing all of the components:

t_c = (100/0.25) + (300/1.35) + (480/0.89) = 400 + 222 + 539 = 1161 seconds ~ 19 minutes
For the post-development conditions, the flow velocities for the first three segments can be determined with Equation 5. For the slopes given in Table 2.3, the velocities are:

$V_1 = \alpha k S^{0.5} = 33 * 0.213 * (0.010)^{0.5} = 0.70$ ft/sec

$V_2 = \alpha k S^{0.5} = 33 * 0.619 * (0.010)^{0.5} = 2.04$ ft/sec

$V_3 = \alpha k S^{0.5} = 33 * 0.457 * (0.008)^{0.5} = 1.35$ ft/sec

Assuming Manning's coefficient equals 0.011 for the concrete pipe and R = D/4, the velocity is:

$V_4 = (1.49/0.011)\ (0.31)^{2/3}\ (0.009)^{1/2} = 5.89$ ft/sec

A slope of 0.009 ft/ft is used since the meandering roadside channel was replaced with a pipe, which resulted in a shorter length of travel and, therefore, a steeper slope. Thus the time of concentration is:

t_c = (50/0.70) + (50/2.04) + (300/1.35) + (420/5.89) = 71 + 25 + 222 + 71 = 389 seconds ~ 6 minutes

Thus the land development decreased the time of concentration from 19 minutes to 6 minutes.

Example 2: Iterative Calculations Using the Kinematic Sheet Flow Equation. Consider the case of overland flow on short grass (n = 0.15) at a slope of 0.005 ft/ft. Assume the flow length is 150 ft. Equation 3 is:

$t_c = (\alpha/i^{0.4})*((nL/(S^{0.5}))^{0.6}$

$t_c = (0.93/4.7^{0.4})*((0.15*150/(0.005^{0.5}))^{0.6} = 16$

The value of i is obtained from an IDF curve for the locality of the project. For this example, the IDF curve of Baltimore is used (see Figure 15), and the problem assumes that a 5-year return period is specified. An initial t_c of 10 minutes will be used to obtain the intensity from Figure 15. The initial intensity is 4.7 inches per hour. Using the above equation gives a t_c of 16 minutes. Since this differs from the assumed tc of 10 minutes, a second iteration is necessary.

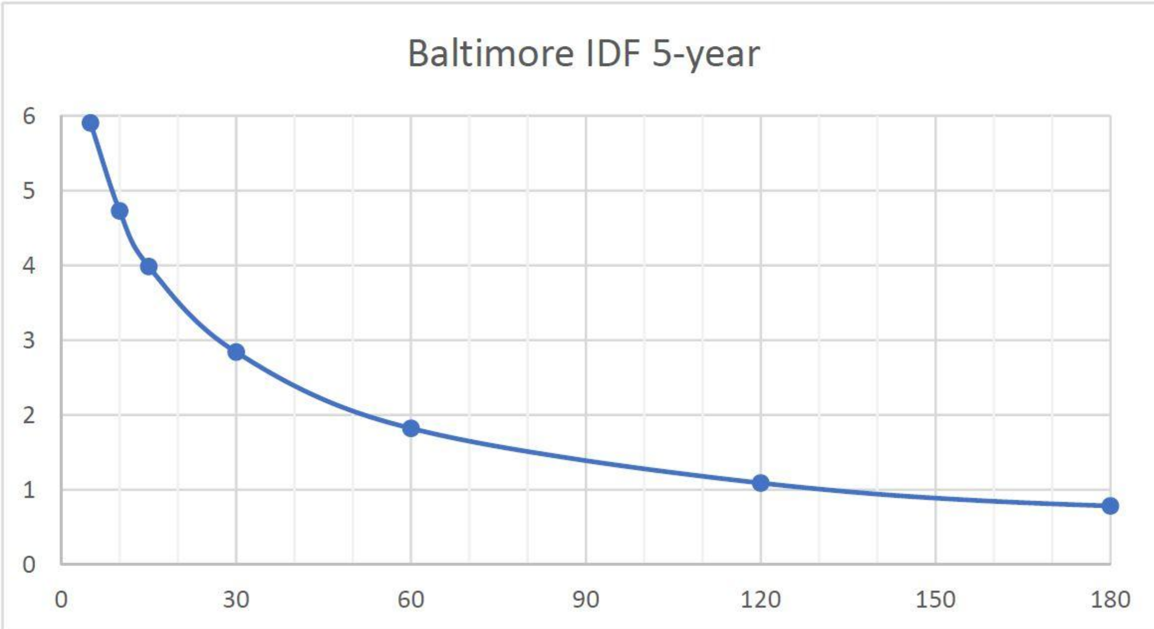

Figure 15 Rainfall Intensity-Duration-Frequency Curve

Using a duration of 16 minutes with Figure 15 gives a rainfall intensity of 3.9 inches per hour, which, when substituted into the equation, yields an estimated tc of 17 minutes. Once again, this differs from the assumed value of 16 minutes, so another iteration is required.

For this iteration, the rainfall intensity is found from Figure 15 using a duration of 17 minutes. This gives an intensity of 3.8 inches per hour. With the equation, the estimated tc is again 17 minutes. Therefore, a time of concentration of 17 minutes is used for this flow path.

Example 2.3: Time of Concentration with Iterative Sheet Flow Computations. Figure 16a shows the principal flow path for the existing conditions of a small watershed. The characteristics of each section are given in Table 4, including the land use/cover, slope, and length.

Table 4. Characteristics of Principal Flow Path for Example 3

Watershed Condition	Flow Segment	Length (ft)	Slope (ft/ft)	n	Land Use/Land Cover
Existing	A to B	500	0.07	-	Overland (forest)
	B to C	3400	0.012	0.040	Natural channel (trapezoidal): w = 1.0 ft, d = 2.0 ft, z = 2:1
	C to D	3600	0.006	0.030	Natural channel (trapezoidal): w = 4.0 ft, d = 2.0 ft, z = 2:1
Developed	E to F	80	0.07	0.013	Sheet flow: i = 2/(0.285 + D) where i = in/hr, D = hours
	F to G	400	0.07	-	Grassed swale
	G to H	900	0.02	-	Paved area
	H to J	2000	0.015	0.015	Storm drain (D = 42 inches)
	J to K	3000	0.005	0.019	Open channel (trapezoidal): w = 5.0 ft, d = 3.0 ft, z = 1:1

The shallow concentrated flow equation is used to compute the velocity of flow for section AB:

$V = \alpha\, k\, S^{0.5} \quad = 33 * 0.076 * (0.070)^{0.5} = 0.66$ ft/sec

Thus, the travel time is:

Tt = 500/(0.66*60) = 13 minutes

For the section BC, Manning's equation is used. For a trapezoidal channel, the hydraulic radius is:

$R = A/P = (wd + zd^2)/[w + 2d(1 + z^2)^{0.5}] = (1*2 + 2*2^2)/[1 + 2*2(1 + 2^2)^{0.5}] = 10/9.94 = 1.01$ ft.

Thus, Manning's equation yields a velocity of:

$V = (1.49/0.040)\, (1.01)^{2/3}\, (0.012)^{1/2} = 4.1$ ft/sec

and the travel time is:

Tt = 3400/(4.1*60) = 14 minutes

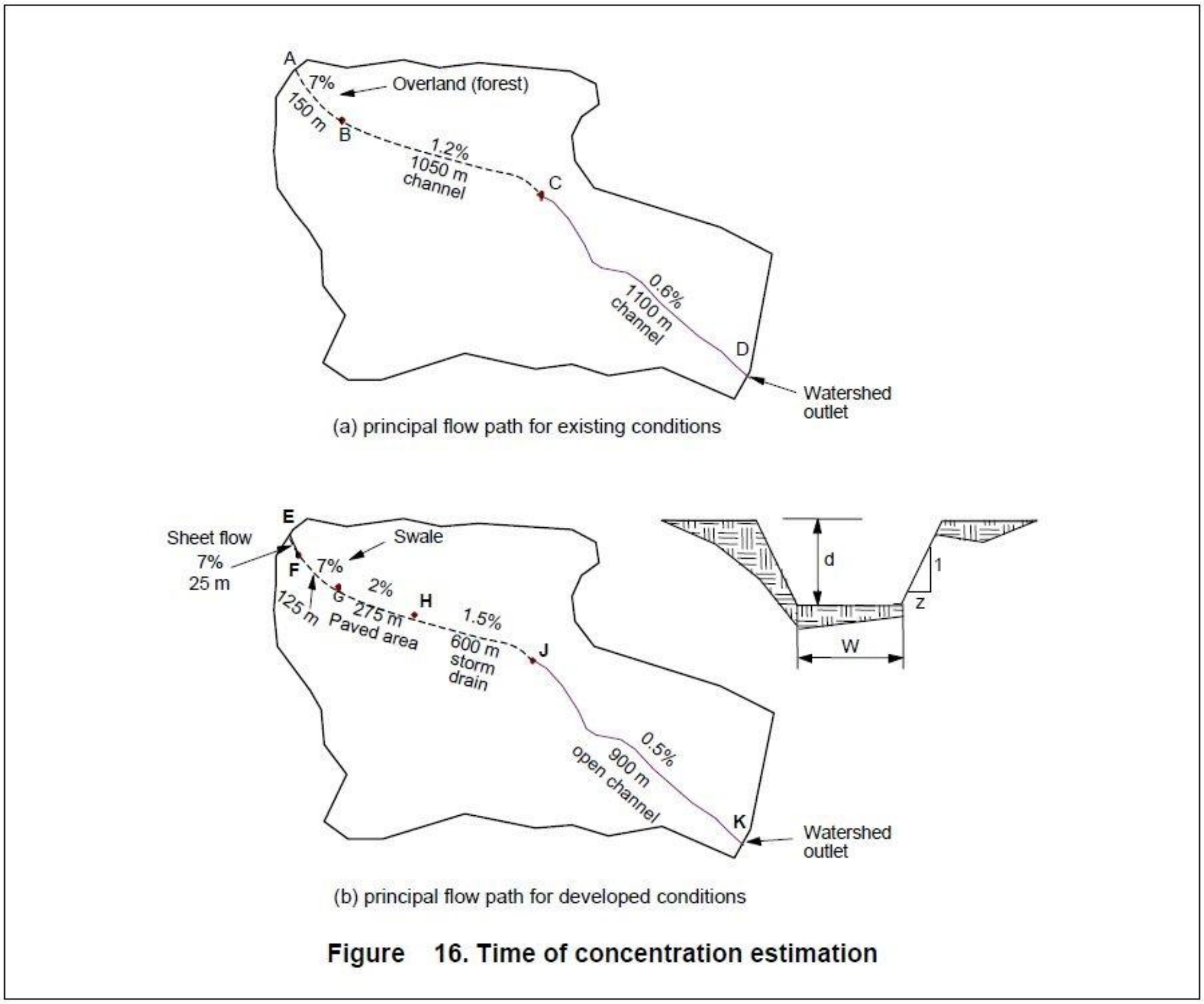

Figure 16. Time of concentration estimation

For the section CD, Manning's equation is used. The hydraulic radius is:

$R = (wd + zd^2)/[w + 2d(1 + z^2)^{0.5}] = (4*2 + 2*2^2)/[4 + 2*2(1 + 2^2)^{0.5}] = 16/12.94 = 1.24$ ft.

Thus, the velocity is:

$V = (1.49/0.030)\ (1.24)^{2/3}\ (0.006)^{1/2} = 4.4$ ft/sec

and the travel time is:

$Tt = 3600/(4.4*60) = 14$ minutes

Thus, the total travel time is the sum of the travel times for the individual segments:

$t_c = 13 + 14 + 14 = 41$ minutes

For the developed conditions, the principal flow path is segmented into five parts (see Figure

16b). For the first part of the overland flow portion, the section from E to F, the runoff is sheet flow; thus, the kinematic wave equation (Equation 4) is used. Since this is an iterative equation and we will use an intensity associated with the time of concentration for the watershed, we will calculate the travel time for this segment last.

For the section FG, the flow path consists of grass-lined swales. Equation 5 can be used to compute the velocity:

$V = \alpha\ k\ S^{0.5} \quad = 33 * 0.457 * (0.070)^{0.5} = 4.0$ ft/sec

Thus, the travel time is:

$Tt = 400/(4.0*60) = 2$ minutes

For the segment GH, the principal flow path consists of paved gutters. Thus, Equation 5 with Table 2 is used:

$V = \alpha\ k\ S^{0.5} \quad = 33 * 0.619 * (0.020)^{0.5} = 2.9$ ft/sec

Thus, the travel time is:

$Tt = 900/(2.9*60) = 5$ minutes

The segment HJ is a 42-inch pipe. Thus, Manning's equation is used. The hydraulic radius is one-fourth the diameter (D/4), so the velocity for full flow is:

$V = (1.49/0.015)\ (0.875)^{2/3}\ (0.015)^{1/2} = 11.1$ ft/sec

and the travel time is:

$Tt = 2000/(11.1*60) = 3$ minutes

The final section JK is an improved trapezoidal channel. The hydraulic radius is:

$R = (wd + zd^2)/[w + 2d(1 + z^2)^{0.5}] = (5*3 + 1*3^2)/[5 + 2*3(1 + 1^2)^{0.5}] = 24/15.39 = 1.56$ ft.

Manning's equation is used to compute the velocity:

$V = (1.49/0.019)\ (1.56)^{2/3}\ (0.005)^{1/2} = 7.5$ ft/sec

and the travel time is:

$Tt = 3000/(7.5*60) = 7$ minutes

Thus, the total travel time through the four segments (excluding the first segment) is:

$Tt = 2 + 3 + 5 + 7 = 17$ minutes

Therefore, we know that the time of concentration will be 17 min plus the time of travel over the sheet flow segment EF. For short durations at the location of this example, the 2-year IDF curve is represented by the following relationship between i and D:

$i = 2/(0.285 + D)$

where,

i = intensity, in/hr
D = duration, hr.

Iteration 1: Assume that travel time on the sheet flow segment is 2 minutes. Therefore, t_c = D = 17 + 2 = 19 min. The 2-year IDF curve is used to estimate the intensity:

i = 2/[0.285 + (19/60)] = 3.32 in/hr

Consequently, Equation 3 yields an estimate of the travel time:

$Tt = (\alpha/i^{0.4}) * [nL/(S^{0.5})]^{0.6} = (0.93/3.32^{0.4}) * [0.013*80/(0.07^{0.5})]^{0.6} = 1$ min.

Since we assumed 2 min for this segment, a second iteration will be performed using the new estimate.

Iteration 2: Assume tc = 17 + 1 = 18 min

i = 2/[0.285 + (18/60)] = 3.42 in/hr

Consequently, Equation 3 yields an estimate of the travel time:

$Tt = (\alpha/i^{0.4}) * [nL/(S^{0.5})]^{0.6} = (0.93/3.42^{0.4}) * [0.013*80/(0.07^{0.5})]^{0.6} = 1$ min.

The change in rainfall intensity did not change the travel time for this segment (rounded to the nearest minute); therefore, the computations are completed. The time of concentration for the post-developed condition is 18 min. This tc is 40 percent of the tc for the existing conditions (41 minutes).

<u>REFERENCES</u>

1. Federal Highway Administration, Hydraulic Design Series 2, “Hydrology”, 2002.
2. McCuen, R.H., 1989. “Hydrologic Analysis and Design.” Prentice-Hall, Inc., Englewood Cliffs, NJ.

HYDROLOGY PART II – PEAK FLOW OF UNGAGED SITES

At many stream crossings of interest to hydraulic engineers, there may be insufficient stream gaging records, or often no records at all, available for making a flood frequency analysis, such as a log-Pearson Type III analysis. Several regional analysis and empirical techniques have been developed and successfully applied to address these situations.

Extrapolation of data from nearby watersheds with comparable hydrologic and physiographic features is referred to as regional analysis and includes regional regression equations and index-flood methods. The USGS has collected a comprehensive series of these regional regression equations into the National Flood Frequency computer program. This tool provides the means for computing a peak discharge for any place in the United States. The USGS has also developed a more powerful software program called StreamStats that can calculate many of the basin parameters automatically and provide results from regional regressions equations. This tool is available in some states.

Empirical methods include such widely applied techniques as the rational formula and the NRCS (formerly the SCS) graphical method. These methods employ empirical relationships between rainfall and runoff that allow estimation of design discharges on ungaged watersheds by development of parameters describing the watershed. If an engineer has an interest in the magnitude of measured maximum flood flows, peak discharge envelope curves can be used alone or in conjunction with other regional or empirical analyses.

Watershed area plays an important role for each of these ungaged watershed peak flow determination methods. Watershed area is the single most important characteristic for determining runoff peaks. As will be seen, the area of the watershed also provides a basis for determining the limits of applicability for many of these methods.

REGIONAL REGRESSION EQUATIONS

Regional regression equations are commonly used for estimating peak flows at ungaged sites or sites with insufficient data. Regional regression equations relate either the peak flow or some other flood characteristic at a specified return period to the physiographic, hydrologic, and meteorologic characteristics of the watershed.

Analysis Procedure

The typical multiple regression model utilized in regional flood studies uses the power model structure:

$Y_T = aX_1^{b1} X_2^{b2} \ldots X_p^{bp}$ Equation 1

Where,

Y_T = the dependent variable

$X_1, X_2 \ldots X_p$ = independent variables

a = the intercept coefficient

$b_1, b_2 \ldots b_p$ = regression coefficients

The dependent variable is usually the peak flow for a given return period T or some other property of the particular flood frequency, and the independent variables are selected to characterize the watershed and its meteorologic conditions. The parameters a, b_1, b_2, ..., b_p are determined using a regression analysis. Regression analysis is described in detail by Sanders 1980), Riggs (1968), and McCuen (1993). The general procedure for making a regional regression analysis is as follows:

1. Obtain the annual maximum flood series for each of the gaged sites in the region.

2. Perform a separate flood frequency analysis (e.g., log-Pearson Type III) on each of the flood series of Step 1 and determine the peak discharges for selected return periods (e.g., the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year discharges are commonly selected).

3. Determine the values of watershed and meteorological characteristics for each watershed for which a flood series was collected in Step 1.

4. Form an (n by p) data matrix of all the data collected in Step 3, where n is the number of watersheds of step 1 and p is the number of watershed characteristics obtained for Step 3.

5. Form a one-dimensional vector with n peak discharges for the specific return period selected.

6. Regress the vector of n peak discharges of Step 5 on the data matrix of Step 4 to obtain the prediction equation.

If more than one return period is of interest, the procedure can be repeated for each return period, with a separate equation developed for each return period. In this case, it is also important to review closely the regression coefficients to ensure that they are rational and consistent across the various return periods. Because of sampling variation, it is possible for the regression analyses to produce a set of coefficients that, under certain sets of values for the predictor variables, result in the computed 10-year discharge, for example, being greater than the computed 25-year discharge. In such cases, the irrational predictions can be eliminated by smoothing the coefficients. If the coefficients need to be smoothed, the goodness-of-fit statistics should be recomputed using the smoothed coefficients. The problem can usually be prevented by using the same predictor variables for all of the equations. This becomes a complex statistical analysis and is generally best left to engineers with an extensive background in statistical analysis.

The most important watershed characteristic is usually the drainage area and almost all regression formulas include drainage area above the point of interest as an independent variable. The choice of the other watershed characteristics is much more varied and can include measurements of channel slope, length, and geometry, shape factors, watershed perimeter, aspect, elevation, basin fall, land use, and others. Meteorological characteristics that are often considered as independent variables include various rainfall parameters, snowmelt, evaporation, temperature, and wind.

As many independent variables as desired can be used in a regression analysis although it would be unlikely that more than one measure of any particular characteristic (such as precipitation) would be included. The statistical significance of each independent variable can be determined and those that are statistically insignificant at a specified level of significance (e.g., 5 percent) can be eliminated. In addition to statistical criteria, it is also important for all coefficients to be reasonable.

The specific predictor variables to be included in a regression equation are usually selected using a stepwise regression analysis (McCuen, 1989). While a 5 percent level of significance is sometimes used to make the decision, it is better to select only those variables that are easily obtained and necessary to provide both a reasonable level of accuracy and rational coefficients. When stepwise regression analysis is used to select variables for a set of equations for different return periods, the same independent variables should be used in all of the equations. In a few cases, this may cause some equations in the set to have less accuracy than would be possible, but it is usually necessary to ensure consistency across the set of equations.

USGS Regression Equations

In a series of studies by the USGS, the Federal Highway Administration, and State Highway Departments, statewide regression equations have now been developed throughout the United States. The highway community has made a significant contribution to acquiring additional stream flow data through funding USGS stream gaging station studies throughout the country since the 1960s. Highway interests have supported these research endeavors with expenditures of $14 million. These equations permit peak flows to be estimated for return periods varying from 2 to 500 years. The published equations (Jennings, et al., 1994) are included in the USGS StreamStats Program discussed later in this course and were originally included in the National Flood Frequency (NFF) Program.

Typically, each state is divided into regions of similar hydrologic, meteorologic, and physiographic characteristics as determined by various hydrological and statistical measures. Using a combination of measured data and rainfall-runoff simulation models such as that of Dawdy, et al. (1972), long-term records of peak annual flow were synthesized for each of several watersheds in a defined region. Each record was subjected to a log-Pearson Type III frequency analysis, adjusted as required for loss of variance due to modeling, and the peak flow for various frequencies determined.

Multiple regression was then used on the logarithmically transformed values of the variables to obtain regression equations of the form of Equation 1 for peak flows of selected frequencies. Only those independent variables that were statistically significant at a predetermined level of significance were retained in the final equations.

Hydrologic Flood Regions

In most statewide flood-frequency reports, the analysts divided the state into separate hydrologic regions. Regions of homogeneous flood characteristics were generally determined by using major watershed boundaries and an analysis of the areal distribution of the regression residuals, which are the differences between regression and station (observed) T-year estimates. In some instances, the hydrologic regions were also defined by the mean elevation of the watershed or by statistical tests such as the Wilcoxon signed-rank test.

Regression equations are defined for 210 hydrologic regions throughout the Nation, indicating that, on average, there are about four regions per state. Figure 1 gives the NFF statewide results for Maine and is used to illustrate the content for one of the 210 regions. Some areas of the Nation, however, have inadequate data to define flood-frequency regions. For example, there are regions of undefined flood frequency in Florida, Texas, and Nevada. For the state of Hawaii, regression equations are only provided for the island of Oahu.

Summary	Procedure
Maine is considered to be a single hydrologic region. The regression equations developed for the state are for estimating peak discharges (QT) having recurrence intervals T that range from 2 to 100 years. The explanatory basin variables used in the equations are drainage area (A) in square miles; channel slope (S) in feet per mile; and storage (St), which is the area of lakes and ponds in the basin in percentage of total area. The constant 1 is added to St in the computer application of the regression equations. The user should enter the actual value of St. All variables can be measured from topographic maps. The regression equations were developed from peak-discharge records through 1974 for 60 sites with records of at least 10 years in length. The regression equations apply to streams having drainage areas greater than 1 square mile and virtually natural flood flows. Standard errors of the estimate of the regression equations range from 31 to 49 percent.	Topographic maps and the following equations are used to estimate the needed peak discharges QT, in cubic feet per second, having selected recurrence intervals, T. $Q2 = 14.0\ A^{0.962}\ S^{0.268}\ ST^{-0.212}$ $Q5 = 21.2\ A^{0.946}\ S^{0.298}\ ST^{-0.239}$ $Q10 = 26.9\ A^{0.936}\ S^{0.315}\ ST^{-0.252}$ $Q25 = 35.6\ A^{0.923}\ S^{0.333}\ ST^{-0.266}$ $Q50 = 42.7\ A^{0.915}\ S^{0.346}\ ST^{-0.275}$ $Q100 = 50.9\ A^{0.907}\ S^{0.358}\ ST^{-0.282}$ Reference: Morrill, R.A., 1975, “A Technique for Estimating the Magnitude and Frequency of Floods in Maine.” U.S. Geological Survey Open-File Report No. 75-292.

Figure 1. Description of NFF regression equations for rural watersheds in Maine (Jennings, et al., 1994).

Example 1. To illustrate the use of regional regression equations for estimating peak flows, consider the following example.

It is desired to renovate a bridge at a highway crossing of the Seco Creek at D'Hanis, TX. The site is ungaged and the design return period is 25 years. The site lies in Region 5 as defined by Schroeder and Massey (1970). The equations have the following form:

$Q_T = a\, A^{b1}\, S^{b2}$ Equation 2

where

Q_T = peak annual flow for the specified return periods, ft^3/s

A = drainage area contributing surface runoff above the site, mi^2

S = average slope of the streambed between points 10 and 85 percent of the distance along the main stream channel from the site to the watershed divide, ft/mi.

The coefficients of Equation 2 are given in Table 1. The range of application of the above equations was specified as:

Variable	Value in English Units
Drainage Area (A)	$1.08 < A (mi^2) < 1950$
Slope (S)	$9.2 < S (ft/mi) < 76.8$

Table 1. Regression Coefficients for Texas, Region 5

Return Period, T (years)	Regression Coefficients			Standard Error (%)*
	a	b1	b2	
2	4.82	0.799	0.966	62.1
5	36.4	0.776	0.706	46.6
10	82.6	0.776	0.622	42.6
25	180	0.776	0.554	41.3
50	278	0.778	0.522	42.0
100	399	0.782	0.497	44.1

* Standard errors were computed using the logarithmic regression and are given as a percentage of the mean

By measuring the drainage area above the site from a topographic map, the area A is found to be 210.6 mi^2 and the channel slope between the 10 and 85 percent points is 14.96 ft/mi. Using Equation 2 and the coefficients of Table 1, the 25-year peak flow is:

Variable	Value in English Units
$Q_{25} = a_{25}\, A^{0.776}\, S^{0.554}$	$= 180\, (210)^{0.776}\, (14.96)^{0.554} = 51{,}200\ ft^3/s$

Assessing Prediction Accuracy

In most cases, regional regression equations are given with associated standard errors, which are indicators of how accurately the regression equation predicts the observed data used in their development. The standard error of estimate is a measure of the deviation of the observed data from the corresponding predicted values and is given by the basic equation:

$$Se = [(\sum (\bar{Q}_i - Q_i)^2)/(n-q)]^{0.5} \qquad \text{Equation 3}$$

Where,

Q_i = observed value of the dependent variable
$\bar{Q}_i$ = corresponding value predicted by the regression equation
n = number of watersheds used in developing regression equation
q = number of regression coefficients (i.e. a, b_1, b_2 … b_p)

In a manner analogous to the variance, the standard error can be expressed as a percentage by dividing the standard error S_e by the mean value ($\bar{Q}_T$) of the dependent variable:

$$V_e = (S_e/\bar{Q}_T) \times 100\% \qquad \text{Equation 4}$$

Where

V_e = coefficient of error variation

V_e of Equation 4 has the form of the coefficient of variation. The standard error of regression S_e has a very similar meaning to that of the standard deviation for a normal distribution in that approximately 68 percent of the observed data should be contained within ±1 standard error of the regression line.

When S_e is computed for regional regression equations, it is usually computed using the logarithms of the flows. Thus $\bar{Q}_i$ and Q_i of Equation 3 are logarithms of the corresponding flows. This is believed to be necessary because the errors (i.e., $\bar{Q}_i - Q_i$) have a constant variance when expressed from logarithms.

Comparison with Gaged Estimates

Because of the extensive use now being made of USGS regression equations, it is of interest to compare peak discharges estimated from these equations with results obtained from a formal flood frequency analysis. A direct comparison cannot be made with the previously used Medina River data because of storage and regulation upstream of the gage.

Since regression equations apply only to totally unregulated flow, Station 08179000, Medina River near Pipe Creek, Texas, has been selected for comparison. This gage has 43 years of record, drains an area of 474 mi^2, is totally unregulated, and has station and generalized skews of -0.005 and -0.234, respectively. The data were analyzed with a log-Pearson Type III distribution, and the 10-, 25-, 50- and 100-year peak discharges estimated using the USGS Bulletin 17B (1982) weighted skew option (G_L = -0.2). These values together with peak flows determined from a frequency curve through the systematic record are summarized in Table 2.

Table 2. Comparison of Peak Flows from Log-Pearson Type III Distribution and USGS Regional Regression Equation

Return Period (years)	Peak Discharge (ft^3/s)		
	Log-Pearson Type III Frequency	Systematic Record	USGS Regression Equations
10	42,700	50,300	55,700
25	68,900	89,000	100,000
50	92,900	129,000	144,000
100	120,900	179,000	197,000

The Pipe Creek gage is located in Region 5 in Texas and the regression equations given for the Seco Creek example above are applicable. The watershed has an average slope of 16.2 ft/mi between 10 and 85 percent points along the main stream channel. The corresponding peak flows calculated from the appropriate regression equations are also summarized in Table 2.

The peak discharges estimated from the regression equations are all substantially higher than the comparable values determined from the log-Pearson Type III analysis, although all are within the USGS Bulletin 17B, upper 95-percent confidence limits. Further review of the data at this station indicates that a frequency curve constructed using the systematic record plots above the log-Pearson Type III distribution curves at least over the range of frequencies considered in the above comparison. This is partially a result of a peak flow in 1978 in excess of 281,000 ft^3/s, which, according to the log-Pearson Type III analysis, is an event approaching the 500-year peak flow.

It has been suggested by some experienced hydrologists that regression equations may give better estimates of peak flows of various frequencies than formal statistical frequency analyses. They reason that regression equations more nearly reflect the potential or capacity of the watershed to experience a peak flow of given magnitude, whereas a frequency analysis is biased by what has been recorded at the gage. Some justification exists for this argument as there are many examples throughout the country of adjacent watersheds of comparable size and physiographic and hydrologic characteristics experiencing the same storm patterns, but wherein only one has recorded major floods. This is obviously a function of where the storm occurs, but frequency analyses of gaged data from the different watersheds may give very different peak flows for the same frequencies. On the other hand, regression equations will give comparable flood magnitudes at the same frequencies for each watershed, all other factors being approximately equal.

This is not to suggest that regional regression equations should take precedence over frequency analysis, especially when sufficient data are available. Regression equations, however, do serve as a basis for comparison of statistically determined peak flows of specified frequencies and provide for further evaluation of the results of a frequency analysis. They may be used to add credence to historical flood data or may indicate that historical records should be sought out and incorporated into the analysis. Regression equations can also provide insight into the treatment of outliers beyond the purely statistical methods. As demonstrated by the above discussion, comparison of the peak flows obtained by different methods may indicate the need to review data from other comparable watersheds within a region and the desirability of transposing or extending a given record using data from other gages.

Sauer (1973) has proposed a methodology for weighting the log-Pearson Type III result with the regression equation estimate for the gaged watershed based on the gage record length and the equivalent record length for the regression equation as follows:

$Q_{gw} = (Q_g N_g + Q_r N_r)/(N_g+Nr)$ Equation 5

This methodology seeks to use information in the gage record as well as similar gaged watersheds in the region via the regression equations. It is presented in many of the USGS reports documenting development of the regression equations.

Application and Limitations

Several points should be kept in mind when using regional regression equations. For the most part, the state regional equations are developed for unregulated, natural, nonurbanized watersheds. They separate out mixed populations (i.e., rain produced floods from snowmelt floods or hurricane associated storms). The equations are regionalized so that it is incumbent on the user to carefully define the hydrologic region and to define the dependent and independent variables in the exact manner prescribed for each set of regional equations. This includes applying the equations to basins that fall within the range of characteristics for basins used to develop the equations. The designer is also cautioned to apply these equations within or close to the range of independent variables utilized in the development of the equations.

Although not a serious problem, the designer should be alert to any discrepancies in results from regression equations when applied at regional boundaries and especially near state boundaries. Within-state regional boundaries generally define hydrologic regions with similar characteristics, and regression equations may not give comparable results near regional boundaries.

Hydrologic regions also may cross state boundaries, and regression equations for adjacent regions in different states can give substantially different peak flows for the same frequency. When working near within-state regional and state boundaries, regression equations for adjacent regions should be checked and any serious discrepancies reconciled.

The following additional limitations should be observed:

- Rural equations should only be used for rural areas and should not be used in urban areas unless the effects of urbanization are insignificant.

- Regression equations should not be used where dams, flood-detention structures, and other human-made works have a significant effect on peak discharges.

- The magnitude of the standard errors can be larger than the reported errors if the equations are used to estimate flood magnitudes for streams with variables outside the ranges for the necessary input variables as stated in the applicable report.

- Drainage area should always be determined. Although a hydrologic region might not include drainage area as a variable in the prediction equation to compute a frequency curve, the drainage area may be used for determining the maximum flood envelope discharge from Crippen and Bue (1977) and Crippen (1982), as well as weighting of curves for watersheds in more than one region.

- Frequency curves for watersheds contained in more than one region cannot be computed if the regions involved do not have corresponding T-year equations. Failure to observe this limitation will lead to erroneous results. Frequency curves are weighted by the percentage of drainage area in each region. No provision is provided for weighting frequency curves for watersheds in two different states.

- In some instances, the maximum flood envelope value might be less than some T-year computed peak discharges for a given watershed. The T-year peak discharge is the discharge that will be exceeded as an annual maximum peak discharge, on average, once every T years. The engineer should carefully determine which maximum flood-region contains the watershed being analyzed and is encouraged to consult Crippen and Bue (1977) and Crippen (1982) for guidance and interpretations.

- The engineer should be cautioned that some hydrologic regions do not have prediction equations for peak discharges as large as the 100-year peak discharge. The engineer is responsible for the assessment and interpretation of any interpolated or any extrapolated T- year peak discharge. Examination of plots of the frequency curves is highly desirable.

Maximum flood envelopes are discussed later in this course.

USGS Urban Watershed Studies

In 1978, the Federal Highway Administration contracted with the USGS to conduct a nationwide survey of flood frequencies under urban conditions. The purposes of the study were to: review the literature of urban flood studies, compile a nationwide data base of flood frequency characteristics including land use variables for urban watersheds, and define estimating techniques for ungaged urban areas. Results of the study are described in detail in USGS Water Supply Paper 2207 (Sauer, et al., 1983).

A review of nearly 600 urbanized sites resulted in a final list of 269 sites that met criteria wherein at least 15 percent of the drainage area was covered with commercial, industrial, or residential development; reliable flood data were available for 10 or more years (either actual peak flow data or synthesized data from a calibrated rainfall-runoff model); and the period of record was coincident with a period of relatively constant urbanization. The complete data base, including topographic and climatic variables, land use variables, urbanization indices, and flood frequency estimates are available from the USGS National Center, Reston, VA.

The USGS study developed a procedure for quantifying the effects of urbanization on peak discharge and flood volume. Regression equations relate the peak discharge at a specified frequency to: (1) drainage area, (2) peak discharge for the same watershed in a rural condition, and (3) a basin development factor (BDF). The basin development factor is a measure of the degree of urbanization that exists (or might exist in the future) in the watershed. The BDF is discussed in more detail later in this course.

The USGS regression equations can be used to estimate the peak discharge for existing conditions of urbanization, and they can also be used to estimate the peak discharge for future conditions. The urban peak flow equations are applicable to a wide variety of geographic and climatologic conditions. They can provide useful estimates of the relative impact that varying amounts of urbanization have on peak discharge. However, these estimates cannot be treated as absolutes, and some judgment must be exercised in their application.

Peak Discharge Equations

Initially, the USGS study developed regression equations for urban peak flow discharge in terms of seven independent variables. Subsequently, it was found that by eliminating the less significant independent variables from the regression analyses, simpler equations could be obtained without appreciably increasing the standard error of regression. Ultimately, the following family of three-parameter equations was developed by the USGS for peak discharges in urbanized watersheds:

$UQ^T = a_T\ A^{C1T}\ (13 – BDF)\ C^{2T}\ RQ_T^{C3T}$

Where,

UQ^T = peak discharge of recurrence interval, T, for an urbanized condition, ft^3/s
T = recurrence interval ranging from 2 to 500 years
A = drainage area of basin, mi^2
BDF = basin development factor as defined below
RQ_T = peak discharge of recurrence interval, T, for rural conditions, ft^3/s
a_T, C_{1T}, C_{2T} and C_{3T} = regression constants summarized in Table 3

This equation is applicable for watersheds between 0.2 and 100 mi^2.

Table 3. Unit Conversion Constants for the USGS Urban Equations

Return Period	a_T	C_{1T}	C_{2T}	C_{3T}
2	13.2	0.21	-0.43	0.73
5	10.6	0.17	-0.39	0.78
10	9.51	0.16	-0.36	0.79
25	8.68	0.15	-0.34	0.80
50	8.04	0.15	-0.32	0.81
100	7.70	0.15	-0.32	0.82
500	7.47	0.16	-0.30	0.82

Basin Development Factor

Several indices of urbanization were evaluated in the course of the USGS study including percentage of basin occupied by impervious surfaces, population and population density, basin response time, and basin development factor. The BDF, which provides a measure of the efficiency of the drainage system within an urbanizing watershed, was selected for a number of reasons. The BDF was highly significant in the regression equations, compared to the other measures of urbanization, and its value may be determined from topographic maps, storm drain maps, and field surveys.

To determine the BDF, the basin is first divided into three sections as shown in Figure 2. Each section contains approximately one-third of the drainage area of the watershed. Travel time is given consideration when drawing these boundaries so that the travel distances along two or more streams within a particular third are about equal. This does not mean that the travel distances of all three subareas are equal, only that within a particular subarea the travel distances are approximately equal.

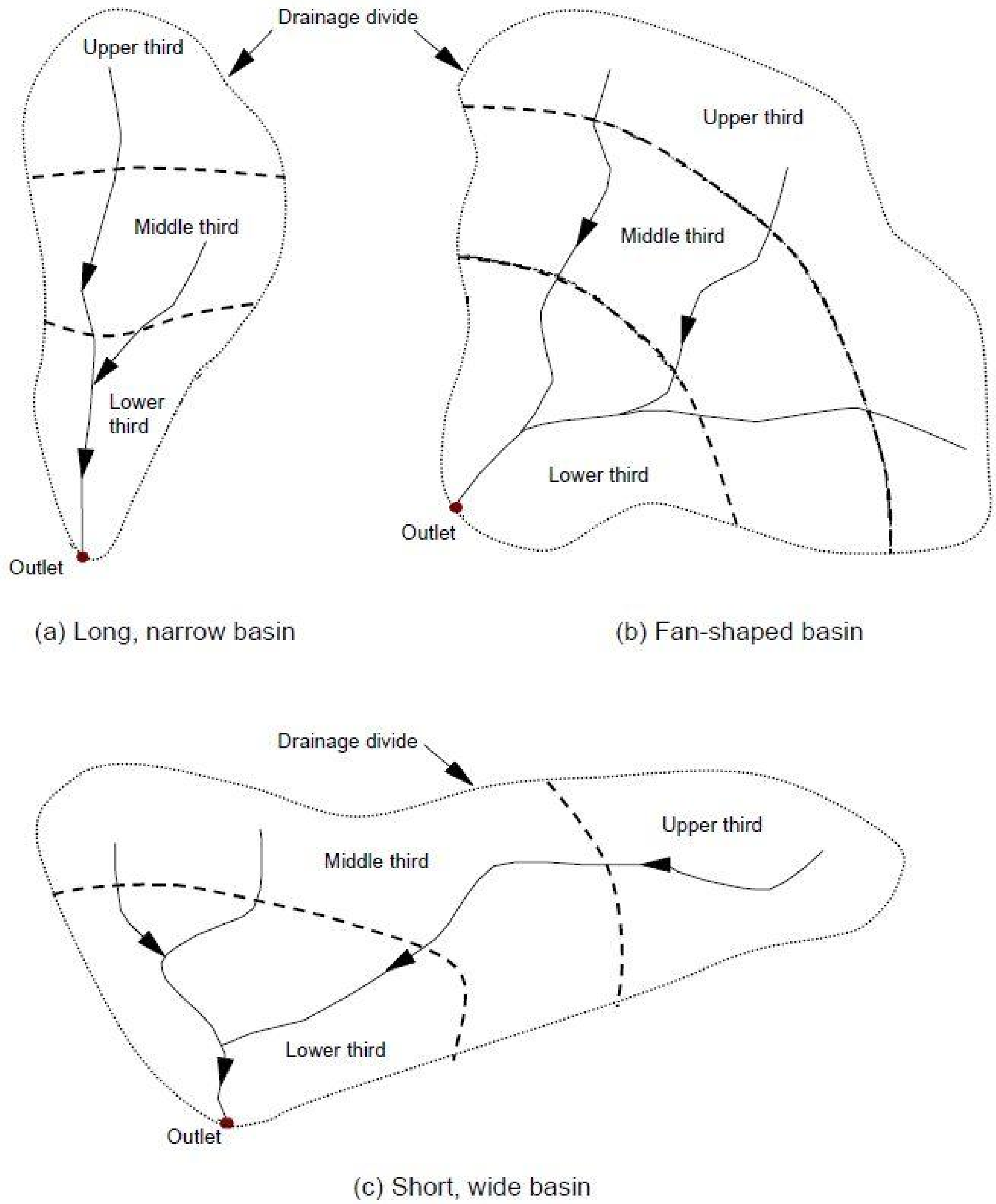

Figure 2. Subdivision of watersheds for determination of BDF

Within each section of the basin, four aspects of the drainage system are evaluated and assigned a code:

1. Channel modifications. If channel modifications such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channel and principal tributaries (those that drain directly into the main channel), a code of 1 is assigned. Any one, or all, of these modifications would qualify for a code of 1. To be considered significant, at least 50 percent of the main drainage channels and principal tributaries must be modified to some extent over natural conditions. If channel modifications are not prevalent, a code of 0 is assigned.

2. Channel linings. If more than 50 percent of the main drainage channel and principal tributaries have been lined with an impervious material, such as concrete, a code of 1 is assigned. If less than 50 percent of these channels are lined, a code of 0 is assigned. The presence of channel linings would probably indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.

3. Storm drains or storm sewers. Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Quite often these drains empty into the main tributaries and channels that are either open channels or in some basins may be enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a section consist of storm drains, a code of 1 is assigned. If less than 50 percent of the secondary tributaries consist of storm drains, a code of 0 is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a code of 1.

4. Urbanization/Curb and gutter streets. If more than 50 percent of a subarea is urbanized (covered by residential, commercial, and/or industrial development), and if more than 50 percent of the streets and highways in the subarea is constructed with curbs and gutters, a code of 1 should be assigned. Otherwise, a code of 0 is assigned. Frequently, drainage from curb and gutter streets will empty into storm drains.

The above guidelines for determining the various drainage system codes are not intended to be precise measurements. Practical determination involves a certain amount of subjectivity and engineering judgment. It is recommended that field checking be performed to obtain the best estimate. The BDF is computed as the sum of the assigned codes. With three subareas per basin, and four drainage aspects to which codes are assigned in each subarea, the maximum value for a fully developed drainage system would be 12. Conversely, if the drainage system has not been developed, a BDF of 0 would result. Such a condition does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, and have some improvements to secondary tributaries, and still have an assigned BDF of 0. It will be shown later that such a condition will still frequently cause increases in peak discharges.

The BDF is a fairly easy index to estimate for an existing urban basin. The 50 percent guideline is usually not difficult to evaluate because many urban areas tend to use the same design criteria throughout, and therefore the drainage aspects are similar throughout. Also, the BDF is convenient to use for projecting future development. Full development and maximum urban effects on peaks would occur when BDF = 12. Projections of full development, or intermediate stages of development, can usually be obtained from city development plans.

Example 2 (Note this example is in metric units). Information is first collected from topographic maps and a field survey for the 99.3-ha watershed. The watershed is divided into three subareas of approximately equal area. The separation is based on homogeneity of hydrologic conditions, with the following values measured:

Subarea	Area (ha)	Main Channel Length (m)	Length of Secondary Tributaries (m)	Road Length (m)	Length of Channel Modified (m)	Length of Channel Lined (m)	Storm Drains (m)	Curb and Gutter (m)
Upper	29.2	780	1580	870	140	0	410	210
Middle	36.3	1140	1200	1430	615	540	680	920
Lower	33.8	910	660	1710	525	480	460	970
Sum	99.3	2830						

The BDF is determined as follows:

Channel Modifications
Upper Third: 140 m have been straightened and deepened — Code = 0
[140/780 < 50%]
Middle Third: 615 m have been straightened and deepened — = 1
[615/1140 > 50%]
Lower Third: 525 m have been straightened and widened — = 1
[525/910 > 50%]

Channel Linings
Upper Third: 0 m of channel are lined — Code = 0
[0/780 < 50%]
Middle Third: 540 m of channel are lined — = 0
[540/1140 < 50%]
Lower Third: 480 m of channel are lined — = 1
[480/910 > 50%]

Storm Drains on Secondary Tributaries
Upper Third: 410 m have been converted to drains — Code = 0
[410/1580 < 50%]
Middle Third: 680 m have been converted to drains — = 1
[680/1200 > 50%]
Lower Third: 460 m have been converted to drains — = 1
[460/660 > 50%]

Curb and Gutter Streets
Upper Third: 20% urbanized with 210 m curb/gutter — Code = 0
[210/870 < 50%]
Middle Third: 70% urbanized with 920 m curb/gutter — = 1
[920/1430 > 50%]
Lower Third: 55% urbanized with 970 m curb/gutter — = 1
[970/1710 > 50%]

Total BDF = 7

Example 2 (English units). Information is first collected from topographic maps and a field survey for the following watershed. The watershed is divided into three subareas of approximately equal area. The separation is based on homogeneity of hydrologic conditions, with the following values measured:

Subarea	Area (ac)	Main Channel Length (ft)	Length of Secondary Tributaries (ft)	Road Length (ft)	Length of Channel Modified (ft)	Length of Channel Lined (ft)	Storm Drains (ft)	Curb and Gutter (ft)
Upper	72.2	2560	5180	2850	460	0	1350	690
Middle	89.7	3740	3940	4690	2020	1770	2230	3020
Lower	83.5	2990	2170	5610	1720	1570	1510	3180
Sum	245.4	9290						

The BDF is determined as follows:

Channel Modifications

Upper Third: 460 ft have been straightened and deepened [460/2,560 < 50%] Code = 0

Middle Third: 2,020 ft have been straightened and deepened [2,020/3,740 > 50%] = 1

Lower Third: 1,720 have been straightened and widened [1,720/2,990 > 50%] = 1

Channel Linings

Upper Third: 0 ft of channel are lined [0/2,560 < 50%] Code = 0

Middle Third: 1,770 ft of channel are lined [1,770/3,740 < 50%] = 0

Lower Third: 1,570 of channel are lined [1,570/2,990 > 50%] = 1

Storm Drains on Secondary Tributaries

Upper Third: 1,350 ft have been converted to drains [1,350/5,180 < 50%] Code = 0

Middle Third: 2,230 ft have been converted to drains [2,230/3,940 > 50%] = 1

Lower Third: 1,510 ft have been converted to drains [1,510/2,170 > 50%] = 1

Curb and Gutter Streets

Upper Third: 20% urbanized with 690 ft curb/gutter [690/2,850 < 50%] Code = 0

Middle Third: 70% urbanized with 3,020 ft curb/gutter [3,020/4,690 > 50%] = 1

Lower Third: 55% urbanized with 3,180 ft curb/gutter [3,180/5,610 > 50%] = 1

Total BDF = 7

Example 3. The 25-year peak discharge is computed for an urban watershed of 26 mi^2 with a BDF of 4. The percentage increase over the undeveloped rural condition is also computed.

1. Determine the equivalent rural discharge using the published USGS statewide regression equation. For this site, the 25-year peak discharge for the rural conditions is determined from the following equation:

Variable	Value in English Units
$RQ_{25} = a_{25}\ A^{CT}$	$= 280\ (26)^{0.666} = 2450\ ft^3/s$

2. Determine the urbanized discharge:

$$UQ_{25} = a_{25}\ A^{C1.25}\ (13\text{-}BDF)^{\ C.2.25}\ RQ_{25}^{C3.25}$$

$$UQ_{25} = 8.68\ (26)^{C0.15}\ (13\text{-}4)^{\ -0.34}\ (2450)^{0.80} = 3450\ ft^3/s$$

3. Determine the percent change:

Variable	Value in English Units
% change = $[(UQ_{25} – RQ_{25})/RQ_{25}]$ x 100%	= [(3450-2450)/2450] x 100% = 41%

Effects of Future Urbanization

The regression equations can also be used to determine the effects of future urbanization upon peak discharges. This calculation is simplified by performing some algebraic manipulation of the regression equations. This is illustrated by showing the impact on the 5-year peak discharge when the BDF changes from 5 to 10.

For the present and future conditions, the 5-yr peak discharge is computed with Equation 5.6:

$$UQ_5 = a_5\ A^{0.17}\ (13 - BDF_i)^{-0.39}\ RQ_5^{0.78}$$

where i = p and i = f for the present and the future BDF, respectively. The change in the BDF is:

$$\Delta BDF = (BDF_f – BDF_p) \qquad \text{Equation 7}$$

which can be rearranged to:

$$BDF_f = BDF_p) + \Delta BDF \qquad \text{Equation 8}$$

The ratio of the future UQ_{5f} to the present UQ_{5p} is:

$$UQ_{5f}/UQ_{5p} = \{a_5\ A^{0.17}\ [13 - (BDF_p + \Delta BDF)]^{-0.39}\ RQ_5^{0.78}\}/(a_5\ A^{0.17}\ [13 - BDF_p]^{-0.39}\ RQ_5^{0.78})$$

Equation 9

Canceling the common terms and rearranging yields:

$UQ_{5f}/UQ_{5p} = [1 - (\Delta BDF/(13 - BDF_p))]^{-0.39}$ Equation 10

For this example, $BDF_p = 5$ and $\Delta BDF = (10 - 5)$; therefore:

$UQ_{5f}/UQ_{5p} = [1 - (5/(13 - 5))]^{-0.39} = 1.47$

Thus, the future 5-year peak discharge is 47 percent higher than the present 5-year peak discharge.

The same approach can be applied to the other recurrence intervals yielding the following general equation:

$UQ_f/UQ_p = [1 - (\Delta BDF/(13 - BDF_p))]^{C2T}$ Equation 11

where C_{2T} varies with recurrence intervals as given in Table 3.

Local Urban Equations

Many of the USGS regression studies include additional equations for some cities and metropolitan areas that were developed for local use in those designated areas only. These local urban equations can be used in lieu of the nationwide urban equations, or they can be used for comparative purposes. It would be highly coincidental for the local equations and the nationwide equations to give identical results.

Therefore, it is advisable to compare results of the two (or more) sets of urban equations, and to also compare the urban results to the equivalent rural results. Ultimately, it is the engineer's decision as to which urban results to use.

Local urban equations are available in many cities throughout the United States. In addition, some of the rural reports contain estimation techniques for urban watersheds. Several of the rural reports suggest the use of the nationwide equations given by Sauer, et al. (1983).

<u>FHWA Regression Equations</u>

In 1977, the Federal Highway Administration published a two-volume report by Fletcher, et al. (1977) that presents nationwide regression equations for predicting runoff from small rural watersheds (<50 m^2). This method is not the equivalent of the USGS regression equations. While it was used rather widely at first, it is rarely used today. The procedure is similar in concept to that of Potter (1961). It was developed using frequency analyses of data in over 1000 small watersheds throughout the United States and Puerto Rico to relate peak flows to various hydrographic and physiographic characteristics. Three-, five-, and seven-parameter regression equations were developed for the 10-year peak runoff for each of 24 hydrophysiographic regions. Since the standard errors of estimate were found to be approximately the same for each regression equation option, the following discussion is limited to the three-parameter equations only.

If a drainage structure is to be designed to carry the probable maximum flood peak, $Q_{p(max)}$ in ft^3/s), Fletcher, et al. (1977) give the equation:

$Q_{p(max)} = 10^{[C0 + C1 \log A + C2(\log A)^2]}$ Equation 12

Where
log A = base 10 logarithm of the drainage area, mi^2
$Q_{p(max)}$ = discharge, ft^3/s
C_0, C_1 and C_2 = regression coefficients equal to 3.920, 0.8130 and -0.0325

If it is feasible to construct a very large drainage structure to handle this probable maximum flow, the hydrologic analysis is essentially complete. Similarly, if a minimum size drainage structure is specified, and its carrying capacity is greater than $Q_{p(max)}$, no further analysis is required.

A more common problem in highway drainage is that the structure must be designed to handle a flow of specified frequency. This can be accomplished with the three-parameter FHWA regression equations. The basic form of these equations is:

$q_{10} = a\ A^{b1}\ R^{b2}\ E_c^{b3}$ Equation 13

where
q_{10} = 10-year peak discharge, ft^3/s
A = drainage area, mi^2
R = isoerodent factor defined as the product of the mean annual rainfall kinetic energy and the maximum respective 30-minute annual maximum rainfall intensity
E_c = difference in elevation measured along the main channel from the drainage structure site to the drainage basin boundary, ft
a, b_1, b_2 and b_3 = regression coefficients

Values of the drainage area and elevation difference are readily determined from topographic maps and R is taken from individual state isoerodent maps given by Fletcher, et al. (1977).

Two options are available to use the three-parameter regression equations. The first involves the application of an equation of the same form as Equation 13 for a specific hydrophysiographic zone. Twenty-four zones are defined covering the United States and Puerto Rico and each has its own regression equation for q_{10}. The second option involves the use of an all-zone equation developed from all of the data. The all-zone, three-parameter equation for the 10-year peak discharge, $q_{10(3AZ)}$, is:

$q_{10(3AZ)}, = 0.02598\ A^{0.56172}\ R^{0.94356}\ E_c^{0.16887}$ Equation 14

For each of the 24 hydrophysiographic zones, is a correction equation is presented to adjust Equation 15 for zonal bias. These correction equations have the form:

$q_{10} = a_1\ q_{10(3AZ)}^{b1}$ Equation 15

where a_1 and b_1 = regression coefficients.

If the surface area of surface water storage is more than about 4 percent of the total drainage area, it is recommended that the value of q_{10} computed from an individual zone equation or the corrected all-zone equation be further adjusted with a storage-correction multiplier given with the equations.

Fletcher, et al. (1977) presented the following equations from which a frequency curve can be drawn on any appropriate probability paper:

$Q_{2.33} = 0.47329\ q_{10}^{1.00243}$ Equation 16
$Q_{50} = 1.58666\ q_{10}^{1.02342}$ Equation 17

$Q_{100} = 1.82393\ q_{10}^{1.02918}$ Equation 18

Where,

$Q_{2.33}$ = mean annual peak flow taken at a return period of 2.33 years
Q_{50} and Q_{100} = 50- and 100-year peak flows, respectively.

From this curve, the flow for any other selected design frequency can be determined.

The concept of risk can also be incorporated into the FHWA regression equations. Recall that risk is the probability that one or more floods will exceed the design discharge within the life of the project. Methods presented by Fletcher, et al. (1977) permit the return period of the design flood to be adjusted according to the risk the designer can accept. The concept of the probable maximum peak flow is also useful because it represents the upper limit of flow that might be expected. It can, therefore, have application to situations where the consequences of failure are very large or unacceptable.

USGS Streamstats Software

How StreamStats Works

StreamStats is a map-based web application that incorporates a Geographic Information System (GIS) to provide users with access to an assortment of analytical tools that are useful for water-resources planning and management, engineering and design purposes. StreamStats incorporates (1) a map-based user interface for the site selection; (2) a relational data base that contains information for data-collection stations and regression equations used to estimate flow statistics for ungaged sites; (3) a GIS program that allows locating sites of interest in the user interface, delineates drainage basins and measures basin characteristics; and (4) a database of geospatial datasets needed for the GIS program to work. Geospatial datasets include digital representations of the land surface (digital elevation models and derivative products), water features, historic climate data, soils information, and land-use information.

Implementation Strategy

Although StreamStats has a single, national user interface from which information for USGS data-collection stations can be obtained anywhere nationally, the functionality that allows for obtaining basin delineations, basin characteristics, and estimates of streamflow statistics for user-selected ungaged sites was implemented for each state individually. Implementation requires (1) the processing of numerous geospatial datasets, (2) assembling information for streamgages and importing it into a database, (3) importing regression equations needed to estimate streamflow statistics for ungaged sites into a database, (4) organizing and installing the geospatial datasets on a server, (5) coding to specify how each state will function and the datasets that it will use, and (6) evaluation to assure that StreamStats is producing accurate information. The needed work is coordinated among a national team, which sets up the applications and maintains it after implementation, and scientists in the USGS Water Science Centers (WSC), who primarily prepare the needed data and evaluate the application before it is released to the public. The work done by WSC scientists generally is accomplished through cooperative agreements with other federal, state, or local agencies, in which those agencies pay for at least half of the cost of the work. As a result, some states have not been implemented in StreamStats because the local WSCs have been unable to obtain cooperative funding agreements with other agencies to accomplish the needed work.

Functionality

StreamStats includes tools for (1) selecting one of several base maps and changing the map scale and center of the map (zoom in or out and pan) by various methods to select locations where information is desired, (2) getting information for USGS data-collection stations, (3) getting information for user-selected ungaged sites on streams, including drainage-basin delineations, basin characteristics, and estimates of streamflow statistics, and (4) obtaining geographic information, such as measuring distances and obtaining land-surface elevation profiles. The StreamStats users' manual provides complete descriptions of the available tools and how to use them.

Streamgages

USGS streamgage locations are identified on the map by triangles in different colors, as defined in the map legend, depending on the types of data collected there. Clicking on a streamgage location will cause a pop-up text box to appear with information about the streamgage and links to additional information. Users can obtain descriptive information about the stations, such as station name and number, type of data collected, and period of record, as well as previously published basin characteristics and streamflow statistics, along with citations to the reports from which the information was taken. The information is provided in tabular format on a pop-up web page, which can be printed or saved in machine-readable format. The cited reports provide descriptions of how the basin characteristics and streamflow statistics were computed. A link is also provided to the NWIS-Web page for the station, which provides access to all USGS data that were collected at the station.

User-Selected Sites

StreamStats can delineate the drainage basins for user-selected sites on streams. The map must be at zoom level 15 or greater before a gridded representation of the stream network will appear and users can select locations of interest. After a delineation has completed, users are given options to clear, edit, or download the basin boundary, or to continue on to compute a variety of physical and climatic characteristics for the basin or obtain estimates of streamflow statistics for the selected site. StreamStats provides prompts and other feedback as users progress through this process.

StreamStats determines drainage-basin boundaries by use of digital elevation data obtained from digital elevation data from the USGS 3D Elevation Program (3DEP). In most cases, the elevation data have been specially processed so that the elevation data conforms to the digital stream channels depicted in the high-resolution version of the National Hydrography Dataset (NHD) and to the drainage-basin boundaries of the Watershed Boundary Dataset WBD. This processing results in drainage-basin boundary delineations that generally are superior to delineations that can be obtained directly from the 3DEP. Still, users should be aware that use of any digital elevation data to delineate drainage boundaries can lead to errors; especially in flat areas. Users should check delineations carefully and use the *Edit Basin* tool to correct any errors, if necessary.

After the user indicates that the boundary is correct, StreamStats can compute basin characteristics (such as drainage area, stream slope, and mean annual precipitation) and estimate streamflow statistics (such as the 1-percent flood, the mean flow, and the 7-day, 10-year low flow) for the site. The basin characteristics and streamflow statistics that are available vary widely among states and individual sites. Unless otherwise noted on a state's *State/Regional Info* page, estimates obtained for ungaged sites assume natural flow conditions at the site.

StreamStats provides estimates of various streamflow statistics for user-selected sites by solving equations that were developed through a process known as regionalization. This process involves use of regression analysis to relate streamflow statistics computed for a group of selected streamgages within or near a region of study (usually a state) to basin characteristics measured for the stations. Once the equations have been published, users can enter basin characteristics measured for ungaged sites into the equations to obtain estimates of the streamflow statistics. Equations generally were developed by the USGS separately for each state through cost-sharing agreements with other Federal, state, or local agencies. As a result, the equations that are available for a given state vary based on which statistics the other agencies need for regulatory, planning, or other purposes.

The USGS National Streamflow Statistics Program (NSS) is a desktop program that contains all of the USGS-developed regression equations for estimating flood-frequency statistics in the Nation, plus equations for estimating other streamflow statistics in many states. NSS relies on manual entry of the basin characteristics used as explanatory variables in the equations and then solves the equations to estimate the statistics. The NSS program has been linked through a background process to StreamStats in which StreamStats provides the needed basin characteristics to NSS for an ungaged site and then NSS estimates the streamflow statistics, sends them back to StreamStats, and then StreamStats presents the statistics and the basin characteristics to the user. All of the equations in NSS, and limitations for their use, are documented in reports that can be accessed through links to each individual state from the NSS Web site. The StreamStats application also provides these links on the *State/Regional Info* tab that can be accessed after selecting a state and clicking on the About button in the black banner above the map in the StreamStats user interface.

A previous version of StreamStats included an additional method for estimating streamflow statistics for user-selected sites on streams where there was an upstream or downstream streamgage with a drainage area that was within 0.5 and 1.5 times the drainage area for selected site. In these cases, the flow per unit area of the statistics for the streamgage would be multiplied by the drainage area for the selected site to estimate the statistics for the selected site. This method is documented on page 9 of a report by Ries (2006). The tool that implements this method of estimation was not available at the time of the release of StreamStats version 4, but efforts are underway to restore it.

Exploration Tools

The Exploration Tools button with the toolbox cartoon image is near the top left of the map. Clicking on the button reveals additional buttons for querying information for streamgages, measuring distances between two or more points on the map, and showing your current location on the map (primarily for mobile users). Several additional tools are in development that will rely on searching along the stream network upstream or downstream from user-selected sites to identify stream reaches and water-related activities along the streams, such as dams and point discharges, and obtain information about those activities.

Outputs

Outputs for streamgages and ungaged sites are provided in pop-up pages within the user interface. At the top of outputs for ungaged sites are text boxes in which users can enter a report title and comments, if desired, and a map of the delineated basin. Below that are tables of basin characteristics and estimated flow statistics for the selected site. Outputs for streamgages include a table of descriptive information, followed by tables of available basin characteristics and streamflow statistics. Each type of output provides short names of the basin characteristics and streamflow statistics. Definitions are provided on the Basin Characteristics Definitions and Streamflow Statistics Definitions pages. The StreamStats users' manual provides complete descriptions of the outputs, which can be printed or saved in machine-readable format. In addition, the delineated boundaries for ungaged sites can be downloaded and saved in a variety of formats with computed basin characteristics and streamflow statistics included as attributes.

SCS GRAPHICAL PEAK DISCHARGE METHOD

For many peak discharge estimation methods, the input includes variables to reflect the size of the contributing area, the amount of rainfall, the potential watershed storage, and the time-area distribution of the watershed. These are often translated into input variables such as the drainage area, the depth of rainfall, an index reflecting land use and soil type, and the time of concentration. The SCS graphical peak discharge method is typical of many peak discharge methods that are based on input such as that described.

Runoff Depth Estimation

The volume of storm runoff can depend on a number of factors. Certainly, the volume of rainfall will be an important factor. For very large watersheds, the volume of runoff from one storm event may depend on rainfall that occurred during previous storm events. However, when using the design storm approach, the assumption of storm independence is quite common.

In addition to rainfall, other factors affect the volume of runoff. A common assumption in hydrologic modeling is that the rainfall available for runoff is separated into three parts: direct (or storm) runoff, initial abstraction, and losses. Factors that affect the split between losses and direct runoff include the volume of rainfall, land cover and use, soil type, and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage.

In developing the SCS rainfall-runoff relationship, the total rainfall was separated into three components: direct runoff (Q), actual retention (F), and the initial abstraction (I_a). The retention F was assumed to be a function of the depths of rainfall and runoff and the initial abstraction. The development of the equation yielded:

$$Q = [(P - I_a)^2]/[(P - I_a) + S] \qquad \text{Equation 19}$$

Where

P = depth of precipitation, inches
I_a = initial abstraction, inches
S = maximum potential retention, inches
Q = depth of direct runoff, inches

Given Equation 19, two unknowns need to be estimated, S and I_a. The retention S should be a function of the following five factors: land use, interception, infiltration, depression storage, and antecedent moisture.

Empirical evidence resulted in the following equation for estimating the initial abstraction:

$Ia = 0.2\ S$ Equation 20

If the five factors above affect S, they also affect Ia. Substituting Equation 20 into Equation 19 yields the following equation, which contains the single unknown S:

$Q = [(P - 0.2\ S)^2]/[(P + 0.8\ S]$ Equation 21

Equation 21 represents the basic equation for computing the runoff depth, Q, for a given rainfall depth, P. It is worthwhile noting that while Q and P have units of depth, Q and P reflect volumes and are often referred to as volumes because it is usually assumed that the same depths occurred over the entire watershed.

Additional empirical analyses were made to estimate the value of S. The studies found that S was related to soil type, land cover, and the hydrologic condition of the watershed. These are represented by the runoff curve number (CN), which is used to estimate S by:

$S = (1000/CN) - 10$ Equation 22

Where

CN = index that represents the combination of hydrologic soil group and a land use and treatment class

Empirical analyses suggested that the CN was a function of three factors: soil group, the cover complex, and antecedent moisture conditions.

Soil Group Classification

SCS developed a soil classification system that consists of four groups, which are identified by the letters A, B, C, and D. Soil characteristics that are associated with each group are as follows:

Group A: deep sand, deep loess; aggregated silts

Group B: shallow loess; sandy loam

Group C: clay loams; shallow sandy loam; soils low in organic content; soils usually high in clay

Group D: soils that swell significantly when wet; heavy plastic clays; certain saline soils

The SCS soil group can be identified at a site using either soil characteristics or county soil surveys. The soil characteristics associated with each group are listed above and provide one means of identifying the SCS soil group. County soil surveys, which are made available by Soil Conservation Districts, give detailed descriptions of the soils at locations within a county; these surveys are usually the better means of identifying the soil group. Many of the more recent reports actually categorize the soils into these four groups.

Cover Complex Classification

The SCS cover complex classification consists of three factors: land use, treatment or practice, and hydrologic condition. Many different land uses are identified in the tables for estimating runoff curve numbers. Agricultural land uses are often subdivided by treatment or practices, such as contoured or straight row; this separation reflects the different hydrologic runoff potential that is associated with variation in land treatment. The hydrologic condition reflects the level of land management; it is separated into three classes: poor, fair, and good. Not all of the land uses are separated by treatment or condition.

Curve Number Tables

Table 5.4 shows the SCS CN values for the different land uses, treatments, and hydrologic conditions; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, the CN will be 66.

Table .4. Runoff Curve Numbers
(average watershed condition, $I_a = 0.2S$)(After: SCS, 1986)

Cover Type		Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Fully developed urban areas[a] (vegetation established)					
Lawns, open spaces, parks, golf courses, cemeteries, etc.					
Good condition; grass cover on 75% or more of the area		39	61	74	80
Fair condition; grass cover on 50% to 75% of the area		49	69	79	84
Poor condition; grass cover on 50% or less of the area		68	79	86	89
Paved parking lots, roofs, driveways, etc. (excl. right-of- way)		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers (excl. right-of-way)		98	98	98	98
Gravel (incl. right-of-way)		76	85	89	91
Dirt (incl. right-of-way)		72	82	87	89
Paved with open ditches (incl. right-of-way)		83	89	92	93
	Average % impervious[b]				
Commercial and business areas	85	89	92	94	95
Industrial districts	72	81	88	91	93
Row houses, town houses, and residential with lots sizes 0.05 ha or less (0.12 acres or less)	65	77	85	90	92
Residential: average lot size					
0.1 ha (0.25 acres)	38	61	75	83	87
0.135 ha (0.33 acres)	30	57	72	81	86
0.2 ha (0.5 acres)	25	54	70	80	85
0.4 ha (1.0 acres)	20	51	68	79	84
0.8 ha (2.0 acres)	12	46	65	77	82
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 25- to 50-mm (1- to 2-in) sand or gravel mulch and basin borders)		96	96	96	96
Developing urban areas[c] (no vegetation established) Newly graded area		77	86	91	94

Table .4. Runoff Curve Numbers (Cont'd)

Cover Type		Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
			A	B	C	D
Cultivated Agricultural Land: Fallow						
Straight row or bare soil			77	86	91	94
Conservation tillage		Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and tillage	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
		Good	62	71	78	81
	Contoured and terraces and conservation tillage	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and tillage	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
		Good	59	70	78	81
	Contoured and terraces and conservation tillage	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadows[e]	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
		Good	57	67	76	80
Noncultivated agricultural land						
Pasture or range	No Mechanical treatment[f]	Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
Meadow - continuous grass, protected from grazing and generally mowed for hay			30	58	71	78

Table 4. Runoff Curve Numbers (Cont'd)

Cover Type	Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group A	B	C	D
Forestland - grass or orchards - evergreen or Deciduous	Poor	55	73	82	86
	Fair	44	65	76	82
	Good	32	58	72	79
Brush - brush-weed-grass mixture with brush the major element[e]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[f]	48	65	73
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[f]	55	70	77
Woods - grass combination (orchard or tree farm)[h]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Farmsteads		59	74	82	86
Forest-range					
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
Sage-grass	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

a For land uses with impervious areas, curve numbers are computed assuming that 100 percent of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a CN of 98.

b Includes paved streets.

c Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area CN, the composite CN can be computed for any degree of development.

d For conservation tillage poor hydrologic condition, 5 to 20 percent of the surface is covered with residue (less than 760 lbs/acre row crops or 310 lbs/acre small grain). For conservation tillage good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 760 lbs/acre row crops or 310 lbs/acre small grain).

e Close-drilled or broadcast.
For noncultivated agricultural land:
Poor hydrologic condition has less than 25 percent ground cover density.
Fair hydrologic condition has between 25 and 50 percent ground cover density. Good hydrologic condition has more than 50 percent ground cover density.
For forest-range.
Poor hydrologic condition has less than 30 percent ground cover density.
Fair hydrologic condition has between 30 and 70 percent ground cover density. Good hydrologic condition has more than 70 percent ground cover density.

f Actual curve number is less than 30: use CN = 30 for runoff computations.

g CNs shown were computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

h Poor: < 50 percent ground cover.
Fair: 50 to 75 percent ground cover.
Good: > 75 percent ground cover.

i Poor: < 50 percent ground cover or heavily grazed with no mulch.
Fair: 50 to 75 percent ground cover and not heavily grazed.
Good: > 75 percent ground cover and lightly or only occasionally grazed.

Estimation of CN Values for Urban Land Uses

The CN table (Table 4) includes CN values for a number of urban land uses. For each of these, the CN is based on a specific percentage of imperviousness. For example, the CN values for commercial land use are based on an imperviousness of 85 percent. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus CN values of 39, 61, 74, and 80 are used for hydrologic soil groups A, B, C, and D, respectively. These are the same CN values for pasture in good condition. Thus the following equation can be used to compute a weighted CN:

$$CN_w = CN_p (1 - f) + f (98) \qquad \text{Equation 23}$$

in which f is the fraction (not percentage) of imperviousness. To show the use of Equation 23, the CN values for commercial land use with 85 percent imperviousness are:

A soil: $39(0.15) + 98(0.85) = 89$
B soil: $61(0.15) + 98(0.85) = 92$
C soil: $74(0.15) + 98(0.85) = 94$
D soil: $80(0.15) + 98(0.85) = 95$

These are the same values shown in Table 4.

Equation 23 can be placed in graphical form (see Figure 2a). By entering with the percentage of imperviousness on the vertical axis at the center of the figure and moving horizontally to the pervious area CN, the composite CN can be read. The examples above for commercial land use can be used to illustrate the use of Figure 2a for 85 percent imperviousness. For a commercial land area with 60 percent imperviousness of a B soil, the composite CN would be:

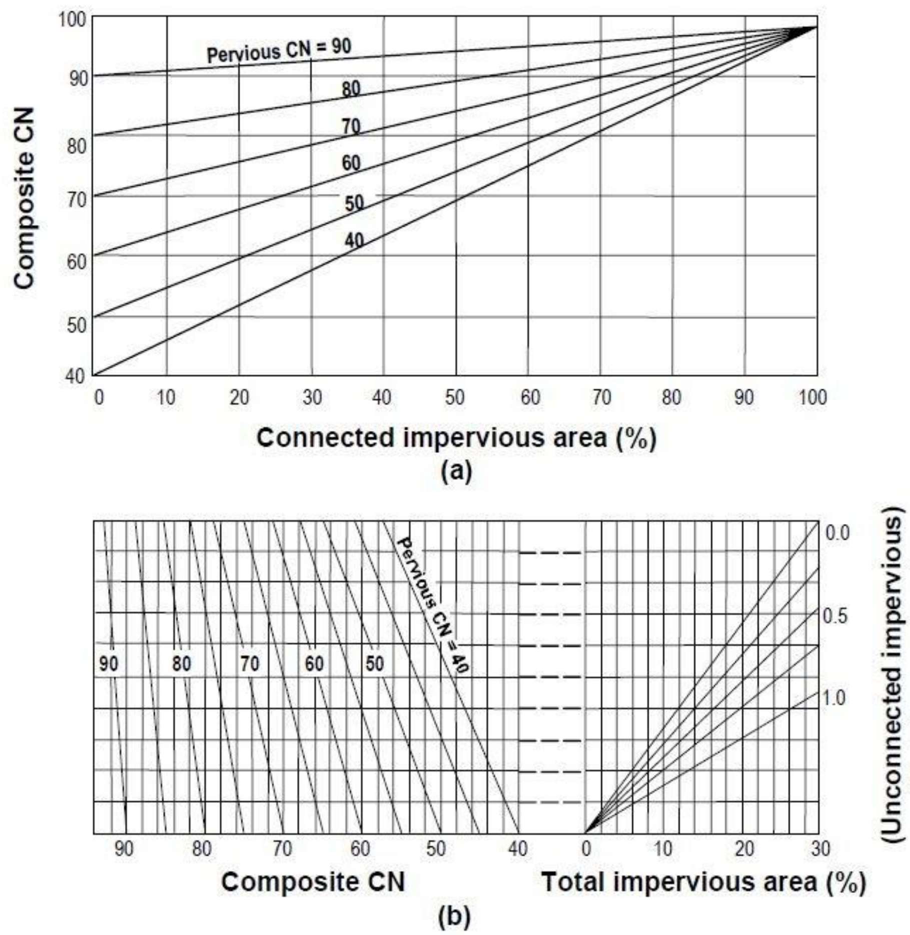

Figure 3. Composite curve number estimation:
(a) all imperviousness area connected to storm drains
(b) some imperviousness area not connected to storm drain

Figure 3. Composite curve number estimation:
(a) all imperviousness area connected to storm drains
(b) some imperviousness area not connected to storm drain

Effect of Unconnected Impervious Area on Curve Numbers

Many local drainage policies are requiring runoff that occurs from certain types of impervious land cover (i.e., rooftops, driveways, patios) to be directed to pervious surfaces rather than being connected to storm drain systems. Such a policy is based on the belief that disconnecting these impervious areas will require smaller and less costly drainage systems and lead both to increased ground water recharge and to improvements in water quality. If disconnecting some impervious surfaces will reduce both the peak runoff rates and volumes of direct flood runoff, credit should be given in the design of drainage systems. The effect of disconnecting impervious surfaces on runoff rates and volumes can be accounted for by modifying the CN.

There are three variables involved in the adjustment: the pervious area CN, the percentage of impervious area, and the percentage of the imperviousness that is unconnected. Because Figure 3a for computing composite CN values is based on the pervious area CN and the percentage of imperviousness, a correction factor was developed to compute the composite CN. The correction is a function of the percentage of unconnected imperviousness, which is shown in Figure 3b. The use of the correction is limited to drainage areas having percentages of imperviousness that are less than 30 percent.

As an alternative to Figure 3b, the composite curve number (CN_C) can be computed by:

$CN_c = CN_p + (P_i/100)\ (98 - CN_p)\ (1 - 0.5\ R)$ for $P_i \leq 30\%$ Equation 24

Where,
P_i = percent imperviousness
R = ratio of unconnected impervious area to total pervious area

Equation 24, like Figure 3b, is limited to cases where the total imperviousness (P_i) is less than 30 percent.

I_a/P Parameter

I_a/P is a parameter that is necessary to estimate peak discharge rates. I_a denotes the initial abstraction, and P is the 24-hour rainfall depth for a selected return period. For a given 24-hour rainfall distribution, I_a/P represents the fraction of rainfall that must occur before runoff begins.

Peak Discharge Estimation

The following equation can be used to compute a peak discharge with the SCS method:

$q_p = q_u\ A\ Q$ Equation 25

where,
q_p = peak discharge, ft^3/s)
q_u = unit peak discharge, $ft^3/s/\ mi^2/in$
A = drainage area, mi^2
Q = depth of runoff, in.

The unit peak discharge is obtained from the following equation, which requires the time of concentration (t_c) in hours and the initial abstraction/rainfall (I_a/P) ratio as input:

q_u = 10 $C^{0 + C1 \log tc + C2 [\log (tc)]^2}$ Equation 26

where,

C_0, C_1 and C_2 – regression coefficient given in Table 5 for various I_a/P ratios

The runoff depth (Q) is obtained from Equation 21 and is a function of the depth of rainfall P
and the runoff CN. The I_a/P ratio is obtained directly from Equation 20.

Table .5. Coefficients for SCS Peak Discharge Method

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	-0.57005	-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	-0.11508
	0.50	2.17772	-0.36803	-0.09525

The peak discharge obtained from Equation 26 assumes that the topography is such that surface flow into ditches, drains, and streams is relatively unimpeded. Where ponding or wetland areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak discharge rate should be reduced to reflect this condition of increased storage. Values of the pond and swamp adjustment factor (F_p) are provided in Table 6. The adjustment factor values in Table 6 are a function of the percent of the total watershed area in ponds and wetlands. If the watershed includes significant portions of pond and wetland storage, the peak discharge of Equation 25 can be adjusted using the following:

$q_a = q_p F_p$ Equation 27

where
q_a = adjusted peak discharge, ft^3/s.

Table 6. Adjustment Factor (F_p) for Pond and Wetland Areas

Area of Pond and Wetland (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

The SCS method has a number of limitations. When these conditions are not met, the accuracy of estimated peak discharges decreases. The method should be used on watersheds that are homogeneous in CN; where parts of the watershed have CNs that differ by 5, the watershed should be subdivided and analyzed using a hydrograph method, such as TR-20 (SCS, 1984). The SCS method should be used only when the CN is 50 or greater and the t_c is greater than 0.1 hour and less than 10 hours. Also, the computed value of I_a/P should be between 0.1 and 0.5. The method should be used only when the watershed has one main channel or when there are two main channels that have nearly equal times of concentration; otherwise, a hydrograph method should be used. Other methods should also be used when channel or reservoir routing is required, or where watershed storage is either greater than 5 percent or located on the flow path used to compute the t_c.

Example 4. A small watershed (43.5 acres) is being developed and will include the following land uses: 26.2 ac of residential (0.25 ac lots), 12.8 ac of residential (0.5 ac lots), 3.0 ac of commercial property (85 percent impervious), and 1.5 ac of woodland. The development will necessitate upgrading of the drainage of a local roadway at the outlet of the watershed. The peak discharge for a 10-year return period is determined using the SCS graphical method.

The weighted CN is computed using the CN values of Table 4:

Land Cover	Lot Size, ac	Soil Group	CN	Area, ac	A*CN
Residential	0.5	B	70	12.8	896
Residential	0.25	B	75	11.4	855
Residential	0.25	C	83	14.8	1228
Commercial (85% Impervious)		C	94	3.0	282
Woodland (Good Condition)		C	70	1.5	105
Total				43.5	3366

The weighted CN is:

Variable	Value in English Units
$CN_w = (\sum A * CN)/ (\sum A)$	= (3366/43.5) = 77.4 (use 77)

The time of concentration is computed using the velocity method for conditions along the principal flowpath:

Conveyance Type	Slope (%)	K	Length (ft)	V (ft/s)	T_t (hr)
Woodland (overland)	2.3	0.152	82	0.76	0.03
Grassed waterway	2.1	0.457	902	2.19	0.12
Grassed waterway	1.8	0.457	820	2.02	0.11
Concrete-lined channel	1.8		164	15.1	0.00
			1968		0.26

The velocity was computed for the concrete-lined channel using Manning's equation, with n = 0.013 and hydraulic radius of 1 ft. The sum of the travel times for the principal flowpath is 0.26 hours.

The rainfall depth is obtained from an IDF curve for the locality using a storm duration of 24 hours and a 10-year return period. (Note that the t_c is not used to find the rainfall depth when using the SCS graphical method. A storm duration of 24 hours is used.) For this example, a 10-year rainfall depth of 4.8 in is assumed. For a CN of 77, S equals 3.0 in and I_a equals 0.6 in. Thus, I_a/P is 0.12. The rainfall depth is computed with Equation 21:

Variable	Value in English Units
$Q = [(P – 0.2 S)^2]/(P + 0.8 S)$	$= [(4.8 – 0.2 (3.0))^2]/(4.8 + 0.8 (3.0)) = 2.45$ in

The unit peak discharge is computed with Equation 26 by interpolating c_0, c_1 and c_2 from Table 5 using a type II distribution. The peak discharge is also calculated as follows.

Variable	Value in English Units
$q_u = 10\ C^{0 + C1 \log tc + C2 [\log (tc)]^2}$	$= 10^{2.85}$ = 708 $ft^3/s/mi^2/in$
$q_p = q_u\ A\ Q$	= 708 (0.068 mi^2) (2.46 in) = 120 ft^3/s

RATIONAL METHOD

One of the most commonly used equations for the calculation of peak discharges from small areas is the rational formula. The rational formula is given as:

$Q = CiA$ Equation 28

Where,

Q = the peak flow, ft^3/s

i = the rainfall intensity for the design storm, in/hr

A = the drainage area, acres

C = dimensionless runoff coefficient assumed to be a function of the cover of the watershed and often the frequency of the flood being estimated

Assumptions

The assumptions in the rational formula are as follows:

1. The drainage area should be smaller than 200 acres.

2. The peak discharge occurs when the entire watershed is contributing.

3. A storm that has a duration equal to t_c produces the highest peak discharge for this frequency.

4. The rainfall intensity is uniform over a storm time duration equal to the time of concentration t_c. The time of concentration is the time required for water to travel from the hydrologically most remote point of the basin to the outlet or point of interest.

5. The frequency of the computed peak flow is equal to the frequency of the rainfall intensity. In other words, the 10-year rainfall intensity, i, is assumed to produce the 10-year peak discharge.

Estimating Input Requirements

The runoff coefficient, C, is a function of ground cover. Some tables of C provide for variation due to slope, soil, and the return period of the design discharge. Actually, C is a volumetric coefficient that relates the peak discharge to the "theoretical peak" or 100 percent runoff, occurring when runoff matches the net rain rate. Hence C is also a function of infiltration and other hydrologic abstractions. Some typical values of C for the rational formula are given in Table 5.7. Should the basin contain varying amounts of different covers, a weighted runoff coefficient for the entire basin can be determined as:

Weighted $C = (\sum C_i A_i)/A$ Equation 29

Where

C_i = runoff coefficient for cover type I that covers area A_i
A = total area

Check for Critical Design Condition

When the rational method is used to design multiple drainage elements (i.e. inlets and pipes), the design process proceeds from upstream to downstream. For each design element, a time of concentration is computed, the corresponding intensity determined, and the peak flow computed. For pipes that drain multiple flow paths, the longest time of concentration from all of the contributing areas must be determined. If upstream pipes exist, the travel times in these pipes must also be included in the calculation of time of concentration.

In most cases, especially as computations proceed downstream, the contributing area with the longer time of concentration also contributes the greatest flow. Taking the case of two contributing areas, as shown in Figure 3a, the longest time of concentration of the two areas is used to determine the time of concentration for the combined area. When the rainfall intensity corresponding to this time of concentration is applied to the rational equation, as shown below, for the combined area and runoff coefficient, the appropriate design discharge, Q, results.

$$Q = (C_1 A_1 + C_2 A_2)\ i_1 \quad \text{Equation 30}$$

However, it may be possible for the larger contributing flows to be generated from the contributing area with a shorter time of concentration. If this occurs, it is also possible that, if the longer time of concentration is applied to the combined drainage area, the resulting design flow would be an underestimate. Therefore, a check for a critical design condition must be made.

$$Q' = [C_1 A_1 (t_2/t_1) + C_2 A_2]\ i_1 \quad \text{Equation 31}$$

Where,

Q' = design check discharge
t_1 = time of concentration for area 1
t_2 = time of concentration for area 2

Table 7. Runoff Coefficients for Rational Formula (ASCE, 1960)

Type of Drainage Area	Runoff Coefficient
Business:	
Downtown area	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.25-0.40
Apartment dwelling areas	0.50-0.70
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Lawns:	
Sandy soil, flat, < 2%	0.05-0.10
Sandy soil, average, 2 to 7%	0.10-0.15
Sandy soil, steep, > 7%	0.15-0.20
Heavy soil, flat, < 2%	0.13-0.17
Heavy soil, average 2 to 7%	0.18-0.22
Heavy soil, steep, > 7%	0.25-0.35
Streets:	
Asphalt	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and walks	0.75-0.85
Roofs	0.75-0.95

If Q’ > Q, Q’ should be used for design; otherwise Q should be used. Equation 31 uses the rainfall intensity for the contributing area with the shorter time of concentration (area 2) and reduces the contribution of area 1 by the ratio of the times of concentration. This ratio approximates the fraction of the area that would contribute within the shorter duration. This is equivalent to reducing the contributing area as shown by the dashed line in Figure 4.

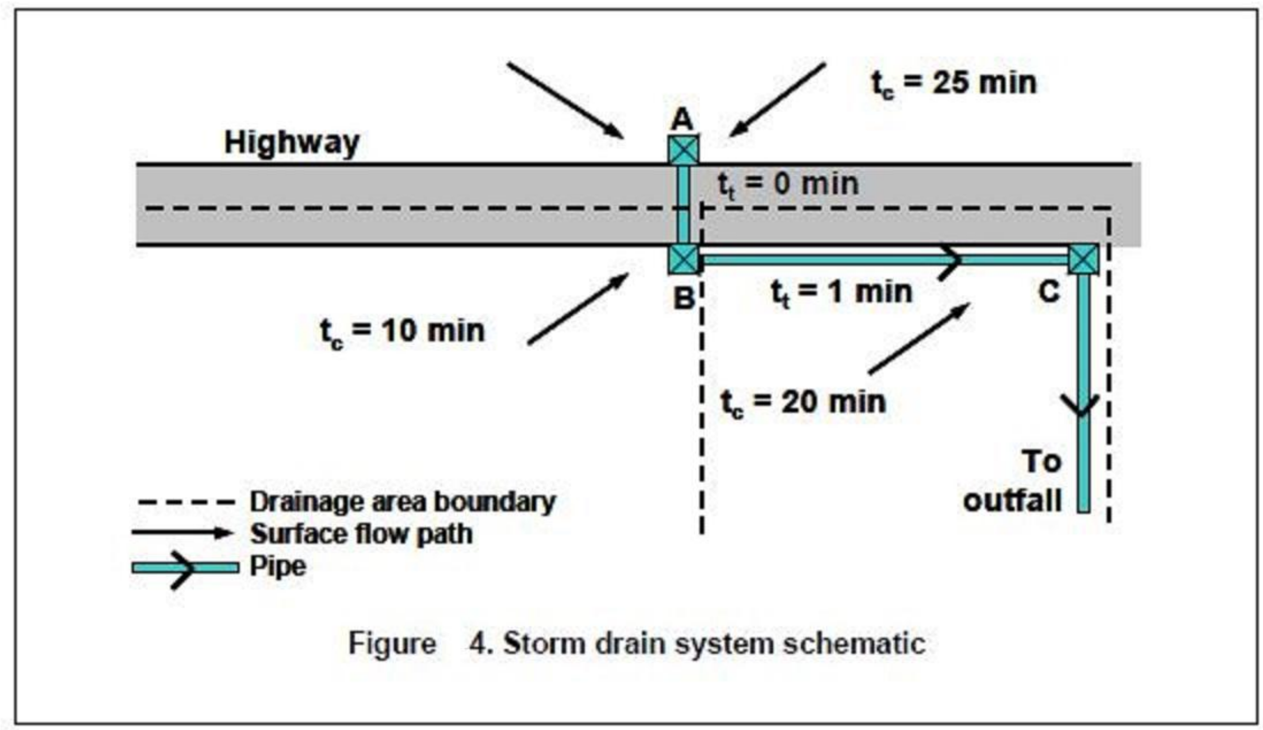

Figure 4. Storm drain system schematic

Example 5. A flooding problem exists along a farm road near Memphis, Tennessee. A low-water crossing is to be replaced by a culvert installation to improve road safety during rainstorms. The drainage area above the crossing is 108 acres. The return period of the design storm is to be 25 years as determined by local authorities. The engineer must determine the maximum discharge that the culvert must pass for the indicated design storm.

The current land use consists of 53.9 acres of parkland, 3.7 acres of commercial property that is 100 percent impervious, and 50.4 acres of single-family residential housing. The principal flow path includes 295 ft of short grass at 2 percent slope, 985 ft of grassed waterway at 2 percent slope, and 2,130 ft of grassed waterway at 1 percent slope. The following steps are used to compute the peak discharge with the rational method:

1. Compute a Weighted Runoff Coefficient: The tabular summary below uses runoff coefficients from Table 7. The average value is used for the parkland and the residential areas, but the highest value is used for the commercial property because it is completely impervious.

Description	C Value	Area (acres)	C_iA_i
Park	0.20	53.9	10.8
Commercial (100% impervious)	0.95	3.7	3.5
Single-family	0.40	50.4	20.2
Total		108.0	34.5

Equation 29 is used to compute the weighted C:

Variable	Value in English Units
Weighted C = ($\sum C_i A_i$)/A	(34.5)/108.0 = 0.32

2. Intensity: The 25-year intensity is taken from an intensity-duration-frequency curve for Memphis. To obtain the intensity, the time of concentration, t_c, must first be estimated. In this example, the velocity method for t_c is used to compute t_c:

Flow Path	Slope (%)	Length (ft)	Velocity (ft/s)
Overland (Short grass)	2	295	1.0
Grassed waterway	2	985	2.1
Grassed waterway	1	2,130	1.5

The time of concentration is estimated as:

Variable	Value in English Units
$T_c = \sum (L/V)$	= (295 ft/1.0 ft/s) + (985 ft/2.1 ft/s) + (2130 ft/1.5 ft/s) = 2180 sec = 36 min

The intensity is obtained from the IDF curve for the locality using a storm duration equal to the time of concentration:
$i = 3.35$in/hr

3. Area (A): Total area of drainage basin, A = 108 acres

4. Peak Discharge (Q):

Q = C i A = (0.32) (3.35) (108) = 116 ft^3/s

INDEX FLOOD METHOD

Other methods exist for determining peak flows for various exceedence frequencies using regional methods where no data are available. The USGS index-flood method is representative of this group.

Procedure for Analysis

The index-flood method of regional analysis described by Dalrymple (1960) was used extensively in the 1960s and early 1970s. This method utilizes statistical analyses of data at meteorologically and hydrologically similar gages to develop a flood frequency curve at an ungaged site. There are two parts to the index-flood method. The first consists of developing the basic dimensionless ratio of a specified frequency flow to the index flow (usually the mean annual flood) and the second involves developing the relation between the drainage basin characteristics (usually the drainage area) and the mean annual flood.

The following steps are used to develop a regional flood frequency curve by the index-flood method:

1. Tabulate annual peak floods for all gages within the hydrologically similar region.

2. Select the base period of record. This is usually taken as the longest period of record.

3. Estimate floods for missing years by correlation with other data.

4. Assign an order to all floods (actual and estimated) at each station, compute the plotting positions, and compute and plot frequency curves using the best standard distribution fit for each gage.

5. Determine the mean annual flood for each gage as the discharge with a return period of 2.33 years. This is a graphical mean, which is more stable than the arithmetic mean, and its value is not affected as much by the inclusion or exclusion of major floods. It also gives a greater weight to the median floods than to the extreme floods where sampling errors may be larger. In some cases, the 2- or 10-year flood is used as the index flood.

6. Test the data for homogeneity. This is accomplished in the following manner.

a. For each gage, compute the ratio of the flood with a 10-year return period, Q_{10}, to the station mean, $Q_{2.33}$. (Both of these values are obtained from the frequency analysis.)

b. Compute the arithmetic average of the ratio $Q_{10}/Q_{2.33}$ for all the gages considered.
c. For each gage, compute $Q_{2.33}\ (Q_{10}/Q_{2.33})_{avg}$ and the corresponding return period.
d. Plot the values of return period obtained in step c against the effective length of record, L_E, for each gage.

e. Test for homogeneity by also plotting on this graph, envelope curves determined from Table 8, taken from Dalrymple (1960). This table gives the upper and lower limits, T_u and T_L, as a function of the effective length of record. (Table 5.8 applies only to homogeneity tests of the 10-year floods.) Return periods that fail this homogeneity test should be eliminated from the regional analysis.

7. Using actual flood data, compute the ratio of each flood to the index flood, $Q_{2.33}$, for each record.

8. Compute the median flood ratios of the stations retained in the regional analysis for each rank or order m, and compute the corresponding return period by the Weibull formula, $T_r = (n+1)/m$. (It is suggested that the median ratio be determined after eliminating the highest and lowest $Q/Q_{2.33}$ values for each ordered series of data.)

9. Plot the median-flood ratio against the return period on probability paper.

10. Plot the logarithm of the mean annual flood for each gage, $Q_{2.33}$ against the logarithm of the corresponding drainage area. This curve should be nearly a straight line.

11. Determine the flood frequency curve for any stream site in the watershed as follows:

a. Determine the drainage area above the site.
b. From Step 10, determine the value of $Q_{2.33}$.
c. For selected return periods, multiply the median-flood ratio in step 9 by the value of $Q_{2.33}$ from Step 11b.

d. Plot the regional frequency curve.

Table 8. Upper and Lower Limit Coordinates of Envelope Curve for Homogeneity Test (Dalrymple, 1960)

Effective Length of Record, L_E (yrs)	Return Period Limits, T_r (yrs)	
	Upper Limit	Lower Limit
5	160	1.2
10	70	1.85
20	40	2.8
50	24	4.4
100	18	5.6

Example problems illustrating the index-flood method are contained in Dalrymple (1960), Sanders (1980), and numerous hydrology textbooks.

<u>Other Considerations</u>

As pointed out by Benson (1962), the index-flood method has some limitations that affect its reliability. The most significant is that there may be large differences in the index or mean annual floods throughout a region. This can lead to considerable variations in the various flood ratios even for watersheds of comparable size. Another shortcoming of the method is that homogeneity is established at the 10-year level, whereas at the higher levels the test may not be sustained. Still another deficiency pointed out by Benson is that all sizes of drainage areas (except the very largest) are included in the index-flood regional analysis. The larger the drainage area, the flatter the frequency curve will be. This effect is most noticeable at the higher return periods.

With the development of regional regression equations for peak-flow in most states, there is only limited application of the index-flood method today. It is used primarily as a check on other solution techniques and for those situations where other techniques are inapplicable or not available.

PEAK DISCHARGE ENVELOPE CURVES

Design storms are hypothetical constructs and have never occurred. Many design engineers like to have some assurance that a design peak discharge is unlikely to occur over the design life of a project. This creates an interest in comparing the design peak to actual peaks of record.

Crippen and Bue (1977) developed envelope curves for the conterminous United States, with 17 regions delineated as shown in Figure 5. Maximum flood flow data from 883 sites that have drainage areas less than 10,000 mi^2 were plotted versus drainage area and upper envelope curves constructed. The curves for the 17 regions were fit to the following logarithmic polynomial model:

$$q_{envlpe} = K_1 A^{K2} [L + A^{0.5}]^{K3} \qquad \text{Equation 32}$$

where,

q_{envlpe} = maximum flood flow envelope, ft^3/s
L = length constant, 5.0 mi
A = drainage area, mi^2

Table 9 gives the values of the coefficients (K_1, K_2, and K_3 of Equation 32) and the upper limit on the drainage area for each region. The curves are valid for drainage areas greater than 0.1 mi^2). Crippen and Bue did not assign an exceedence probability to the flood flows used to fit the curves, so a probability cannot be given to values estimated from the curve.

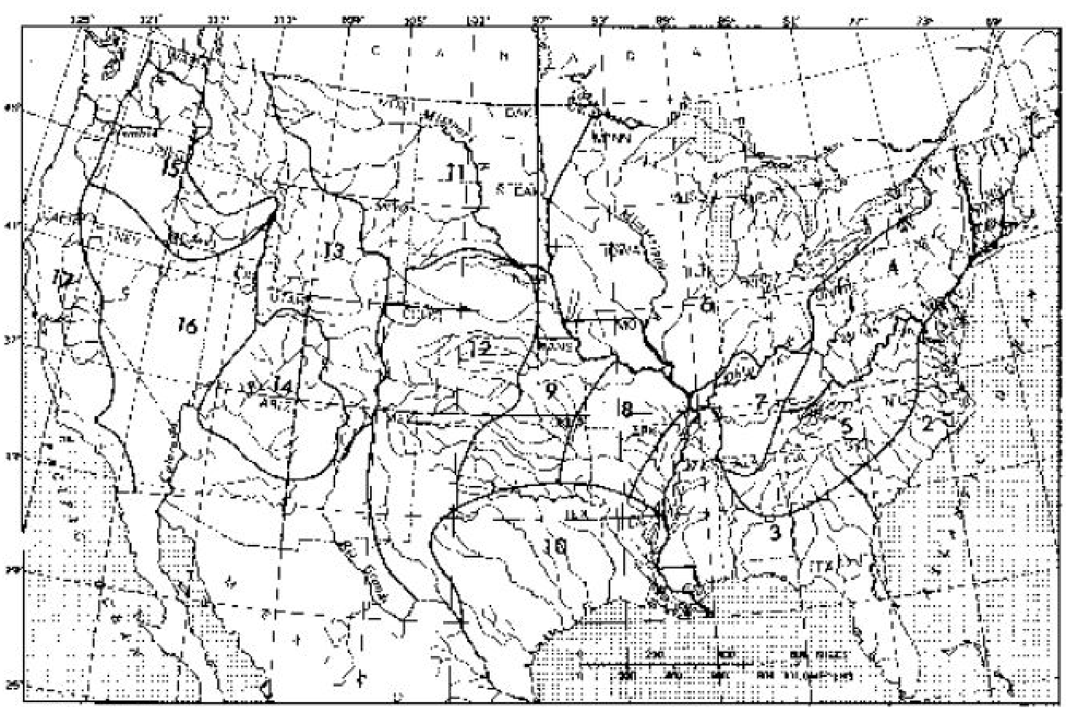

Figure 5. Map of the conterminous United States showing flood-region boundaries

Table 9. Coefficients for Peak Discharge Envelope Curves

Region	Upper limit (mi^2)	Coefficients		
		K_1	K_2	K_3
1	10,000	23200	0.895	-1.0852
2	3,000	28000	0.770	-0.897
3	10,000	54400	0.924	-1.373
4	10,000	42600	0.938	-1.327
5	10,000	121000	0.838	-1.354
6	10,000	70500	0.937	-1.297
7	10,000	49100	0.883	-1.352
8	10,000	43800	0.954	-1.357
9	10,000	75000	0.849	-1.368
10	1,000	62500	1.116	-1.371
11	10,000	40800	0.919	-1.352
12	7,000	89900	0.935	-1.304
13	10,000	64500	0.873	-1.338
14	10,000	10000	0.710	-0.844
15	19	116000	1.059	-1.572
16	1,000	98900	1.029	-1.341
17	10,000	80500	1.024	-1.461

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Hydrology Parts I and II - Quiz

Updated: 1/28/2019

1. In order to have precipitation, which of the following must occur?

   a. Moisture droplets must coalesce
   b. Moisture droplets must become of sufficient size to overcome the air resistance
   c. Both of the above
   d. None of the above

2. Precipitation that is frozen water in a massive state is referred to as which of the following?

   a. Snow
   b. Hail
   c. Sleet
   d. None of the above

3. Precipitation that is forced to rise over a fixed-position geographic feature is referred to as which of the following?

   a. Orographic Storms
   b. Cyclonic Storms
   c. Convective Storms
   d. Hurricanes

4. What are the most common units of measure for rainfall intensity?

   a. Inches
   b. Inches per hour
   c. Inches per day
   d. None of the above

5. What term is used for the physical removal of water from the watershed by life actions associated with the growth of vegetation?

   a. Evaporation
   b. Depression storage
   c. Transpiration
   d. Interception

6. Which of the following are components of a typical hydrograph?

a. Rising limb
b. Falling limb
c. Peak flow
d. All of the above

7. What is the return period for a flood event that has a 10% chance of occurring in any year?

a. 5 years
b. 10 years
c. 25 years
d. 100 years

8. Which of the following factors impact the hydraulic character of the natural drainage system?

a. Drainage area
b. Slope
c. Hydraulic roughness
d. All of the above

9. How does a low hydraulic roughness affect the hydrograph for a given storm, compared to a high hydraulic roughness?

a. The hydrograph will have a smaller peak and be longer in duration
b. The hydrograph will have a larger peak and be longer in duration
c. The hydrograph will have a smaller peak and be shorter in duration
d. The hydrograph will have a larger peak and be shorter in duration

10. A watershed that is well covered by a pattern of interconnected drainage channels will have what impact on overland flow time?

a. Overland flow time will be relatively short
b. Overland flow time will be relatively long
c. Overland flow time will be the same

11. Which of the following abstractions must be satisfied before there is an excess of water available to run off the land surface?

a. Interception
b. Depression storage
c. Infiltration
d. All of the above

12. Which of the following types of flow typically occurs in the uppermost portions of a drainage basin?

a. Sheet flow
b. Shallow concentrated flow
c. Open channel flow
d. None of the above.

13. Regional regression equations relate the peak flow to what type of watershed characteristics?

a. Physiographic
b. Hydrologic
c. Meteorologic
d. All of the above

14. Which of the following parameters are included in the regional regression equations for Maine?

a. Drainage area
b. Channel slope
c. Percentage of total area that is lakes and ponds
d. All of the above

15. Why would some experts consider that regression equations give better estimates of peak flows than frequency analysis of a stream gage?

a. Regression equations more nearly reflect the capacity of the watershed to experience a peak flow of a given magnitude
b. Gage records indicate that adjacent watersheds of comparable size and parameters will produce very similar peak flows from the same event
c. Accuracy of gage records is suspect
d. None of the above

16. Which of the following statements on limitations of regressions equations is not true?

a. Rural equations should be used only for rural areas
b. Regression equations can be used even where there are dams or flood retention structures
c. Drainage area should always be determined
d. Frequency curves for watersheds contained in more than one region cannot be computed.

17. Which of the following factor is included in the USGS urban regression equations?

a. Drainage area
b. Peak discharge for the same watershed in a rural condition
c. A basin development factor
d. All of the above

18. In order to determine the basin development factor (BDF) for use in the USGS urban regression equations, the basin is divided into how many approximately equal sections?

a. Three
b. Four
c. Five
d. At least six

19. In using the USGS urban regression equations, changes such as straightening, enlarging, deepening and clearing are considerations for which of the following parameters?

a. Channel linings
b. Channel modifications
c. Storm sewers
d. Urbanization

20. The USGS Streamstats software includes geospatial datasets that include which of the following parameters?

a. Water features
b. Historic climate data
c. Soils information
d. All of the above

21. The USGS StreamStats software can compute which of the following basin characteristics?

a. Drainage area
b. Stream slope
c. Mean annual precipitation
d. All of the above

22. The depth of direct runoff in the SCS Graphical Peak Discharge Method is a not a function of which of the following?

a. Depth of precipitation
b. Intensity of precipitation
c. Initial abstractions
d. Maximum potential retention

23. A soil consisting of clays loams would likely be classified into which hydrologic soil group?

a. Group A
b. Group B
c. Group C
d. Group D

24. What is the appropriate value of CN for a wooded area with good cover and soil group C?

a. 55
b. 66
c. 70
d. 85

25. What is the appropriate CN value for commercial land use with 85 percent imperviousness, and Group A soils?

a. 89
b. 92
c. 94
d. 95

26. The SCS Method should only be used when the CN is greater than what value?

a. 40
b. 50
c. 60
d. 70

27. What is the recommended maximum drainage area for the Rational Method?

a. 10 acres
b. 100 acres
c. 200 acres
d. 5 square miles