PROCEEDINGS OF A WORKSHOP ON COUNTY HYDROLOGY MANUALS

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FREQUENTLY-USED HYDROLOGIC TECHNIQUES FOR SMALL WATERSHED ANALYSIS

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INTRODUCTION

Small watersheds typically respond quickly to the precipitation which exceeds the basin losses and produces runoff from the basin. Surface runoff is typically the dominant contributor to stream flow. Simplified runoff computation techniques usually give an accurate evaluation of runoff magnitude for small watersheds. Commonly used small watershed hydrologic techniques are summarized in this paper.

The subjects covered include precipitation, loss rate estimation, and runoff computations. Runoff may be computed as either a peak discharge or as a runoff hydrograph. The rational method is the most common peak discharge calculation procedure. Various unit hydrograph techniques, such as the dimensionless S-graph method and the SCS unit hydrograph method, are also discussed.

WATERSHED RUNOFF CYCLE

The processes shown schematically in the watershed runoff cycle in Figure 1 are those which are usually most important in rainfall-runoff analysis. The processes in small watersheds which have the major influence on short term runoff are *surface infiltration*, *interception*, *depression storage* (these are usually termed "losses") and the *overland flow* and *surface detention* processes. Evaporation is usually negligible during the time that direct surface runoff occurs and is usually ignored.

The general hydrologic processes involved in producing runoff from small watersheds are described by the following series of events: During the early portions of the storm, a quantity of rainfall is trapped by watershed vegetative cover as interception; if the storm is of light intensity or of short duration, all the storm rainfall may be intercepted. If the initial rainfall quantities exceed the initial interception capacity of the watershed surfaces, natural and man-made surface depressions begin filling. Runoff will generally not appear downstream from such depressions until the total volume is occupied by runoff (or by a combination of runoff, debris, and sediments) from the upstream tributary areas. These two processes are usually grouped together and called the initial abstraction of the watershed. This may be either a significant or negligible factor in the subsequent rainfall-runoff budget. For example, a light storm occurring over a watershed which contains major obstacles to direct runoff flow (such as gravel pits, dams, or natural sumps) may result in no runoff to the downstream portions of the watershed. In contrast, a fully developed watershed with storm drains and connected street systems may have an initial abstraction which is negligible; consequently, a severe storm event may satisfy the interception and depression storage capacity in the early portions of the storm.

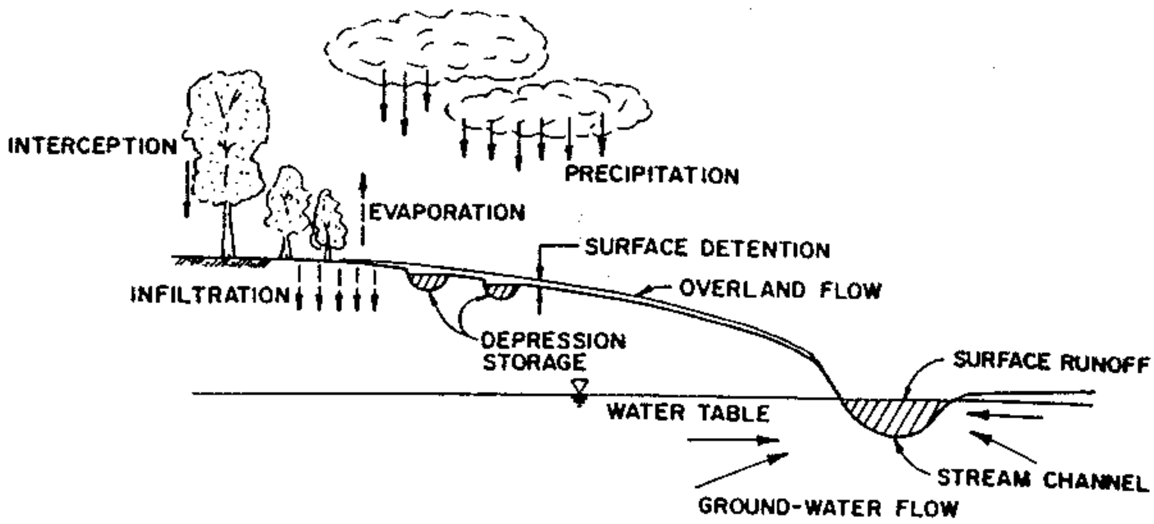


Figure 1. Watershed Runoff Cycle.

The infiltration capacity of the soil depends on several variables including the condition of the soil and the soil moisture content. If the soil infiltration capacity is exceeded by the storm rainfall intensity, overland flow begins, and the soil surface is covered by a very thin depth of water (the surface detention or detention storage). Typically, surface runoff is the main contributor to the runoff which appears in the watershed's stream system. If the storm is of long duration (usually days), portions of the infiltrated water may reappear as runoff through the process of lateral groundwater movement into lower elevation stream channels.

In urban hydrologic studies, flood control systems are typically designed to accommodate a severe storm event which is considered to provide a reasonable level of public protection for the corresponding cost to the community. For such severe storm conditions, the initial abstraction in an urban watershed is generally assumed to have a negligible effect on the storm runoff peak flow. This conservative assumption is based on the common situation where rainfall early in the storm event fills the available depression storage capacity. This is common in fully developed watersheds. Consequently, in urban design storm studies, many of the watershed processes may be assumed to have a small impact on the subsequent runoff quantities, and thus the watershed modeling approach is simplified.

PRECIPITATION

Precipitation, usually in the form of rain, is the major factor in the runoff process. Precipitation can vary greatly in space and in time; these variations result from the highly varied physical processes which govern the mechanics of precipitation. For most design and analysis applications for small watersheds "synthetic" rainfall data are used. The time and space variations of precipitation are represented statistically, with rainfall depth and intensity related to storm duration and frequency of storm occurrence. Hydrologic design procedures for small watersheds usually require only "point" rainfall data. Larger watersheds require the use of average rainfall depths which more properly represent the distribution of the rainfall over the full watershed, and depth-area adjustments are applied to the point rainfall values to produce an average rainfall depth which is representative of the volume of precipitation which occurs over a large area.

The primary source of rainfall data in the United States is the U. S. National Weather Service. The National Weather Service maintains rain gage networks throughout the country and provides information of maximum precipitation for durations of 5 minutes and longer. Other agencies which provide weather information are the U. S. Geological Survey, the U. S. Department of Agriculture, and the state of California, local government (counties and cities), and private cooperative weather observers. An important source of precipitation data for design are the National Weather Service publications such as NOAA Atlas 2 (Miller et al. 1973). These publications provide point precipitation isohyetal maps for 6- and 24-hour durations with return frequencies of 2- and 100-years. These publications also provide procedures for adjusting the rainfall data for other return frequencies and intervals.

PRECIPITATION-DEPTH-FREQUENCY RELATIONSHIPS

A strictly deterministic analysis of precipitation quantity is not possible, due to the randomness of precipitation patterns and intensities, and as a result, statistical evaluations are used. The terms *precipitation depth, duration,* and *frequency*, and *intensity* are used in these analyses. Precipitation depth (in inches) is the amount of precipitation occurring during a specified duration of a storm. Duration is the specified length of storm time and may be expressed in units of minutes, hours, days, or seasons. Frequency is usually expressed as a *return period* (or *recurrence interval*) and is the average period of time (in years) in which the precipitation depth (or intensity) is equaled or exceeded. *Intensity* is obtained by dividing the precipitation depth by duration to give an average intensity for a specified duration.

In small watershed analyses, rainfall intensity is usually the most important parameter. Intensity relates both precipitation volume and storm duration. The storm runoff is also related to storm precipitation through the intensity since the intensity provides an upper bound to the watershed runoff rate.

The statistical concept of return frequency used in the design of flood protection works for two reasons: 1) it aids in assigning a probability to a precipitation event and 2) it permits the corresponding *risk* to public protection to be defined. Terms used with this statistical interpretation are: *Exceedence (cumulative) probability* which is the probability that a precipitation event of a specified depth and duration will be equaled or exceeded in one year, and *Return period* which is the long-term average number of years within which a given depth and duration will be equaled or exceeded. The exceedence probability (P) and the return period (T) are related by the following equation:

$$P = 1/T \tag{1}$$

EVENT DEPTH DURATION

The maximum intensities of any type of precipitation event possible within a watershed are of interest in hydrologic studies. If a history of such maximum rainfall intensity duration data exists, a statistical interpretation can be made to determine estimates of maximum rainfall intensities or depths as a function of storm duration and return frequency.

For most small watershed hydrologic studies the most important relationship is that of precipitation depth for the rainfall event, with respect to a specified duration.

POINT PRECIPITATION AND THE DESIGN STORM

When rain gage records are examined to identify tendencies and patterns, an extremely wide range of variations is found. The random variations are generally so great that they obscure any long-term pattern or periodicity which may exist. Therefore, a combination of probabilistic and deterministic methods are needed to utilize point precipitation data in a design storm method. The duration and magnitude of individual storm events are assumed to be probabilistic, while the distribution of rainfall within the design storm is often assumed to be deterministic. Additionally, because severe storms can have different types of origin (i.e., thunderstorms, marine-type storms, etc.) it can be very difficult to develop a comprehensive analysis and to establish a conclusive description of the actual probabilistic distribution of severe storm rainfall quantities.

INTENSITY-DURATION-FREQUENCY CURVES

Intensity-duration relationships may be presented as curves giving rainfall intensity in inches per hour versus duration in minutes. Intensity-duration data for durations of less than 3 hours tend to plot as straight lines on standard log-log paper, and the curves for various return frequencies tend to be parallel to each other.

For each rain gage, the collected precipitation records are analyzed to determine the annual maximum rainfall depth for several durations of interest, such as 5 minutes, 1 hour, 3 hours, etc. This information can then be arranged in an increasing order of magnitude for each storm duration for the history of the gage.

This is then plotted on logarithmic graph paper. From this accumulation of depth-duration data, various statistical models can be applied to assign return frequencies. The resulting synthesized data are termed point rainfall values to distinguish them from area-averaged values for large watersheds.

ESTIMATION OF EFFECTIVE RAINFALL

Effective rainfall, also known as rainfall excess, is that fraction of the total rainfall which is converted into direct surface runoff. It is equal to the total rainfall minus the watershed losses. Watershed losses consist of infiltration, depression storage, interception, and to a minor extent, evaporation and transpiration. The two most frequently used loss rate procedures are the phi-index and SCS methods. Other empirically based methods are also used.

Infiltration--For severe design storm conditions, rainfall intensities usually exceed the infiltration capacity of the soil during a large portion of the storm, and the soil infiltration rate is usually at its lowest rate during the period of peak rainfall intensities.

In most small watershed studies a simplified effective rainfall estimation method procedure based on assuming a constant or average value of infiltration for the entire storm is used (Hromadka, McCuen and Yen 1987). This concept is analogous to the phi-index approach of representing the watershed losses (Viessman, et al. 1989).

Depression Storage--Depression storage is associated with a rainfall volume which is permanently stored in natural or human-made obstacles to runoff. Table 1 provides data for estimating depression storage; as the values in the table indicate, this loss can vary over a wide range, which reflects the uncertainty associated with this parameter. When antecedent rainfall is large or when the storm is of long duration and appreciable runoff occurs in the initial periods of the storm, depression storage will produce no change in effective rainfall.

Interception by Vegetation and Structures--This is the volume of rainfall retained on the leaves and stems of plants. For a long duration storm or when there has been significant prior precipitation, the interception loss will have little effect on the peak runoff.

In most small watershed runoff computation procedures the interception loss is simply subtracted from the rainfall at the beginning of the rainfall period. In many cases it is ignored.

Phi-Index Method--Field studies have shown that the infiltration capacity is greatest at the start of a storm and that it decreases rapidly to a relatively constant rate. The time over which infiltration goes from an initially high rate to a constant rate may be as short as 10 to 15 minutes. Therefore, it is not unreasonable to assume the infiltration rate to be constant over the entire storm duration. When the rainfall rate exceeds the rate of infiltration, the loss rate is assumed to equal the potential infiltration, which is called the phi index. When the rainfall rate is less than the value of phi, the infiltration rate is assumed to be equal to the rainfall intensity.

Table 1

DEPRESSION STORAGE FOR VARIOUS TYPES OF LAND COVER

Land Cover Type	Depression Storage (in.)	Recommended Value (in.)
Paved Surfaces	0.05 - 0.15	0.10
Roofs (flat)	0.10 - 0.30	0.10
Lawn and Grass	0.20 - 0.50	0.30
Wooded Areas & Open Fields	0.20 - 0.60	0.30

Mathematically, the phi-index method for modeling losses is described by

$$f(t) = \begin{cases} I(t), \text{ for } I(t) < \phi \\ \phi \text{ , for } I(t) > \phi \end{cases} \qquad (2)$$

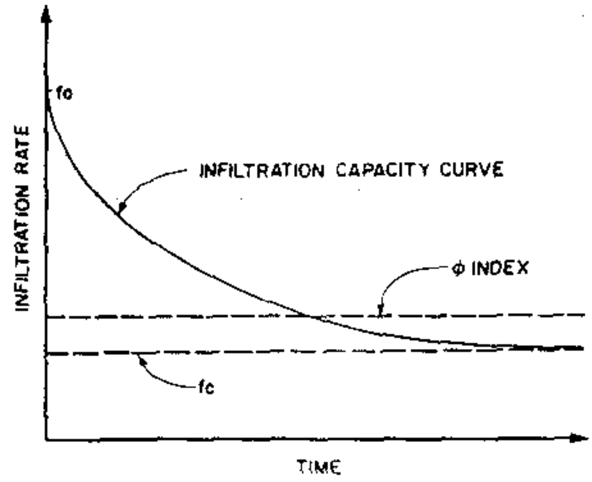
where f(t) is the loss rate, I(t) is storm rainfall intensity; t is time; and ϕ is the phi index.

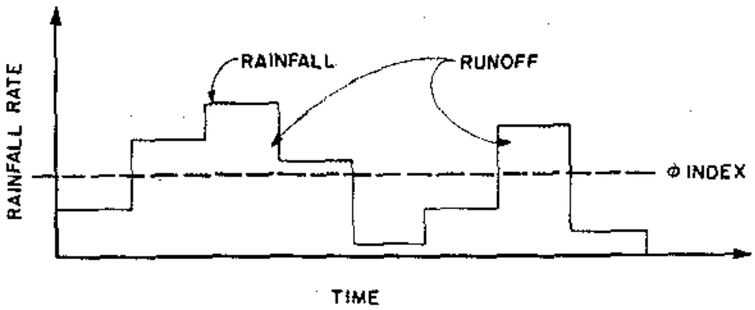
When measured storm event data are not available, the ultimate capacity of Horton's equation, f_c, can be used (f_c is defined below). Typical values of the ultimate infiltration rate capacity for bare soils are given in Table 1 and can be used as estimates of phi. At different locations on the watershed, the actual infiltration can vary considerably from these values.

Constant-Proportion Loss Rate--A constant-proportion loss rate, where the effective rainfall is taken to be a constant proportion of precipitation, is also used (Schilling and Fuchs, 1986, Williams et al. 1980, and Scully and Bender 1969). The loss rate function f(t) associated with the constant-proportion loss rate procedure for estimation of effective rainfall is given by (see Fig. 3):

$$f(t) = Y\, I(t) \qquad (3)$$

where f(t) is the loss rate; I(t) is the storm rainfall intensity; t is time; and Y is a calibration constant with $0 < Y < 1$.





Horton Loss Function with Phi-Index **Phi-Index Loss Function**

Figure 2. Loss Rate Functions

Coupled Phi-Index and Constant-Proportion Loss Rate Function--The phi-index and constant-proportion loss rate methods are both single parameter loss functions. This function combines both methods by using the phi-index as an upper limit to the loss rate determined from the constant-proportion loss rate approach. That is, the loss rate f(t) is defined as a function of time, $t > 0$ by

$$f(t) = \begin{cases} Y\,I(t), \text{ for } I(t) < \phi/Y \\ \phi, \text{ otherwise} \end{cases} \tag{4}$$

where ϕ and Y are calibration constants similar to the original definitions given by Equations (2) and (3). Use of this two-parameter model enables a calibration of the loss rate function to both storm peak rainfall intensity loss rates and the total storm yield. This type of loss function was used in the calibration of the hydrology manuals for Orange and San Bernardino Counties in California (Hromadka and McCuen 1986a, 1986b).

Horton Loss Rate--The Horton loss function shown in Fig. 2 is described by the three-parameter model

$$f(t) = f_c + (f_o - f_c)e^{-kt} \tag{5}$$

Where f_o is the initial loss rate, f_c is the ultimate loss rate, and k reflects the time rate of decay from f_o to f_c.

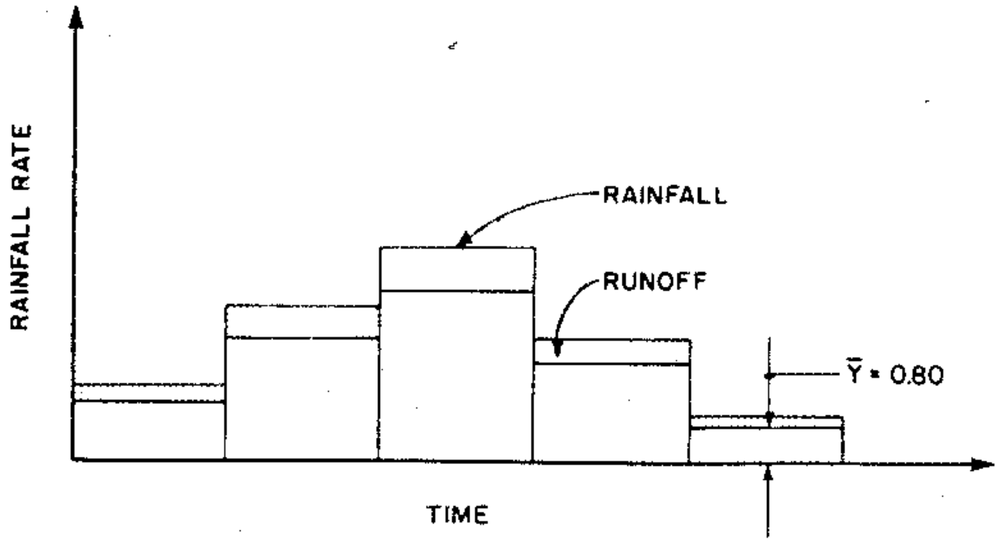


Figure 3. Constant-Porportion Loss Function

Initial Abstraction--The total loss used in the storm rainfall-runoff budget is usually separated into two parts: the initial abstraction, I_a, and a time-varying loss. Two models for the initial abstraction estimation which can be coupled to the previously discussed loss rate functions are: 1) total rainfall up to a total depth of I_a, and 2) percentage of total rainfall up to a total depth of I_a.

In the first initial abstraction model, all precipitation is budgeted towards the total rainfall depth exceeding I_a. The second I_a model accounts for a fixed percentage of rainfall until the total I_a depth is satisfied. In the second model, however, the loss rate is usually set equal to f(t) or the initial abstraction requirements, whichever is larger.

The first initial abstraction method is a one-parameter model and the second a is two-parameter model. Use of either initial abstraction method with the previously discussed loss rate models for f(t) will result in an effective rainfall estimation model which uses up to five parameters. Other effective rainfall estimation models have been developed which include budgets for accounting for groundwater flow, transpiration, and many other effects; this can result in models utilizing a dozen or more calibration parameters. Applying such a sophisticated approach to small watersheds does not produce an increase in modeling accuracy, and thus only the very simple approaches are normally used.

SCS Loss Model--The SCS runoff depth equation (U. S. Soil Conservation Service 1972, 1973) relates runoff depth to rainfall depth by the following equation

$$R = (P - I_a)^2/(P - I_a + S) \qquad (7)$$

in which R is runoff depth (inches), P is precipitation depth (inches), and the maximum watershed retention S is given by:

$$S = 1000/CN - 10 \qquad (8)$$

in which CN is a runoff index called the runoff *Curve Number*. The difference between the storm event depths of rainfall (P) and runoff (R) represents the total storm event losses.

Table 2

SCS HYDROLOGIC SOIL GROUPS

Soil Group	Description
A	Low runoff potential. Soils have a high infiltration rate even when thoroughly wetted. Consists chiefly of deep, well drained gravels and sands.
B	Soils having moderate infiltration rates when thoroughly wetted. Consists mainly of moderately deep, well drained soils with moderately fine to moderately coarse texture.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes the downward migration of water, or soils with moderately fine to fine texture.
D	High runoff potential. Soils having a very low rate of infiltration when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a high water table (eliminating soil water storage capacity), soils with a claypan or clay layer at or near the soil surface, or shallow soils over a nearly impervious layer.

The total loss is separated into two parts: the initial abstraction I_a and the other losses (or retention). The initial abstraction is related to the CN by the following empirical relationship:

$$Ia = 0.2S \tag{9}$$

For large values of S, the initial abstraction can be substantial and may appear to be irrationally large. In spite of this, the relationship of Equation 9 is a basic concept in the use of CN values and is a necessary factor in the computation of the total runoff depth R.

SCS Hydrologic Soil Groups--A major factor affecting infiltration is the condition of the soil itself. The soil surface characteristics, the ability of the soil to transmit water through subsurface layers, and the available soil storage capacity all control the infiltration rate. The SCS has classified more than 4000 soil types into four general categories to provide a general purpose classification of infiltration rates and corresponding runoff rates. The soil groups are defined in Table 2. Relative minimum infiltration rates, which depend on the condition and cover of the soil, are given in Table 3.

Table 3

MINIMUM INFILTRATION RATES

SCS Soil Group	Infiltration Rate (in./hr)
A	0.03 - 0.45
B	0.15 - 0.30
C	0.05 - 0.15
D	0.00 - 0.10

Watershed Development Conditions--If flood control facilities are expected to provide public protection for an extended period, then maximum urbanization should be assumed in determining the watershed loss rates. All available long range urbanization master plans should be examined to ensure that reasonable land use assumptions are included in the analysis. Particular attention should be directed to local landscape practices. For example, in more arid regions it is common to use ornamental gravels underlain by impervious plastic materials as a substitute for lawns and shrubs, resulting in an increase in the effective impervious percentage.

Impervious Areas--In the analysis of urban watersheds, the effects of impervious surfaces on the assumed area-averaged infiltration rate for the entire watershed must be included. Estimated ranges of impervious percentages for various types of land use are usually used (Hromadka, McCuen and Yen 1987). The percentage of area considered to be covered by an impervious surface however, may need to be reduced due to the local drainage practices. For example, runoff from an impervious surface may cross a pervious surface where infiltration may take place. To account for this infiltration, the actual impervious area should be reduced by a factor such as 10 percent. This type of adjustment is generally included in the computation of runoff coefficients for use in the rational method and in the estimation of watershed loss rates for use in synthetic unit hydrograph methods.

In using the SCS methods, the effects of impervious land covers are handled through adjustments to the CN and the time of concentration. The definition of CNs includes CNs for both urban and rural/natural land covers; however, the

CN's for the urban land covers are based on specific percentages of imperviousness and are accurate only when these stated percentages are representative of actual watershed conditions. Where the stated percentages of imperviousness are not accurate, the CN's can be adjusted using methods developed by the SCS (U. S. Soil Conservation Service 1973).

PEAK DISCHARGE ESTIMATION

Some rainfall-runoff analysis problems require only estimates of maximum flow rates (or peak discharges) and the time characteristics of the runoff. These types of problems are discussed in this section before we proceed to a discussion of hydrograph analysis methods.

The time from the beginning of rainfall excess to the occurrence of the peak runoff rate is important both for determining the duration of the rainfall and in balancing discharge rates from subareas of a watershed. The *time of concentration*, which is the most important time parameter, is a required input to most hydrologic models, including the rational method.

Time of Concentration Estimation--The time of concentration (T_c) is usually defined as the duration required for runoff at the point of concentration to become a maximum under a uniform and constant rainfall intensity. This occurs when all parts of the drainage area are contributing to the flow. Generally, the time of concentration is the interval of time from the beginning of rainfall for water from the hydraulically most remote point of the drainage area to reach the point of concentration (e.g., a drainage structure). T_c is a function of several variables including the length of the flow path from the most remote point of the watershed to the concentration point, the slope of the flowpath, characteristics of natural and improved channels within the drainage area, the infiltration properties of the soil, and the extent and type of development.

Recognizing the importance of the time of concentration as input to hydrologic models, a wide array of methods have been proposed for estimating T_c. Most methods use two or more of the following factors: (1) flow length, (2) flowpath slope, (3) land use or a representative surface roughness, and (4) the intensity of the rainfall. Because time of concentration is required for all types of flow, both overland flow and channel flow, the inputs for estimating T_c should reflect the primary flow regime in the flowpath being analyzed. Given the wide array of methods available, it is important to use a T_c method for design that uses inputs that correspond directly to the character of the flowpath being analyzed. The following paragraphs describe methods commonly used for design.

The literature contains a number of procedures and overland flow formulas which can be used to estimate the initial subarea T_c. The simplest method is to estimate a T_c at the point of concentration by using a generalized overland flow formula. Various empirical equations are also in use. A variation of the Kirpich (1940) formula that is widely used has the form

$$T_c = k(L^3/H)^E \tag{10}$$

where L is length of initial subarea flowpath (feet), H is drop in elevation along flowpath (feet), k is a coefficient depending on development type , and E is a constant exponent, usually 0.2. Due to the inherent inaccuracy in the determination of a generalized equation for computing T_c for overland flow, Equation (10) should only be used on subareas of less than about 10 acres.

An alternative version of the Kirpich formula is:

$$T_c = 0.00013L^{0.77}S^{-0.385} \tag{11}$$

in which L is the hydraulic length in feet, S is the slope in ft/ft, and T_c is the time of concentration in hours. Equation (11) is based on data from watersheds in Tennessee that have areas from 1 to 112 acres and slopes from 3 to 10 percent. The computed times of concentration should be multiplied by 0.4 for watersheds where the overland flow path is concrete or asphalt and the channel is lined.

The Federal Aviation Agency (U.S. Department of Transportation, 1975) proposed the equation

$$T_c = 1.8(1.1 - C)L^{0.50}S^{-0.333} \tag{12}$$

where C is a runoff coefficient for the rational method, S is average surface slope in percent, and L is a characteristic flow length.

The kinematic wave equation is widely used for very small flow lengths:

$$T_c = 0.94\, L^{0.6}n^{0.6}i^{-0.4}S_f^{-0.3} \tag{13}$$

in which L is the flow length in feet, n is Manning's roughness coefficient, i is the excess rainfall rate in inches/hr, and Sf is the slope in ft/ft. Equation (13) is valid only for very shallow sheet flow over lengths of 300 feet or less. It is especially useful for T_c estimation of gutter flow in urban areas.

For watersheds having flow paths over mixed land cover, the time of concentration (T_c) can be estimated by summing the runoff travel times (T_t) through the several flow paths as the flood peak travels downstream to the watershed outlet. These flow paths include overland flow, street flow, pipe flow, and open channel flow in natural or improved channels. T_c must include the effects of the flood peak increasing in magnitude as the tributary area to the main collection stream increases. The mixed velocity method is applied to waterrsheds that have flow paths of mixed land covers.

For upland flow paths equations such as Equation (10) or Equation (13) can be used. As the runoff enters an area of more concentrated flow, the flow velocity can be estimated with Manning's equation. The travel time for the flow path with velocity V in feet/second can be computed using the relationship

$$T_t = L/V \tag{14}$$

where T_t is the travel time in seconds and L is the length in feet. The total travel time for the watershed is the sum of the individual travel times:

$$T_t = \sum_{i=1}^{n} [L_i/V_i] \tag{15}$$

in which n is the number of flow paths. All travel times computed for the individual flow paths are included in the summation. For larger watersheds where conditions are not homogeneous, the watershed should be divided into subareas and the times of concentration computed for each subarea. The main flowpath is identified such that the watershed can be subdivided into subareas with each subarea tributary to the collection stream. The main flowpath is segmented into reaches that are relatively homogeneous in runoff characteristics. The subareas gradually increase in size in the downstream direction along the collection stream. Additionally, nodal points (i = 1, 2, ..., m) are defined along the main stream so that each subarea has an associated upstream and downstream node number. The initial subarea time of concentration for the overland flow between node numbers 1 and 2 is estimated by one of the overland flow formulas or by using an assumed average flow velocity for the runoff traveling along the main flowpath within the initial subarea. Subsequent T_c values are determined by

$$T_c(i+1) = T_c(i) + T_t(i,i+1) \tag{16}$$

where $T_c(i+1)$ is T_c at node number i+1, $T_c(i)$ is T_c at node number i, $T_t(i,i+1)$is travel time for the flow between nodes i and i+1.

The travel time for each segment of the flowpath is then computed. To estimate the travel time values $T_t(i,i+1)$, Manning's formula is used to calculate a normal depth for the runoff flowing in the channel linking nodes i and i+1 and the corresponding flow velocity is used to estimate the time for the peak Q to move from node i to node i+1. Then the time of concentration computed by the velocity method is:

$$T_t(i,i+1) = L(i,i+1)/V(i,i+1) \tag{17}$$

where L(i,i+1) is the length of channel linking nodes i and i+1, and V(i,i+1) is the normal depth flow velocity for Q(i).

RATIONAL METHOD

The most widely used hydrologic equation for estimation of peak runoff rates is the rational method. This approach is typically used to estimate runoff rates from small urban areas. In the past this method has been directly applied to watersheds several square miles in area. In modern practice the watershed size is usually limited to about one square mile.

The rational method equation relates rainfall intensity, a runoff coefficient, and drainage area size to the direct peak runoff rate. This relationship is expressed by the equation

$$Q = CIA \tag{18}$$

where Q is the peak runoff rate in cubic feet per second (cfs) at the point of concentration, C is a runoff coefficient representing the area-averaged ratio of runoff to rainfall rates, I is the time-averaged rainfall intensity in inches per hour corresponding to the time of concentration, and A is the drainage area in acres.

The values of the runoff coefficient and rainfall intensity are based on drainage area characteristics such as the type and condition of the runoff surfaces and the time of concentration. These factors and the limitations of the rational method equation are discussed in the following sections.

Data required for the computation of peak discharge by the rational method include (1) rainfall intensity for a storm of specified duration and selected return frequency; (2) drainage area characteristics of size, shape, slope; and (3) a land use index that reflects the amount of rainfall that will appear as direct runoff. The drainage area may be determined by planimetering a suitable topographic map of the tributary watershed area. The duration of the storm rainfall required in the rational method equation is based on the time of concentration of the tributary drainage area. Rainfall intensity (I) is determined from local precipitation intensity-duration curves of the desired return frequency. Since one acre-inch/hour is equal to 1.008 cfs, the rational method equation gives the peak flowrate in cfs since the factor for conversion of units is taken as 1.0.

Intensity duration curves for a particular region can be developed making a log-log plot of the area-averaged point rainfall value for the one hour duration, and drawing a straight line through the one hour value with a slope based on shorter duration rainfall intensity values.

Runoff Coefficient--The runoff coefficient (C) is the ratio of peak rate of runoff to the rate of rainfall at an average intensity when the total drainage area is contributing runoff to the point of concentration. The selection of the runoff coefficient depends on drainage area slope, type and amount of vegetative cover, distribution and magnitude of the soil infiltration capacity, and various other factors.

For calculation purposes, the runoff coefficient is most often defined to be either (1) a constant value depending on soil cover type and quality, or (2) a function of rainfall intensity, soil cover type, and quality. Table 4 lists typical C values for use with the rational method.

A second class of runoff coefficients relate the C value to the rainfall intensity. One approach that is used for urban design purposes is to assume that the watershed loss rate is equal to the infiltration loss rate which corresponds to the limiting value of the infiltration capacity curve. For design storm conditions, it can be argued that the impervious area runoff rate is independent of the rainfall intensity and that the pervious area infiltration loss rate is a constant. For urban design studies, the runoff coefficient is sometimes assumed to be function of the impervious and pervious area fractions, a characteristic infiltration rate (F_p) for the pervious area fraction, and the effects of watershed detention in the estimation of travel time of the peak runoff rate through the watershed channel system. Estimates for runoff coefficients are developed using a relationship of the form

$$C_m = 0.85\,[A_i + (I - F_p)A_p/I] \tag{19}$$

where C_m is the modified runoff coefficient, I is rainfall intensity (inches/hour), F_p is the infiltration rate for pervious area fraction, A_i is impervious area fraction, A_p is the pervious area fraction, and the value 0.85 is a calibrated (or assumed) coefficient to correlate rainfall and runoff frequencies. The infiltration rate for the pervious area (F_p) can be estimated for various combinations of soil type, cover,

Table 4
RUNOFF COEFFICIENTS FOR THE RATIONAL FORMULA

Land use	A			B			C			D		
	0-2%	0-6%	6%+	0-2%	2-6%	6%+	0-2%	0-6%	6%+	0-2%	2-6%	6%+
Cultivated land----	0.08[1]	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14[2]	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture------------	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow-------------	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest-------------	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.16	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential lot	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
size 1/8 acre-----	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Residential lot	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
size 1/4 acre-----	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Residential lot	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
size 1/3 acre-----	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Residential lot	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
size 1/2 acre-----	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Residential lot	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
size 1 acre-------	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Industrial---------	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial---------	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets------------	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open space---------	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking------------	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

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

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

and antecedent moisture conditions. When the drainage area is composed of several types of surfaces an area-averaged runoff coefficient can be developed.

Limitations of the Rational Method--The relationship expressed by the rational method equation holds true only if certain assumptions are reasonably correct and limitations are observed. Four basic assumptions underlying this method are: (1) the frequency of the storm runoff is the same as the return frequency of rainfall producing the runoff (that is, a 25-year recurrence interval rainfall will result in a 25-year recurrence interval storm runoff); (2) the peak runoff rate occurs when all parts of the drainage area are contributing to the runoff; (3) the design rainfall is uniform over the watershed area tributary to the point of concentration; and (4) the rainfall intensity is essentially constant during the storm duration, which is equal to the time of concentration.

The rational method is only applicable where the rainfall intensity can be assumed to be uniformly distributed over the drainage area at a uniform rate throughout the storm duration. This assumption applies fairly well to small drainage areas of less than about one square mile. Beyond this limit, the rainfall distribution may vary considerably from the point values given in rainfall isohyetal maps.

The selection of the runoff coefficient is another major limitation of the method. For small urban areas, the runoff coefficient can be reasonably estimated from field investigations and studies of aerial photographs. For larger areas, the determination of the runoff coefficient is based on vegetation type, cover density, the infiltration capacity of the soil, and the slope of the drainage area. For these larger areas, an estimate of the runoff coefficient may be subject to a much greater error due to the variability of the drainage area characteristics, watershed storage, and the greater importance of hydraulic flow characteristics. Rainfall losses due to evaporation, transpiration, and depression and channel storage cannot be properly evaluated and may affect the estimate of the watershed peak rate of runoff.

UNIT HYDROGRAPH METHODS

The unit hydrograph method is an approach initially advanced by Sherman (1932). The underlying basis of the method is the assumption that watershed discharge is related to the total volume of runoff, and that the time-related factors which influence the unit hydrograph shape are invariant. The basic unit hydrograph theory was extended by Snyder (1938) to transpose storm rainfall-runoff relationships from gaged watersheds to hydrologically and geographically similar watersheds that lack runoff stream gage data. The basic assumptions used by Snyder are that the watershed rainfall-runoff relationships are functions of watershed area, slope, and certain shape factors. The method is used to estimate a time distribution for runoff accumulating at the watershed downstream point of concentration when stream gage data is either unavailable or inadequate to provide a sound statistical analysis.

To determine the rainfall-runoff relationships to be transposed to ungaged watersheds, streamgage records are studied for various types and sizes of gaged watersheds. For example, the Los Angeles District of the U.S. Army Corps of Engineers has determined several runoff time-distribution patterns for watersheds in California. Such relationships provide a basis for transposing to ungaged watersheds a characteristic time distribution of runoff that is the average distribution for similar gaged watersheds. This approach is considered applicable when watersheds are physiographically and hydrologically similar. In Southern California, for example, the counties of Orange, Riverside, and San Bernardino (which together represent a vast spectrum of flood control conditions) have successfully utilized this approach in the development of county-wide flood control facilities.

Although there are several theoretical shortcomings associated with the unit hydrograph approach (such as the assumption of a linear system in which runoff hydrographs resulting from a unit period of effective rainfall can be directly summed), the general approach continues to be widely used throughout the United States as a runoff synthesis method for ungaged watersheds. The following paragraphs discuss the general unit hydrograph approach and several currently used variations of the method.

The unit hydrograph design storm approach involves several assumptions that are imprecise approximations of the corresponding hydrologic processes. These basic assumptions are that (1) the critical storm rainfall pattern is uniformly distributed through the watershed; (2) there exists a direct proportionality between watershed runoff and the effective rainfall volume; (3) for any volume of effective rainfall occurring within a specified duration, the resulting runoff hydrograph is of a constant duration; and (4) the basin unit hydrograph is invariant throughout the critical design storm. The requirement that the watershed runoff is proportional to the effective rainfall has a direct analogy to a linear systems approach. Consequently, the unit hydrograph method can be considered a black-box modeling approach where the major characteristics of the model are determined by correlating the model output (runoff hydrograph) to the input data (rainfall records). Although the lumped-system model produces only approximations of the complex hydrologic characteristics of the watershed, its use continues to be widespread due to the ease of its application and the wide range of applications available for master planning purposes.

Unit Hydrograph Terminology--The following definitions are presented in order to illustrate unit hydrograph concepts and definitions (see Fig. 4).

Effective Rainfall: Total rainfall minus losses, which includes infiltration losses, evaporation, transpiration, and detention. This portion of the rainfall runs off the watershed surface in a relatively brief time period. Effective rainfall is also referred to as rainfall excess, with the volume of excess equaling the runoff volume.

Unit Hydrograph: Hydrograph for a point of concentration on a watershed representing the time distribution of runoff that results from 1 inch of effective rainfall over the entire watershed. The effective rainfall is assumed to occur as a constant rainfall in both space and time throughout the unit duration.

Distribution Graph: The distribution graph is a unit hydrograph whose ordinates are expressed in terms of percent of the ultimate discharge. A distribution graph is generally developed as a block graph with each block representing its associated percent of unit runoff that occurs during the specified unit time period. The unit time period corresponds to the one specified for the unit hydrograph.

Summation Hydrograph: The summation hydrograph for a point of concentration is a hydrograph showing the time distribution of the rates of runoff that would result from a continuous series of unit effective rainfalls over the tributary watershed. The ordinates are expressed as rate of runoff in percent of the ultimate rate of runoff.

Lag: Lag is defined in several ways. The definition used for many small watershed applications is that watershed lag is the time in hours from the beginning of a continuous series of unit effective rainfalls over the watershed to the instant when the rate of resulting runoff at the point of concentration reaches 50 percent of the ultimate rate of runoff. Another definition for lag is the time from the center of mass of the effective rainfall to the peak of the corresponding runoff hydrograph. Some hydrologists define lag as the time from the center of mass of effective rainfall to the center of mass of the runoff hydrograph. In this discussion, the first definition of lag will be used.

Ultimate Discharge: The ultimate discharge (or the ultimate rate of runoff) is the maximum rate of watershed runoff that can result from a specified effective rainfall intensity. Ultimate discharge occurs when the rate of runoff based on the summation hydrograph is equivalent to the rate of effective rainfall. For a unit effective rainfall intensity (one inch) occurring in a unit interval (one hour), the ultimate discharge is 645 cfs for each square mile of watershed.

S-Graph: The S-graph is a summation hydrograph developed by plotting watershed discharge in percent of the ultimate discharge as a function of time expressed in percent of the watershed lag.

<u>Convolution</u>--A unit hydrograph serves as the link between rainfall excess and direct runoff. The unit hydrograph is the modeling element that transforms a part of the total rainfall (the rainfall excess) into a part of the total runoff (the direct runoff). Thus, the unit hydrograph reflects watershed storage and physical processes of the watershed. Convolution is the process of converting a rainfall excess distribution into a direct runoff hydrograph. Convolution involves the steps of multiplication, time translation, and addition. Each ordinate of the hydrograph is multiplied by the first ordinate of the rainfall excess distribution, which results in a direct runoff hydrograph having a duration equal to the duration of the unit hydrograph. The unit hydrograph is then translated by one time unit and again each ordinate is multiplied by the second ordinate of the rainfall excess distribution, which produces a second direct runoff hydrograph. The unit hydrograph is then translated another time unit and the same multiplication operation is repeated. The translation/multiplication process is continued over all ordinates of the rainfall excess distribution. For each time increment, a separate

direct runoff hydrograph is generated. Then for each time period, all of the ordinates of the individual direct runoff hydrographs are added to get a total direct runoff hydrograph. The unit hydrograph is applied to each individual segment of rainfall excess. After the multiiplication-translation operation has been performed on each segment of rainfall, the individual direct runoff hydrographs are summed to get the total direct runoff hydrographs.

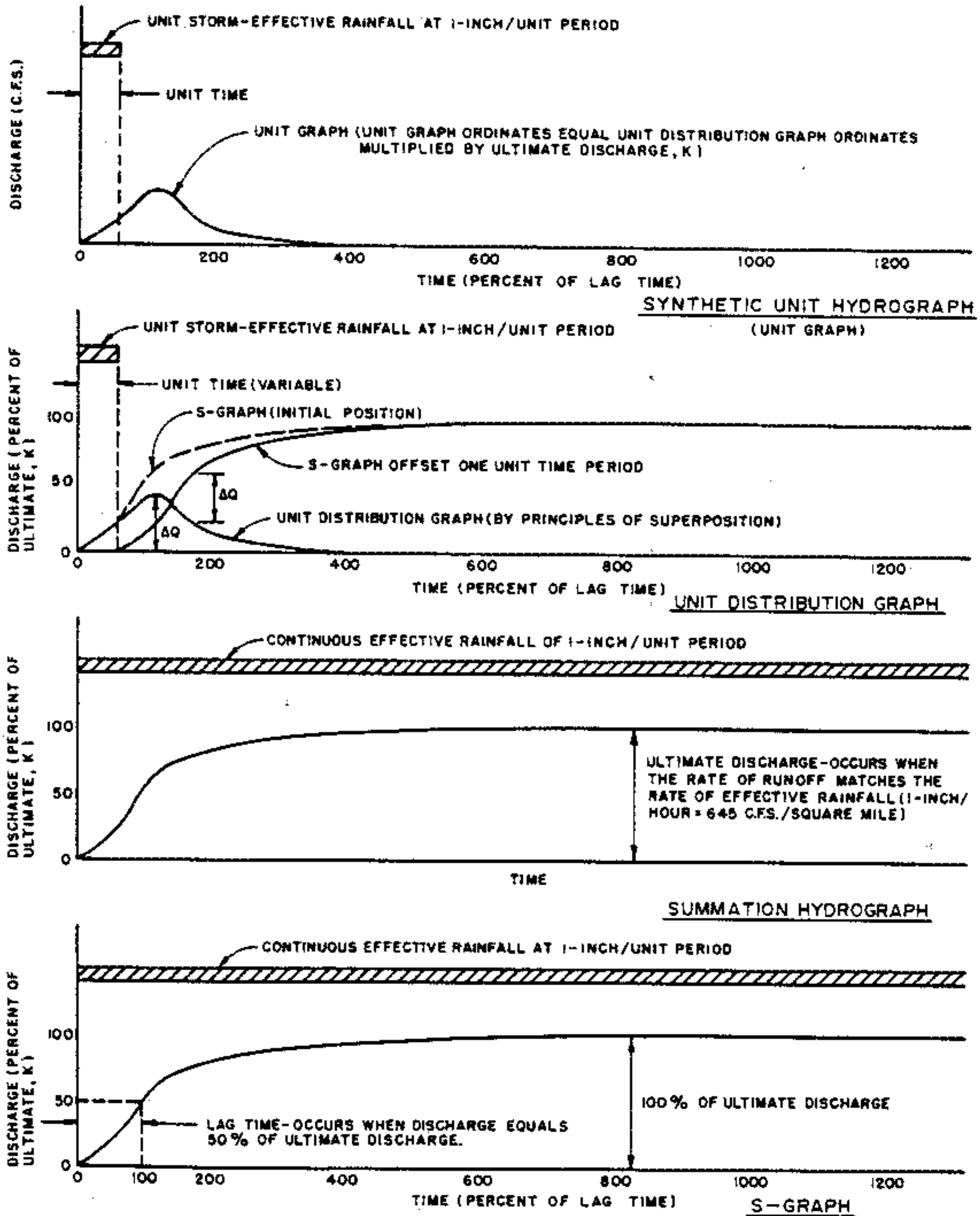


Figure 4. Definitions used in the Unit Hydrograph Concept

The convolution process is used in modeling both actual measured storm events and synthetic design storms. The rainfall distribution is converted to a distribution of rainfall excess by subtracting losses. The rainfall excess distribution is then convoluted with a unit hydrograph to get the total direct runoff hydrograph. Baseflow can then be added, if appropriate.

Unit Hydrograph Analysis--The conceptual development of the previous section suggests that, if a unit hydrograph for a duration of T hours is needed, it is necessary to find one or more storms having a uniform rainfall intensity for durations close to the desired duration of T hours. Convolution can then be applied to each storm to derive unit hydrographs for each storm and an average watershed unit hydrograph computed. This is the procedure that was used prior to the development of more elaborate, computer assisted methods.

Since very few storms have a reasonably constant intensity over the duration of the storm, it is difficult to find measured storm-event data that can be used to accurately estimate unit hydrographs. Thus, methods have been developed so that complex storms with nonuniform intensities can be evaluated. While unit hydrographs could be developed by trial and error analyses of complex storms, this would be exceptionally difficult for all except very simple cases and the resulting unit hydrographs would probably not be very reproducible; that is, analyses made by different individuals would not lead to the same unit hydrograph. Some methods require the assumption of a particular model form, which is then fit by the method of moments. For example, Aron and White (1982) used a gamma function, which has the general shape of a unit hydrograph, and then fit the parameters of the gamma function using the method of moments (which is the fitting of the two parameters of the gamma function using the mean and standard deviation of the hydrologic function data). Other techniques such as least squares regression analysis have also been used (Hydrologic Engineering Center 1982).

Developing Synthetic Unit Hydrographs--The data required to analyze unit hydrographs is rarely available for developing a site specific unit hydrograph. This is especially true in small urban watersheds because data collection networks have been for the most part located on large drainage basins. Furthermore, the process of developing a unit hydrograph would be made very difficult because urban watersheds are constantly being changed and thus watershed characteristics over the duration of the data record would not be constant. Nonhomogeneity of the data records would affect the characteristics of the resulting unit hydrograph, including the magnitude and timing of the peak.

To overcome the problem of not being able to develop a site specific unit hydrograph, a number of synthetic unit hydrographs have been formulated. The Snyder and Clark synthetic hydrographs and the SCS dimensionless unit hydrographs are widely used for ungaged locations (Hydrologic Engineering Center 1982, Hromadka, McCuen and Yen 1987). These methods are based on many analyses of watersheds where data were available, and such methods are generally usable in most regions of the country.

In transposing a unit hydrograph (or S-graph) between watersheds for design work (or when regionalizing unit hydrographs for a policy manual), it is assumed that the drainage areas within a given region are physiographically and hydrologically similar. However, no two drainage areas have identical hydrologic characteristics; the corresponding rainfall-runoff patterns are dissimilar and the distribution graphs may differ. However, most distribution graphs exhibit certain characteristics that appear to be related to watershed characteristics such as the drainage area and the timing factor of watershed lag. Based on such relationships, generalized synthetic unit hydrographs can be dimensionalized for use at ungaged locations.

To illustrate the development and use of synthetic unit hydrographs, a method developed for southern California will be discussed. It is important to recognize the two distinct phases involved: analysis and design. In the analysis phase the form of generalized unit hydrograph and the relationships necessary to dimensionalize it are developed through the analysis of measured rainfall and runoff data. The second phase involves dimensionalizing the generalized form using watershed information for the watershed where a design is required. This method is based on a watershed lag input. Watershed lag is defined here as the time from the beginning of the unit effective rainfall to the instant that the summation hydrograph reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gaged watersheds are regressed on the hydrological characteristics of the watersheds, a significant empirical relationship can usually be obtained. Analysis of watershed lags for various types of watersheds indicated that lag may be expressed by a calibrated formula such as

$$\mathbf{lag} = C_t[LL_c/S^{\frac{1}{2}}]^m \tag{20}$$

where **lag** is the watershed lag in hours, C_t is a calibration constant, L is the length of longest watercourse in miles, L_c is the length along longest watercourse, measured from the point of concentration to a point opposite the watershed area centroid in miles, s is the watercourse slope in feet per mile, and m is a calibration constant.

Studies have shown that m = 0.38 and C_t = 24n* for Southern California watersheds (n* is a basin factor, see Table 5). In many other areas of the United States a value for m of 0.30 is commonly used. The basin factors of Table 5 were determined by the U.S. Army Corps of Engineers by calibration using major storm hydrographs from several Southern California watersheds.

The second element of the synthetic unit hydrograph method is the dimensionless unit hydrograph, or S-graph. In the dimensionless form, the ordinate is expressed as a percent of the ultimate discharge and the abscissa as a percentage of the lag. Various S-graphs were developed from calibration studies of measured significant rainfall-runoff events. These S-graphs were assumed to be transferable for use in watersheds classified as valley, foothill, mountain, or desert. According to the definition, the S-graph reaches 50 percent of ultimate discharge at 100 percent of watershed lag, where the **lag** is determined using Equation 20.

The final step in developing the synthetic unit hydrograph is to multiply the ordinates of the distribution block graph by the ultimate discharge defined by

$$K = 645\ A/T \qquad (21)$$

where K is the watershed ultimate discharge in cfs, A is drainage area in square miles, and T is the unit time period in hours.

Table 5

BASIN FACTORS

n*= 0.015
1. Drainage area has fairly uniform, gentle slopes
2. Most watercourses are either improved or along paved streets
3. Groundcover consists of some grasses - large % of area impervious
4. Main watercourse is improved channel or conduit

n*= 0.020
1. Drainage area has some graded and non-uniform, gentle slopes
2. Over half of the area watercourses are improved or paved streets
3. Groundcover consists of equal amount of grasses and impervious area
4. Main watercourse is partly improved channel or conduit and partly greenbelt (see n*= 0.025)

n*= 0.025
1. Drainage area is generally rolling with gentle side slopes
2. Some drainage improvements in the area - streets and canals
3. Groundcover consists mostly of scattered brush and grass and small % impervious
4. Main watercourse is straight channels which are turfed or with stony beds and weeds on earth bank (greenbelt type)

n*= 0.030 (Foothill Area)
1. Drainage area is generally rolling with rounded ridges and moderate side slopes
2. No drainage improvements exist in the area
3 Groundcover includes scattered brush and grasses
4. Watercourses meander in fairly straight, unimproved channels with some boulders and lodged debris

n*= 0.040 (Foothill Area)
1. Drainage area is composed of steep upper canyons with moderate slopes in lower canyons
2. No drainage improvements exist in the area
3. Groundcover is mixed brush and trees with grasses in lower canyons
4. Watercourses have moderate bends and are moderately impeded by boulders and debris with meandering courses

n*= 0.050 (Mountain Areas)
1. Drainage area is quite rugged with sharp ridges and steep canyons
2. No drainage improvements exist in the area
3. Groundcover, excluding small areas of rock outcrops, includes many trees and considerable underbrush
4. Watercourse meander around sharp bends, over large boulders and considerable debris obstruction

n*= 0.200
1. Drainage area has comparatively uniform slopes
2. No drainage improvements exist in the area
3. Groundcover consists of cultivated crops or substantial growths of grass and fairly dense small shrubs, cacti, or similar vegetation
4. Surface characteristics are such that channelization does not occur

To use an S-graph, a watershed lag is estimated from Equation 20. A duration is selected (usually 15 to 25 percent of the watershed lag time) and amassed unit periods are expressed as accumulated percentages of the watershed lag. These percentages of lag are used for superimposing a block graph on the selected S-graph and the resulting block graph pattern is used in determining the accumulated mean percentage of ultimate discharge for each accumulated unit period. Finally, the incremental mean percentage of ultimate discharge for each unit period is estimated by a series of successive subtractions.

<u>The SCS Unit Hydrograph</u>--The U. S. Soil Conservation Service (1972) has developed a dimensionless unit hydrograph using data from a large number of measured storm events. Their analysis indicated that the unit hydrograph (in dimensionless form) can be based on the drainage area A in square miles, the runoff volume Q in inches, and the time to peak T_p in hours. A dimensionless unit hydrograph relates the ratios of q/q_p and t/T_p, where q is the discharge at any time t and q_p is the peak discharge which occurs at time T_p. The peak discharge of the unit hydrograph is computed by the following:

$$q_p = 484\, A\, Q/T_p \tag{22}$$

The time to peak can be estimated from the time of concentration, T_c. SCS uses the following relationship:

$$T_p = 0.6\, T_c \tag{23}$$

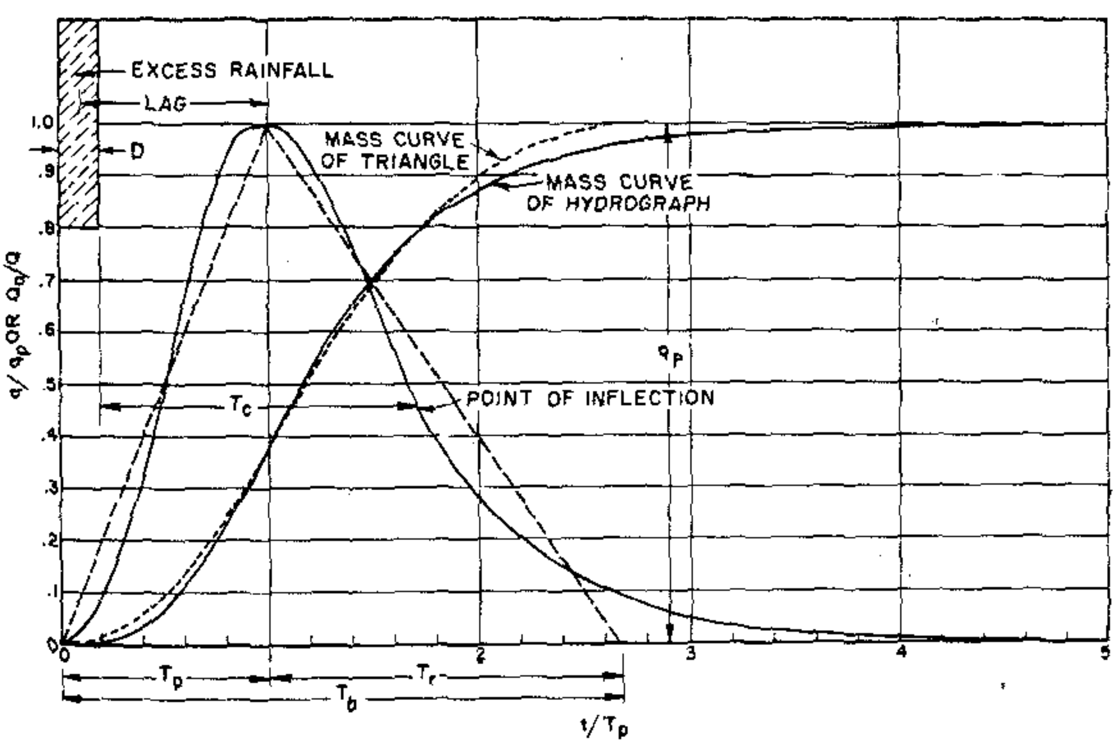


Figure 11. *SCS Unit Hydrograph Definitions*

Given the drainage area, runoff volume, and time of concentration, the peak discharge and the time to peak can be computed from Equations 22 and 23, respectively; these values are then used with the dimensionless unit hydrograph to compute the watershed unit hydrograph. For a unit hydrograph the volume of runoff equals 1 inch. In the SCS approach, whatever the condition or classification of the watershed, the ratio of the time to peak to total unit hydrograph duration is a constant. The volume of runoff under the rising limb of the unit hydrograph is a fixed 37.5 percent of the total runoff volume, and the associated peak rate factor is a constant value of 484.

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