Encyclopedia of Fluid Mechanics

VOLUME 10

Surface and Groundwater Flow Phenomena

Nicholas P. Cheremisinoff, Editor

in collaboration with—

S. Bhuvendralingam
J. F. Botha
C.-L. Chiu
A. B. Clark
D. Couillard
W. Czernuszenko
R. Desmarais
J. J. DeVries
B. J. Dutka
D. P. Galya
N. Hogben
T. V. Hromadka II
T. Huttula
A. J. Johnston
N. G. Kafoussias
K. C. Khilar
K. Krogerus
G. Morin
D. Munro
D. Myrhaug
P. J. W. Roberts
M. C. Roco
G. E. Rossi
D. S. H. Sita Ram Sarma
P. Seidl
A. K. Singh
V. P. Singh
G. J. Van Tonder
S. Vigneswaran
M. Virtanen
E. Weiss

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VOLUME 10

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CHAPTER 14

RAINFALL-RUNOFF MODELS

T. V. Hromadka II and J. J. DeVries

Williamson and Schmid
Irvine. California. USA

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Runoff is the flow that occurs at the outlet of a watershed or drainage basin as a result of precipitation on the basin. The runoff process includes *surface runoff* (water that travels over the ground surface to watershed channels) and *subsurface runoff* (runoff that moves below the surface of the ground to reach stream channels). Some of the subsurface runoff travels through the upper layers of the soil to the stream channels, while some water percolates downward to the saturated groundwater zone where it moves as groundwater flow to and through stream channels. Surface runoff travels very rapidly to stream channels, groundwater moves very slowly, and the upper layer subsurface flows move at an intermediate speed. Total runoff consists of surface runoff plus subsurface runoff; the flow of water in the stream channel making up the total runoff is called *streamflow*.

Streamflow is measured as a volume of water per unit time, in cubic feet per second (cfs) or cubic meters per second (m^3/s). Runoff is frequently referred to in units of water depth, for example as inches of runoff from a watershed. Implicit in the depth specification is an area (the basin area) as well as the time over which the runoff occurred, i.e., for a given storm, a given season, or a year. Runoff, thus, has units of volume per unit time, also. The instantaneous rate of streamflow (called the *discharge*) is frequently plotted as a function of time. This plot is called a *hydrograph*. The area under the hydrograph curve represents a runoff volume. This volume represents both surface runoff and subsurface runoff contributions to streamflow. A typical hydrograph occurring as a result of storm precipitation is shown in Figure 1. Prior to the start of the storm the only contribution to streamflow is groundwater flow. The rainfall during this storm was great enough to produce surface runoff, and the streamflow increases rapidly due to the surface runoff contribution. Surface runoff from different parts of the watershed reaches the stream at different times, and thus the peak flow (hydrograph peak) occurs some time after the occurrence of the peak rainfall. The part of the hydrograph prior to the peak is called the *rising limb*; the part after the peak is called the *recession limb*. The hydrograph can be divided into a part that represents groundwater flow (*base flow*) and a part that represents *direct surface runoff* (SRO), as shown in Figure 1.

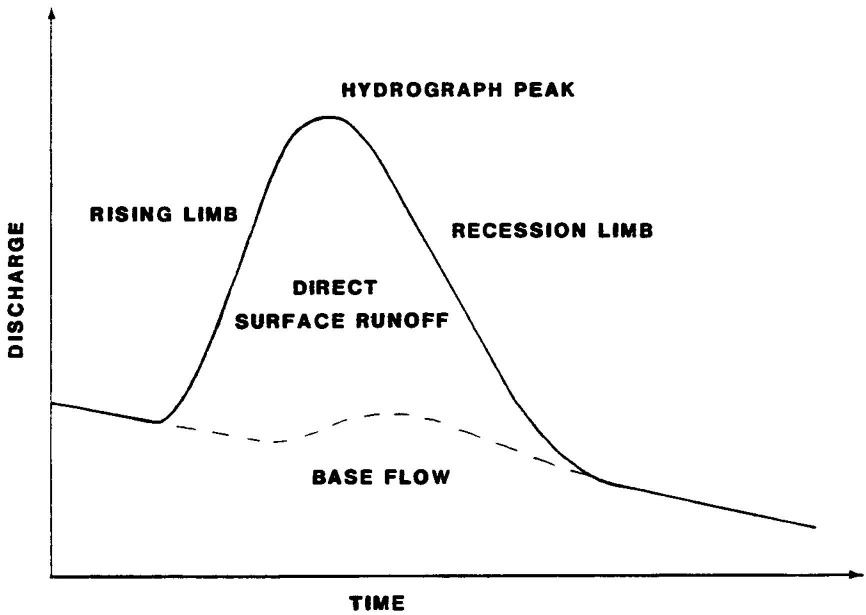


Figure 1. Stormflow hydrograph.

Much of the groundwater flow that shows up as base flow during and immediately after the current period of precipitation is runoff that is due to previous precipitation. The surface runoff part of the hydrograph represents the direct contribution of the storm rainfall to runoff. The volume of the SRO will always be less than the total rainfall. There are various types of losses which must be subtracted from the total rainfall to determine the runoff. For a given rainfall event, the losses are equal to the total rainfall minus the volume of surface runoff (converted to equivalent depth of precipitation on the basin).

The surface runoff volume can be determined by separating the base flow from the SRO and numerically computing the SRO volume. Various base-flow separation procedures can be used. All involve approximations and the use of judgment in defining when the surface runoff ends and how base flow changes during the storm runoff period. In many cases, using a straight line connecting the beginning and the end of the direct surface runoff period to separate the two types of flow is adequate. In most urban watersheds, the base flow is very small when compared with direct surface runoff, and base flow is frequently ignored in rainfall-runoff computations.

RUNOFF CYCLE

The processes shown schematically in the watershed-runoff cycle in Figure 2 are those that are usually most important in rainfall-runoff analysis. The watershed processes that have the most 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 ignored. The general hydrologic process involved in producing watershed runoff 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

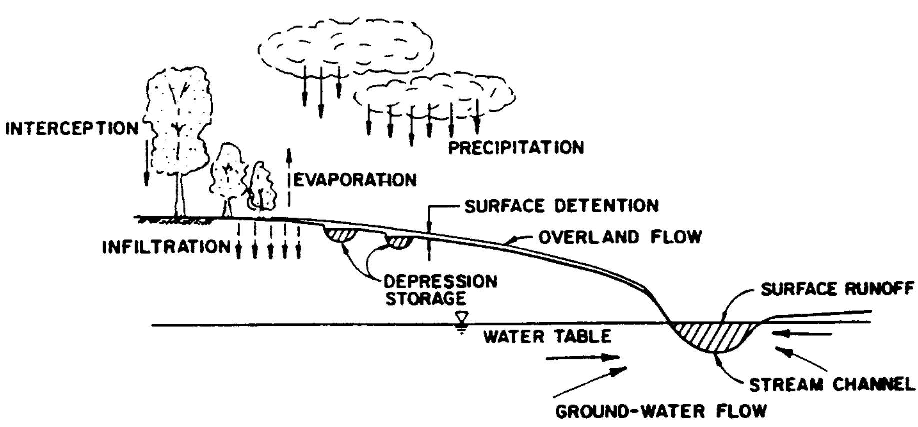


Figure 2. Watershed-runoff cycle.

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.

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 entire soil surface is covered by a film of water (the surface detention or detention storage). Typically, surface runoff is the main contributor to the runoff quantities that appear in the watershed's stream system. If the storm is of long duration (say, 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 severe assumption anticipates the occurrence of a prior storm event that depletes the available depression storage capacity, or it assumes a fully developed watershed condition with total positive drainage. Consequently, in urban design storm studies, several of the watershed model variables may be assumed to have a small impact on the subsequent runoff quantities, and the corresponding watershed-modeling approach is significantly simplified.

PRECIPITATION

Precipitation, usually in the form of rain, is the initial factor in the rainfall-runoff process. Precipitation can vary markedly in space and in time: these variations result from the highly varied physical processes that govern the formation of precipitation. Two types of precipitation data are used as input in the rainfall-runoff modeling process. Measured precipitation at a rain gauge can serve as the basic model input, but for most design and analysis problems, "synthetic" rainfall data are used. In this latter type, the time and space variations of precipitation are represented using a statistical model, with rainfall depth and intensity statistically related to the storm duration and frequency of occurrence. Most hydrologic designs require only "point" rainfall characteristics, but some large watersheds require the use of average rainfall depths that more properly represent the distribution of the rainfall over the full watershed. In this case, depth-area adjustments are applied to the point rainfall values to produce an average rainfall depth that is representative of the volume of precipitation which occurs over an large area. Rainfall varies spatially as well as in time, and this type of variation must be considered in rainfall-runoff analyses.

The primary source of rainfall data in the United States is the U.S. National Weather Service (NWS). The National Weather Service maintains rain gauge 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 state and local government and private cooperative weather observers. Of prime importance as a source of precipitation data in design are the National Weather Service publications such as NOAA Atlas 2 [20]. These 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

Due to the apparent randomness of precipitation patterns and intensities, a strictly deterministic analysis of precipitation quantity is not possible, and as a result, a statistical evaluation is used. In the statistical analysis, the following definitions of *precipitation depth, duration, frequency*, and *intensity* are used: Precipitation depth is the amount of precipitation occurring during a specified duration of a storm, and precipitation depth is usually specified in inches. 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*), which is the average period of time, expressed 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 the average intensity for the specified duration.

In hydrologic analyses, the 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.

In order to provide a reasonable level of flood protection, the statistical concept of return frequency is used since it aids in assigning a probabilistic meaning to a precipitation event as well as the corresponding *risk* to public protection. Definitions often used for 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:

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

From the above definitions, it can be argued that a 100-year precipitation event, for example, will not necessarily occur once in every 100 years, but actually has a finite probability of occurring in two or more years in succession.

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 hydrologic studies, the most important relationship is that of precipitation depth for any rainfall event (regardless of storm origin) with respect to a specified duration. Such information should include total precipitation from storms of given duration, as well as depths from independent continuous partial storm durations. This is what is done, for example, in the NOAA Atlas 2. This information can be represented by event depth-duration curves that are constructed by ranking in the order of increasing rainfall depth all storm events of some specified duration. From the position of a precipitation depth an estimate can be made of the number of years during which the subject event will be equaled or exceeded.

Point Precipitation and the Design Storm

When rain gauge 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 that may exist. Therefore, to utilize point-precipitation data in a design storm method, a combination of probabilistic and deterministic methods are needed. The duration and magnitude of individual storm events are assumed to be probabilistic, while the internal assemblage of the design storm is often assumed deterministic. Additionally, the origin of severe storm events adds to the difficulty of developing a comprehensive analysis and establishing 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 gauge, the collected precipitation records are analyzed to determine the annual maximum rainfall depth for several durations of interest, such as 5 minutes, 1 hour, or 3 hours. Th information can then be arranged in an increasing order of magnitude for each storm duration fc the history of the gauge. 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.

Estimating Effective Rainfall

Effective rainfall, also known as rainfall excess, is that fraction of the total rainfall that 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. A discussion of watershed losses and loss-rate computation follows.

Infiltration

For severe design storm conditions, it can be argued that during a large portion of the storm, rainfall intensities exceed the infiltration capacity of the soil, and the soil infiltration rate is at a minimum during the storm's peak rainfall intensities. For example, Viessman et al. [41] show that the infiltration capacity essentially reaches equilibrium approximately 20 to 30 minutes after the beginning of rainfall. Consequently, for many cases in urban hydrologic studies, the equilibrium infiltration capacity value may be an appropriate value for the minimum watershed infiltration loss rate.

Several other infiltration models have been reported in the literature, including the model of Green and Ampt [7], the classic infiltration studies by Philip [22–27]. Freeze [4] provides a detailed discussion of numerical modeling procedures based on solving the fundamental infiltration equations. However, due to the difficulties in applying mathematical models, many hydrologic studies utilize a simplified effective rainfall estimation method such as assuming a constant or average value of infiltration for the entire storm. This concept is analogous to the phi-index approach of representing the watershed losses (see Figure 3) and has the added advantage of being simple to apply.

Depression Storage

Depression storage is associated with a rainfall volume that is permanently stored in natural or human-made obstacles to runoff. Estimates of this loss range from 0.01 to 0.5 inches of rainfall depending on land use, soil type, and other factors. Table 1 provides some estimates for 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. For design storm analyses where the antecedent rainfall is significant, or if the storm is of long duration so that the initial periods of the storm generate appreciable runoff, depression storage may be neglected in the determination of effective rainfall.

Vegetative Interception

This is the volume of rainfall retained on the leaves and stems of plants. That portion of the volume of rainfall that remains on the plant and does not contribute to runoff is the interception loss. 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 runoff-computation procedures, the interception loss is simply subtracted from the rainfall at the beginning of the rainfall period. Horton [11] determined values of interception for various magnitudes of precipitation and vegetative cover. For example, he measured interception

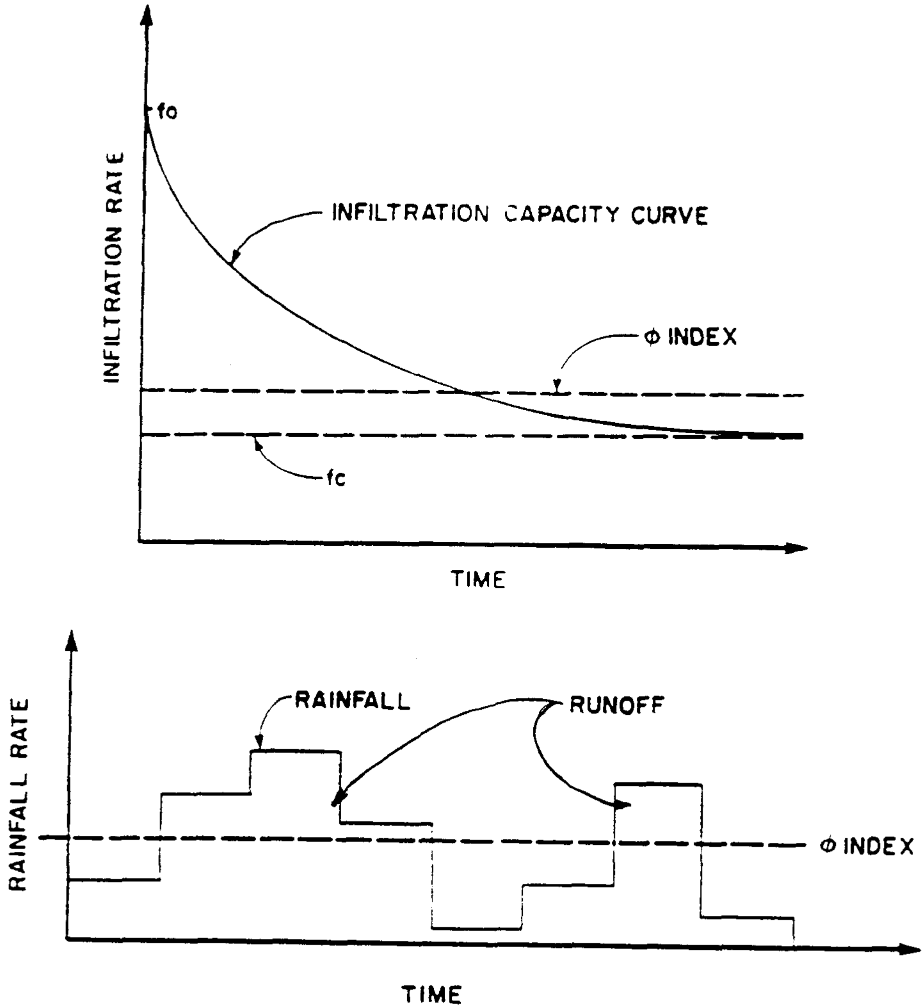


Figure 3. Loss-rate functions.

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 and open fields	0.20–0.60	0.30

losses for willow trees of 0.10 inches for a rainfall of 0.20 inches and 0.50 inches for a rainfall of 1.30 inches, indicating that there can be a correlation between total rainfall volume and the total interception loss.

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 the infiltration capacity goes from the initial rate to a constant rate may be as short as 10 to 15 minutes. Therefore, it is not unreasonable to assume the infiltration capacity to be constant over the entire storm duration. When the rainfall rate exceeds the infiltration capacity, the loss rate is assumed to equal the constant capacity, which is called the phi index. When the rainfall rate is less than the value of phi, the infiltration rate is assumed to equal the rainfall intensity.

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

$$f(t) = \begin{cases} I(t), & \text{for } I(t) < \phi \\ \phi, & \text{for } I(t) \geq \phi \end{cases} \tag{2}$$

where $f(t)$ = loss rate,
$I(t)$ = storm rainfall intensity
t = time
ϕ = phi index

If measured storm data are available, the value of the phi index can be determined by separating baseflow from the total runoff volume, computing the volume of direct runoff, and then finding the value of phi that results in the volume of rainfall excess being equal to the volume of direct runoff. A watershed mean phi index can then be computed as the average of storm event phi values. Where measured storm-event data are not available, the ultimate capacity of Horton's equation, fc, can be used (fc is defined below). Typical values of the ultimate capacity for bare soils are given in Table 1 and can be used as estimates of phi. For an individual watershed, actual ultimate capacities can vary considerably from these values. Furthermore, these values represent ultimate capacities, and the phi-index value is usually somewhat higher, as indicated in Figure 3. However, the difference is small and is probably not significant when compared to the variations in fc that are evident from the tabulated values.

Loss-Rate Functions

Constant Proportion Loss Rate

The use of a constant proportion loss-rate, where the effective rainfall is estimated to be a constant proportion of precipitation, is reported for several studies [29, 30, 42]. Schilling and Fuchs [29] recommend "simplifying the loss rate model with a correct overall runoff coefficient" as a means of increasing computational efficiency in rainfall-runoff models . . . without seriously affecting the modeling accuracy." Williams et al. [42] used a unit-hydrograph method to analyze rainfall-runoff data where "effective rainfall was taken to be a constant proportion of total rainfall" and compared the unit-hydrograph modeling results to results from a more complex model, noting that "the two calculated hydrographs are very similar."

Scully and Bender [30] found that in using a unit-hydrograph procedure with given rainfall-runoff data, the infiltration loss rates were primarily a function of the rainfall intensity. Because interception storage capacity is usually satisfied early in the storm, they reasoned that some other phenomenon must account for the changes in the infiltration rates. Scully and Bender proposed

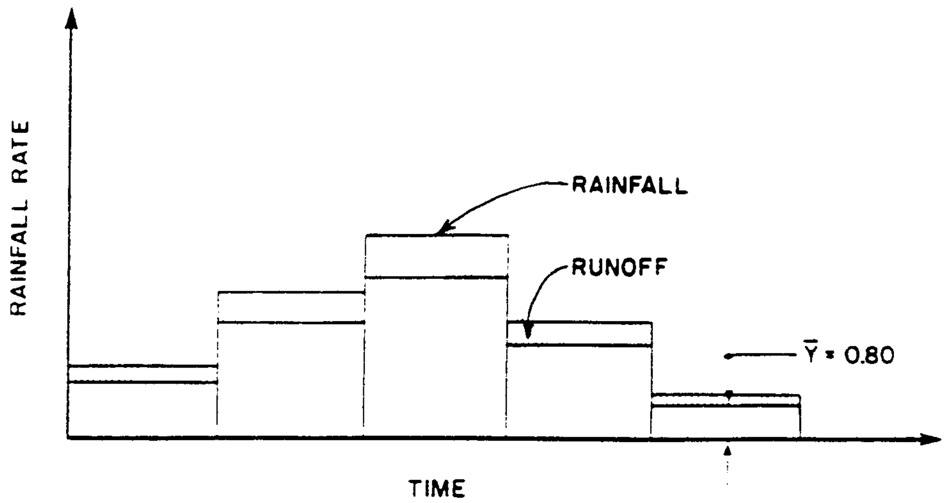


Figure 4. Constant fraction loss function.

that this phenomenon might be attributed to surface detention, which acts like depression storage, except that the surface detention is stored only temporarily and increases as the rainfall intensity increases.

The loss-rate function f(t) associated with the constant proportion loss-rate procedure for estimation of effective rainfall is given by (see Figure 4)

$$f(t) = \bar{Y}I(t) \tag{3}$$

where f(t) = loss rate
I(t) = storm rainfall intensity
t = time
$\bar{Y}$ = calibration constant with $0 < \bar{Y} < 1$

Coupled Phi-Index and Constant Proportion Loss-Rate Function

The phi-index and constant proportion loss-rate methods are both single-parameter loss functions. A hybridization of the two techniques combines both methods by using the phi index as an upper bound to the loss rate determined from the constant proportion loss-rate approach. That is, the loss rate f(t) is defined by

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

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 [13].

Horton Loss Rate

The Horton loss function of Figure 3 is described by the three-parameter model

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

where fo = initial loss rate
fc = ultimate loss rate
k = time rate of decay fromm fo to fc

Exponential Loss-Rate Function

Another three-parameter loss-rate function is reported by Unver and Mays [33], who suggest that the optimum parameter values of loss-rate functions and the optimum unit hydrograph can be determined when using the loss-rate function f(t) defined by

$$f(t) = A[I(t)]^E \tag{6}$$

where I(t) = rainfall intensity at time t
A and E = calibration constants

Initial Abstraction

The total loss used in the storm rainfall-runoff budget is usually separated into two parts: the initial abstraction, Ia, and a time-varying loss. Two models for the initial abstraction estimation that can be coupled with the previously discussed loss-rate functions are:

1. Total rainfall up to a total depth of Ia
2. Percentage of total rainfall up to a total depth of Ia

In the first initial abstraction model, all precipitation is budgeted toward the total rainfall depth exceeding Ia. The second Ia model accounts for a fixed percentage of rainfall unil the total Ia 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 Ia method is a one-parameter model, whereas the second method is a 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 that 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 using a dozen or more calibration parameters. However, it is unclear if using such a sophisticated approaches produces an increase in modeling accuracy.

It should be noted that the initial abstraction model can have an important influence in rainfall-runoff analyses of storms that have peak rainfall intensities occurring during the early portion of the storm. In such cases, both the loss-rate and initial abstraction models are in operation, resulting in the hydrologic model having two similarly acting functions, with up to five parameters requiring calibration simultaneously.

SCS Loss Model

The SCS runoff-depth equation relates runoff depth to rainfall depth by the following equation:

$$R = (P - Ia)^2/(P - Ia + S) \tag{7}$$

where R = runoff depth, in.,
P = precipitation depth, in.

Maximum watershed retention S is given by

$$S = 1000/CN - 10 \tag{8}$$

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.

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

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

For large values of S, the initial abstraction can be substantial and may appear to be irrationally large. In spite of this, 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

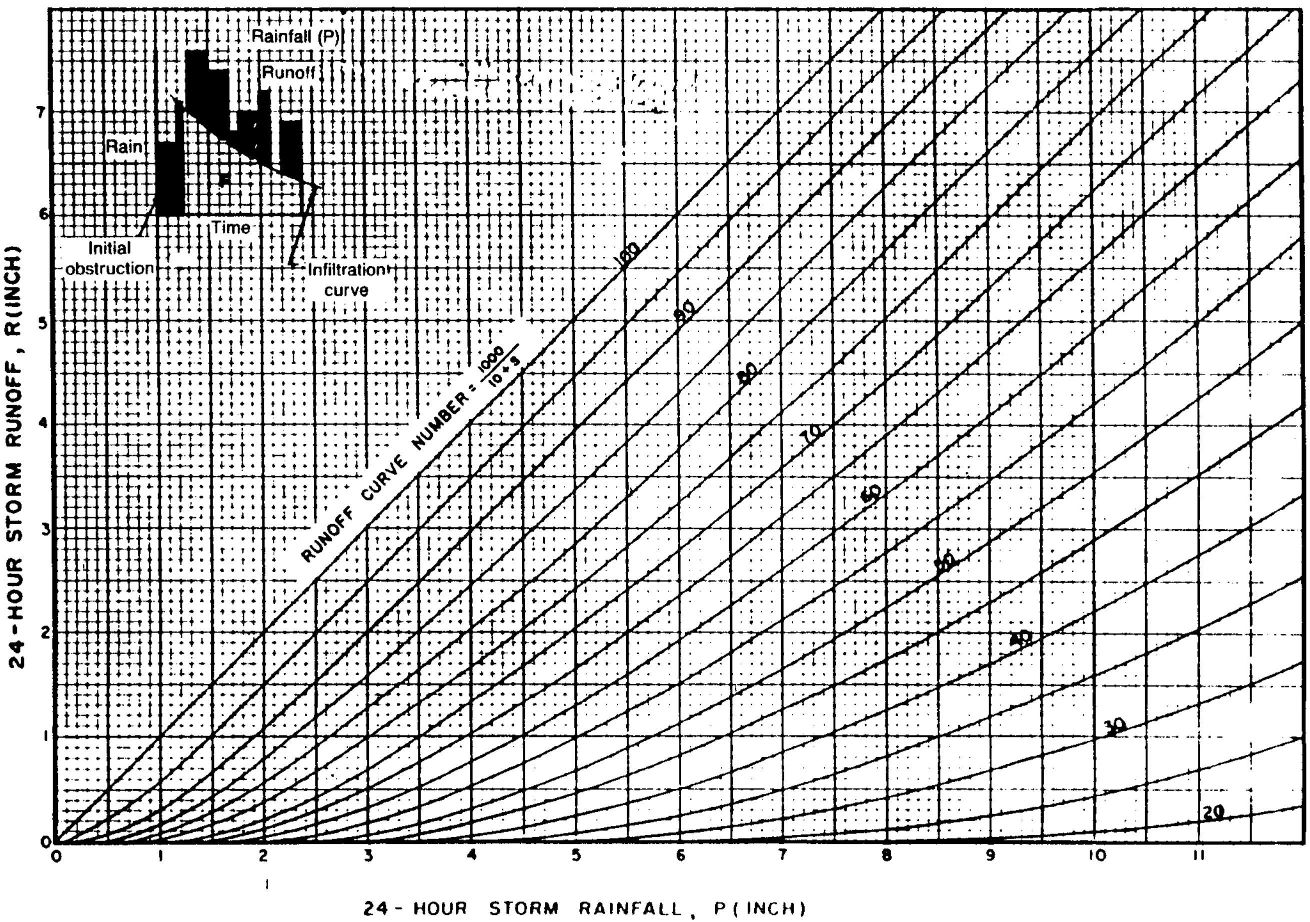
A major factor affecting infiltration is the condition of the soil itself. The soil-surface characteristics, the ability to transmit water through subsurface layers, and the available soil storage capacity all control the infiltration rate. The SCS has classified more than 4,000 soil types into four general categories to provide a general purpose classification of infiltration rates and corresponding runoff rates. The soil groups are defined below:

- *Low runoff potential:* Soils have a high infiltration rate even when thoroughly wetted. Consists chiefly of deep, well-drained gravels and sands.
- *Soils having moderate infiltration rates when thoroughly wetted:* Consist mainly of moderately deep, well-drained soils with moderately fine to moderately coarse texture.
- *Soils having slow infiltration rates when thoroughly wetted:* Consist chiefly of soils with a layer that impedes the downward migration of water and soils with moderately fine to fine texture.
- *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, and shallow soils over a nearly impervious layer.

Soil infiltration rates can be estimated for each of the soil groups by laboratory studies and field measurements. Such measurements show that given a sufficient water supply, a initially dry soil will have an associated infiltration rate that decreases with time. Consequently, if the soil is subject to a continuous rainfall rate that exceeds the infiltration capacity, the infiltration loss rate will decrease with increasing storm duration, as is shown in Figure 5. Relative minimum infiltration rates, which depend on the condition and cover of the soil, are given in Table 2.

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



,ure 5. SCS rainfall-runoff relationships.

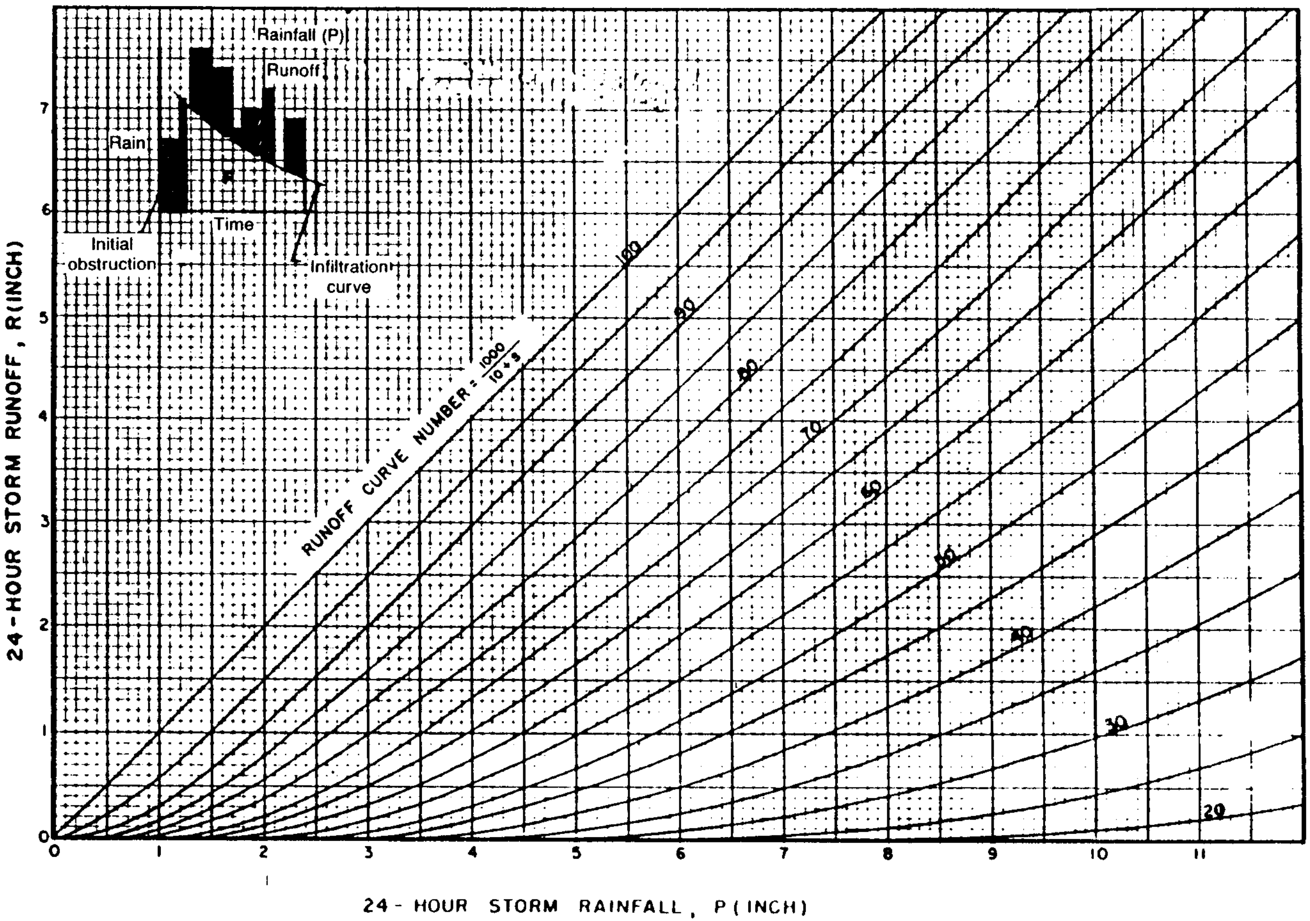


Figure 5. SCS rainfall-runoff relationships.

Table 2
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

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 given in Table 3. Values given are for the actual percentage of area covered by an impervious surface; however, the actual impervious area may need

Table 3
Actual Impervious Cover

Land Use[1]	Range (%)	Recommended Value For Average Conditions[2] (%)
Natural or agriculture	0–10	0
Single-Family Residentials[3]		
100,000 SF (2.5 acre) lots	5–15	10
40,000 SF (1 acre) lots	10–25	20
20,000 SF (1/2 acre) lots	30–45	40
7,200–10,000 SF lots	45–55	50
Multiple-Family Residentials		
Condominiums	45–70	65
Apartments	65–90	80
Mobile-home park	60–85	75
Commercial, downtown, business, or industrial	80–100	90

[1] Land use should be based on ultimate development of the watershed. Long-range master plans for the County and incorporated cities should be reviewed to ensure reasonable land use assumptions.

[2] Recommended values are based on average conditions, which may not apply to a particular study area. The percentage impervious may vary greatly even on comparable sized lots due to differences in dwelling size, improvements, etc. Landscape practices should also be considered, as it is common in some areas to use ornamental gravels underlain by impervious plastic materials in place of lawns and shrubs. A field investigation of a study area should always be made, and a review of aerial photos, where available, may assist in estimating the percentage of impervious cover in developed areas.

[3] For typical horse-ranch subdivisions, increase impervious area 5% over the values recommended above.

to be reduced due to the local drainage practices. For example, runoff from an impervious surface where infiltration may drain across a pervious surface where infiltration may occur. To account for this infiltration, the actual impervious area may be reduced by a factor such as 10%. 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 CN's includes CN's 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 CNs can be adjusted using Table 3. The pervious CN is obtained from the CN table and used with Table 3 to obtain the composite CN. Table 3 is used when all or most of the impervious cover is connected directly to the drainage system. In some localities, drainage policies require parts of the impervious cover to be directed to pervious areas where the water is spread as sheet flow over the pervious area. For example, rooftop runoff can be directed to adjacent grassy areas. Where the impervious land cover is unconnected, the composite CN can be computed from the pervious area CN, the percentage of imperviousness, and the percentage of the imperviousness that is unconnected using Figure 6. Figure 6b should only be used when the total imperviousness is less than 30%; values should not be extrapolated for percentages greater than 30%.

ESTIMATING PEAK DISCHARGE

Some rainfall-runoff analysis problems require only estimates of maximum flowrates (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 necessary input to most hydrologic models, including the rational method. In addition to basic rational-method computations, methods for analyzing subdivided watersheds are discussed in this section.

Estimation of Time of Concentration

The time of concentration (Tc) 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). Tc 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 Tc. Most methods use two or more of the following factors: (a) flow length, (b) flowpath slope, (c) land use or a representative surface roughness, and (d) 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 Tc should reflect the primary flow regime in the flowpath being analyzed. Given the wide array of methods available, it is important to use a Tc 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 that can be used to estimate the initial subarea Tc. The simplest method is to estimate a Tc at the point of concentration by using a generalized overland flow formula. Various empirical equations are also in use. A variation of the Kirpich [15] formula that is widely used has the form

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

where L = length of initial subarea flowpath, ft
H = drop in elevation along flowpath, ft
k = coefficient depending on development type
E = constant exponent, usually 0.2

Due to the inherent inaccuracy in the determination of a generalized equation for computing Tc 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

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

where L = hydraulic length, ft
S = slope in ft/ft
Tc = time of concentration, hr

Equation 11 is based on data from watersheds in Tennessee that have areas from 1 to 112 acres and slopes from 3% to 10%. The computed times of concentration should be multiplied by 0.4 watersheds, where the overland flowpath is concrete (or 0.2 for asphalt) and the channel is lined.

The Federal Aviation Agency [36] proposed the equation

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

where C = runoff coefficient for the rational method,
S = average surface slope (%),
L = characteristic flow length

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

$$Tc = 0.94\ L^{0.6}n^{0.6}i^{-0.4}Sf^{-0.3} \tag{13}$$

where L = flow length, ft
n = Manning's roughness coefficient
i = excess rainfall rate (in./hr)
Sf = 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 Tc estimation of gutter flow in urban areas.

For watersheds having flowpaths over mixed land cover, the time of concentration (Tc) can be estimated by summing the runoff travel times (Tt) through the several flowpaths as the flood peak travels downstream to the watershed outlet. These flowpaths include overland flow, street flow, pipe flow, and open-channel flow in natural or improved channels. Tc 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 watersheds that have flowpaths of mixed land covers.

For upland flowpaths, 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 (ft/sec) can be computed using the relationship

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

where Tt = travel time, sec
L = length, ft

The total travel time for the watershed is the sum of the individual travel times:

$$Tt = \sum_{i=1}^{n} L_i/V_i \tag{15}$$

in which n is the number of flow paths. All travel times computed for the individual flowpaths 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 ($i = 1, 2, \ldots, m$) 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 Tc values are determined by

$$Tc(i + 1) = Tc(i) + Tt(i, i + 1) \tag{16}$$

where $Tc(i + 1)$ = Tc at node number $i + 1$
$Tc(i)$ = Tc at node number i
$Tt(i, i + 1)$ = 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 $Tt(i, i + 1)$, 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

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

where $L(i, i + 1)$ = length of channel linking nodes i and $i + 1$
$V(i, i + 1)$ = 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 of variable size. In the past, this method was directly applied to watersheds with sizes greater than several square miles. 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 = peak runoff rate in cubic feet per second (cfs) at the point of concentration,
C = runoff coefficient representing the area-averaged ratio of runoff to rainfall rates,
I = time-averaged rainfall intensity, in./hr, corresponding to the time of concentration,
A = drainage area (acres)

The values of the runoff coefficient and rainfall intensity are based on a study of 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 (a) rainfall intensity for a storm of specified duration and selected return frequency; (b) drainage area characteristics of size, shape, and slope, and (c) 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-in./hr is equal to 1.008 cfs, the rational-method equation gives the peak flowrate in cfs (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 (a) a constant value depending on soil-cover type and quality or (b) a function of rainfall intensity and soil-cover type and quality. Table 4 lists typical C values for use with the rational method.

The second class of runoff coefficient representations relate the C value to the rainfall intensity. One approach used for urban design purposes is to assume that the watershed loss rate is equal to the infiltration loss rate that 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 a function of the impervious and pervious area fractions, a characteristic infiltration rate (Fp) 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

$$Cm = 0.85(Ai + (I - Fp)Ap/I) \tag{19}$$

where Cm = modified runoff coefficient
I = rainfall intensity, in/hr
Fp = infiltration rate for pervious-area fraction
Ai = impervious-area fraction
Ap = pervious-area fraction
0.85 = a calibrated (or assumed) coefficient to correlate rainfall and runoff frequencies

The infiltration rate for the pervious area (Fp) can be estimated for various combinations of soil type, cover, and antecedent moisture conditions.

When the drainage area is composed of several types of runoff surfaces, an area-averaged runoff coefficient can be developed as demonstrated by the following example.

Example: Area-Averaged Runoff Coefficient

A watershed is composed of 3.5 acres of paved parking lot and associated street system, 35.6 acres of a condominium development, and 12.5 acres of a neighboring apartment complex. The

Table 4
Runoff Coefficients for the Rational Formula for a Hydrologic Soil Group and Slope Range

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 $\frac{1}{8}$ 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 $\frac{1}{4}$ 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 $\frac{1}{3}$ 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 $\frac{1}{2}$ 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.

area-averaged runoff coefficient is estimated by tabulating each area fraction's contribution and computing an area-weighted average:

Area (acres)	Type of Surface	C	CA
3.5	concrete pavement	1.00	3.50
35.6	condominiums	0.67	23.85
12.5	apartments	0.77	9.63
Sum = 51.6			36.98

$$\text{Area-averaged } C = 36.98/51.6 = 0.72$$

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: (a) 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); (b) the peak runoff rate occurs when all parts of the drainage area are contributing to the runoff; (c) the design rainfall is uniform over the watershed area tributary to the point of concentration; and (d) 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 1 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, infiltration capacity of the soil, and 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, depression, and channel storage cannot be properly evaluated and may appreciably affect the estimate of the watershed peak rate of runoff.

UNIT-HYDROGRAPH METHODS

The unit-hydrograph method is an approach initially advanced by Sherman [31]. The keystone of the method is the assumption that watershed discharge is related to the total volume of runoff, and that the time factors that affect the unit-hydrograph shape are invariant. The basic unit-hydrograph theory was extended by Snyder [32] to transpose storm rainfall-runoff relationships from gaged watersheds to hydrologically and geographically similar watersheds that lack rainfall-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 are either unavailable or inadequate to provide a sound statistical analysis.

To determine the rainfall-runoff relationships to be transposed to ungaged watersheds, stream-gage 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 several similar 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. The same is true for most other areas of the United States.

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 (a) the critical storm rainfall pattern is uniformly distributed through the watershed; (b) there exists a direct proportionality between watershed runoff and the effective rainfall volume; (c) for any volume of effective rainfall occurring within a specified duration, the resulting runoff hydrograph is of a constant duration; and (d) 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 application and the wide range of applications available for master planning purposes.

Unit-Hydrograph Terminology

In order to discuss the development of unit hydrographs for both gaged and ungaged watersheds, the following definitions are presented (Figure 7 illustrates each of the unit-hydrograph concepts and definitions):

- *Effective Rainfall:* The total rainfall less losses, which includes infiltration losses, evaporation, transpiration, absorption, and detention. This part of the rainfall runs off the watershed surface in a relatively brief time period. This is also referred to as *rainfall excess*, with the volume of rainfall excess equaling the volume of direct runoff.
- *Unit Hydrograph:* A hydrograph for a point of concentration on a watershed showing 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 with respect to 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 is 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:* The 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% 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, however, 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 of 1 inch occurring in a unit interval of 1 hour, the ultimate discharge is 645 cfs for every 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.

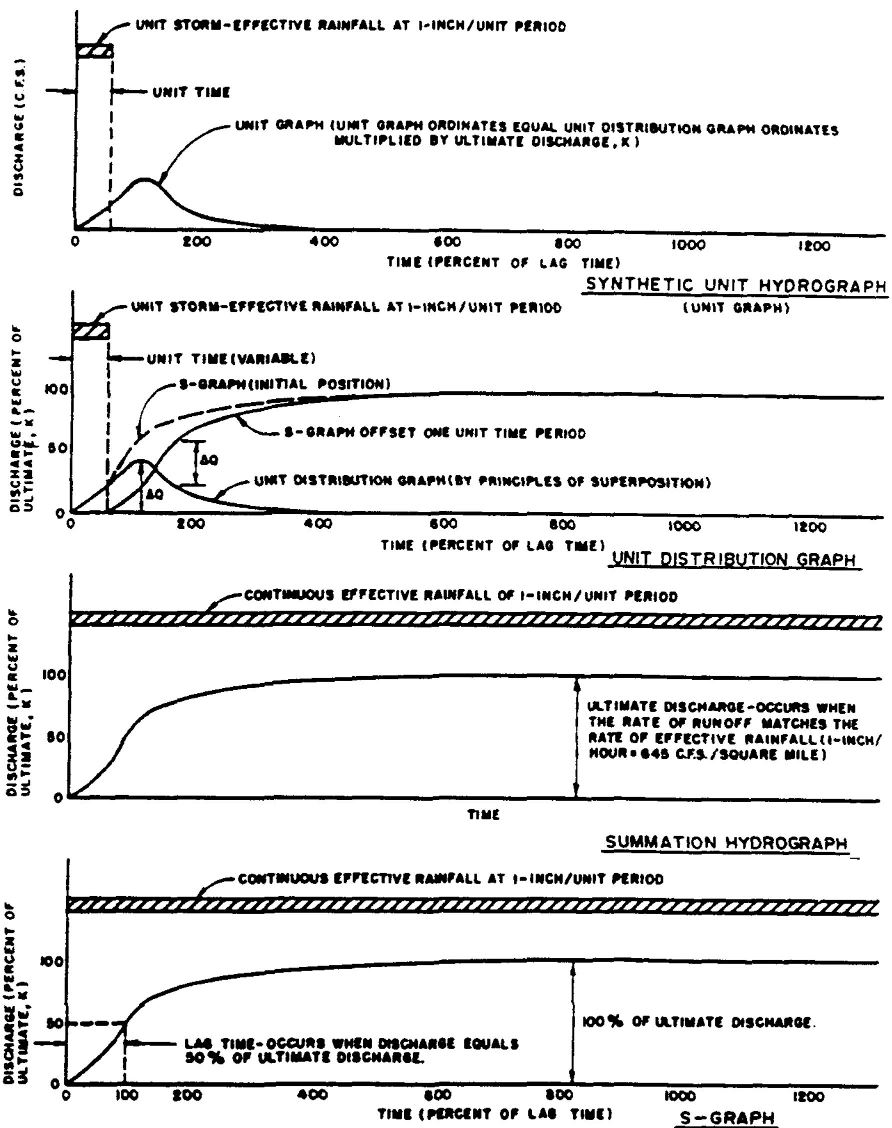


Figure 7. Definitions used in the unit-hydrograph concept.

Conceptual Evaluation of Unit Hydrographs

A simple example can be used to illustrate the unit-hydrograph concepts. A 2-hour rainfall of 1.4 inches fell on a 1.600 acre watershed. producing the triangular direct runoff hydrograph shown in Figure 8. Baseflow is assumed to be zero throughout the storm. Converting the area under the

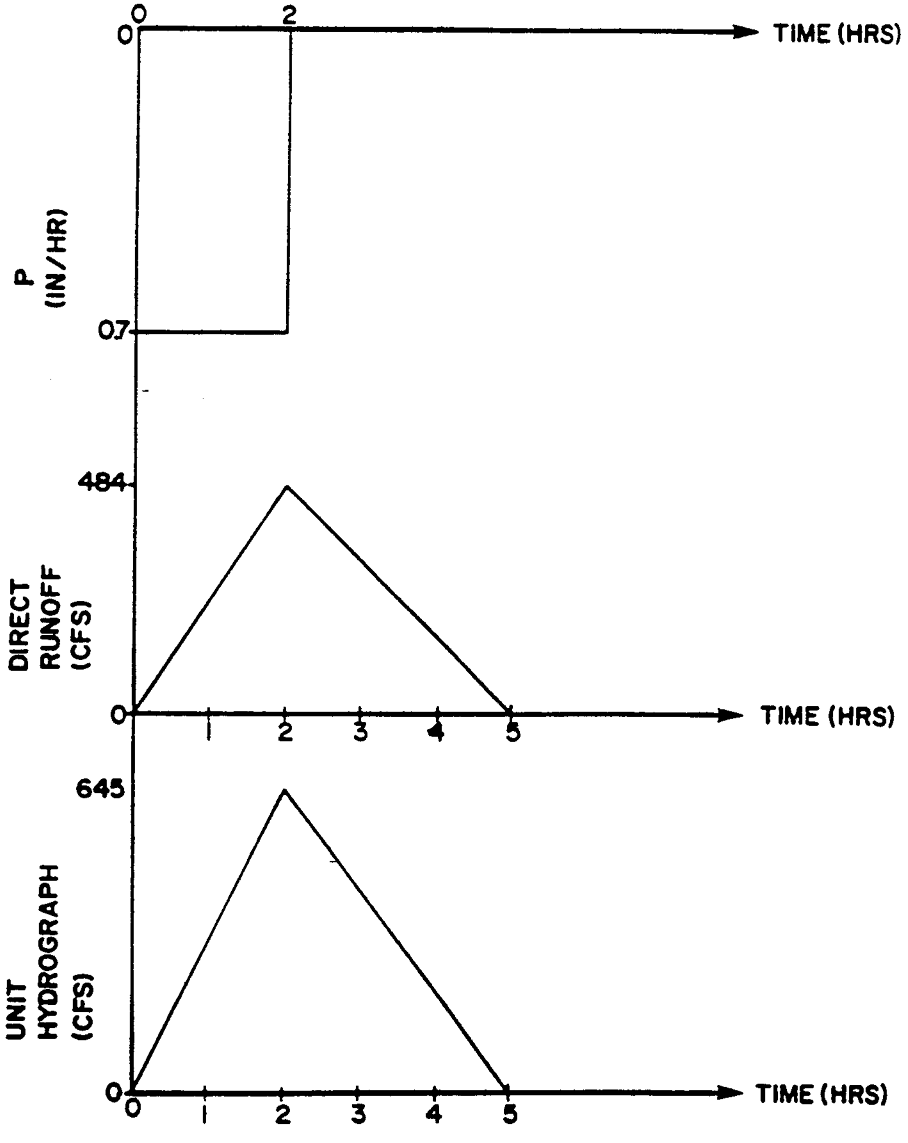


Figure 8. Unit hydrograph, conceptual example.

direct runoff hydrograph to units of inches (or area-inches) yields a direct runoff depth of 0.75 inches. According to the definition provided above, a unit hydrograph has a runoff depth of 1 inch; therefore, the unit hydrograph is found by dividing each ordinate by 0.75. The unit hydrograph is also shown in Figure 8. In order to find the duration of the unit hydrograph, it is necessary to compute the time distribution of rainfall excess (i.e., effective rainfall). The phi-index method can be used to separate the rainfall into rainfall excess and losses. While the total rainfall is 1.4 inches and the intensity is 0.7 in./hr, the depth of rainfall excess just equals the depth of direct runoff, or 0.75 inches. Thus, the equivalent depth of losses is $(1.4 - 0.75) = 0.65$ inches over the 2-hour storm duration, and the phi-index loss rate is 0.325 in./hr. Since the rainfall excess is constant over the 2-hour period, the unit hydrograph has a unit duration of 2 hours; it is referred to as a 2-hour unit hydrograph because the duration of the rainfall excess equals 2 hours.

Convolution

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. However, a strict interpretation of the definition of the unit hydrograph is that it represents the watershed response under a restrictive set of conditions, specifically a constant rate of rainfall excess occurring uniformly over the watershed during a specific time duration. Rarely, if ever, will actual storms meet these restrictive conditions, and certainly the characteristics of synthetic storms such as those discussed above will not correspond to the unit storm. Thus, a mechanism is needed for converting a rainfall-excess distribution that is different from the unit storm for which the unit hydrograph was developed.

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 into 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 process is illustrated in the previous example. The unit hydrograph is applied to each individual burst of rainfall excess. After the multiplication-translation operation has been performed on each burst of rainfall, the individual direct-runoff hydrographs are summed to get the total direct-runoff hydrograph.

In practice, the convolution process is used in modeling of 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. The above procedure can then be applied to each storm to derive unit hydrographs for each storm and an average watershed unit hydrograph can be 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 by the above procedure. Thus. methods have been developed so that complex storms with non-uniform 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. Other methods require the assumption of a particular model form, which is then fit by the method of moments. For example, Aron and White [1] used a gamma function, which has the general shape of a unit hydrograph, and 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.

Forming 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 for the most part been located on large drainage basins. Furthermore, the process of developing a unit hydrograph would be very difficult because urban watersheds are constantly being changed and thus watershed characteristics over the duration of the data record would not be constant. The 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 [35]. 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 presented. It is important to recognize the two distinct phases involved: analysis and design. In the analysis phase, the form of the 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% 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

$$\text{lag} = C_t L^m Lc^m S^{-m/2} \tag{20}$$

where lag = watershed lag, hr.
- Ct = calibration constant
- L = length of longest watercourse, miles
- Lc = length along longest watercourse, measured from the point of concentration to a point opposite the watershed area centroid, miles
- m = calibration constant

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 nonuniform, 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 that 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 meanders 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.

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 commonly used value for m is 0.30.

The basin factors of Table 6 were determined by the U.S. Army Corps of Engineers by calibration with major storm hydrographs from several Southern California watersheds. The watershed lag relationship of Equation 20 is illustrated in the plot of Figure 9.

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. A typical S-graph assumed applicable for Southern California valley watersheds is shown in Figure 10. Various S-graphs were developed from calibration studies of measured significant rainfall-runoff events. There 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% of ultimate discharge at 100% 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

$$K = 645A/T \tag{21}$$

where K = watershed ultimate discharge, cfs
A = drainage area, square miles
T = unit time period, hr

To use an S-graph, a watershed lag is estimated from Equation 20 and Figure 9. A duration is selected (usually 15% to 25% 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.

The SCS Unit Hydrograph

The above unit-hydrograph analysis process has been used many times using a number of data bases. Watershed and regionally averaged synthetic unit hydrographs have been developed for use at ungaged locations. The more data used to calibrate a synthetic unit hydrograph, the greater the accuracy that can be expected when the synthetic unit hydrograph is used at ungaged locations.

The Soil Conservation Service (SCS) 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 (square miles), the runoff volume Q (in.), and the time to peak Tp (hr) in order to dimensionalize it. The dimensionless unit hydrograph relates the ratios of q/qp and t/Tp, where q is the discharge at any time t and qp is the peak discharge, which occurs at time Tp. The peak discharge of the unit hydrograph is computed by the following:

$$qp = 484AQ/Tp \tag{22}$$

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

$$Tp = 0.6Tc \tag{23}$$

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

		DRAINAGE AREA SQ MI	L MILES	L_{CA} MILES	S FT./MI	LAG HOURS
1	SAN GABRIEL RIVER AT SAN GABRIEL DAM NO 1*	162	23.2	11.6	350	3.3
2	WEST FORK SAN GABRIEL RIVER AT SAN GABRIEL DAM NO 2	40.4	9.3	4.2	450	1.6
3	SANTA ANITA CREEK AT SANTA ANITA DAM	10.8	5.8	2.5	890	1.1
4	SAN DIMAS CREEK AT SAN DIMAS DAM	16.2	8.6	4.8	440	1.5
5	EATON WASH AT EATON DAM	9.5	7.3	4.4	600	1.3
6	MURRIETA CREEK AT TEMECULA	220	27.2	10.3	95	4.0
7	SANTA CLARA RIVER NEAR SAUGUS	355	38.0	15.8	140	5.6
8	TEMECULA CREEK AT PAUBA CANYON**	168	28.0	11.3	150	3.7
9	SANTA MARGARITA RIVER NEAR FALLBROOK	645	46.0	22.0	105	7.3
10	EAST FULLERTON CREEK AT FULLERTON DAM	3.1	3.2	1.7	140	0.8
11	TUJUNGA CREEK AT BIG TUJUNGA DAM NO 1	81.4	15.1	7.3	280	2.5
12	SANTA MARGARITA RIVER AT YSIDORA	740	61.2	34.3	85	9.5

* EXCLUDES AREA ABOVE SAN GABRIEL DAM NO. 2

** PALOMAR MOUNTAIN PORTION ENTIRE AREA IS 319 SQUARE MILES, OF WHICH 151 SQUARE MILES DID NOT CONTRIBUTE APPRECIABLE FLOOD FLOWS DURING THE 1937 AND 1940 OCCURRENCES

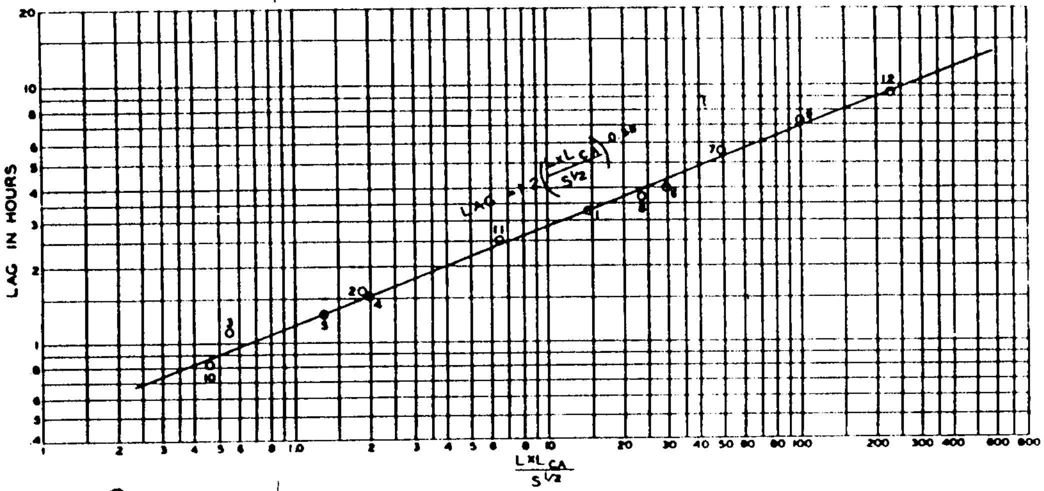
TERMINOLOGY

L = LENGTH OF LONGEST WATERCOURSE

L_{CA} = LENGTH OF LONGEST WATERCOURSE MEASURED UPSTREAM, TO POINT OPPOSITE CENTER OF AREA

S = OVER-ALL SLOPE OF DRAINAGE AREA BETWEEN HEADWATERS AND COLLECTION POINT

LAG = ELAPSED TIME FROM BEGINNING OF UNIT RAINFALL TO INSTANT THAT SUMMATION HYDROGRAPH REACHES 50% OF ULTIMATE DISCHARGE



STATE OF CALIFORNIA
THE RESOURCES AGENCY OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
SOUTHERN DISTRICT

FLOOD HYDROLOGY–CASTAIC RESERVOIR

LAG RELATIONSHIP
DRAINAGE AREAS IN SOUTHERN CALIFORNIA
1963

Fig. 9 . *LAG RELATIONSHIP TO GEOMETRIC DATA*

Figure 9. Lag relationship to geometric data.

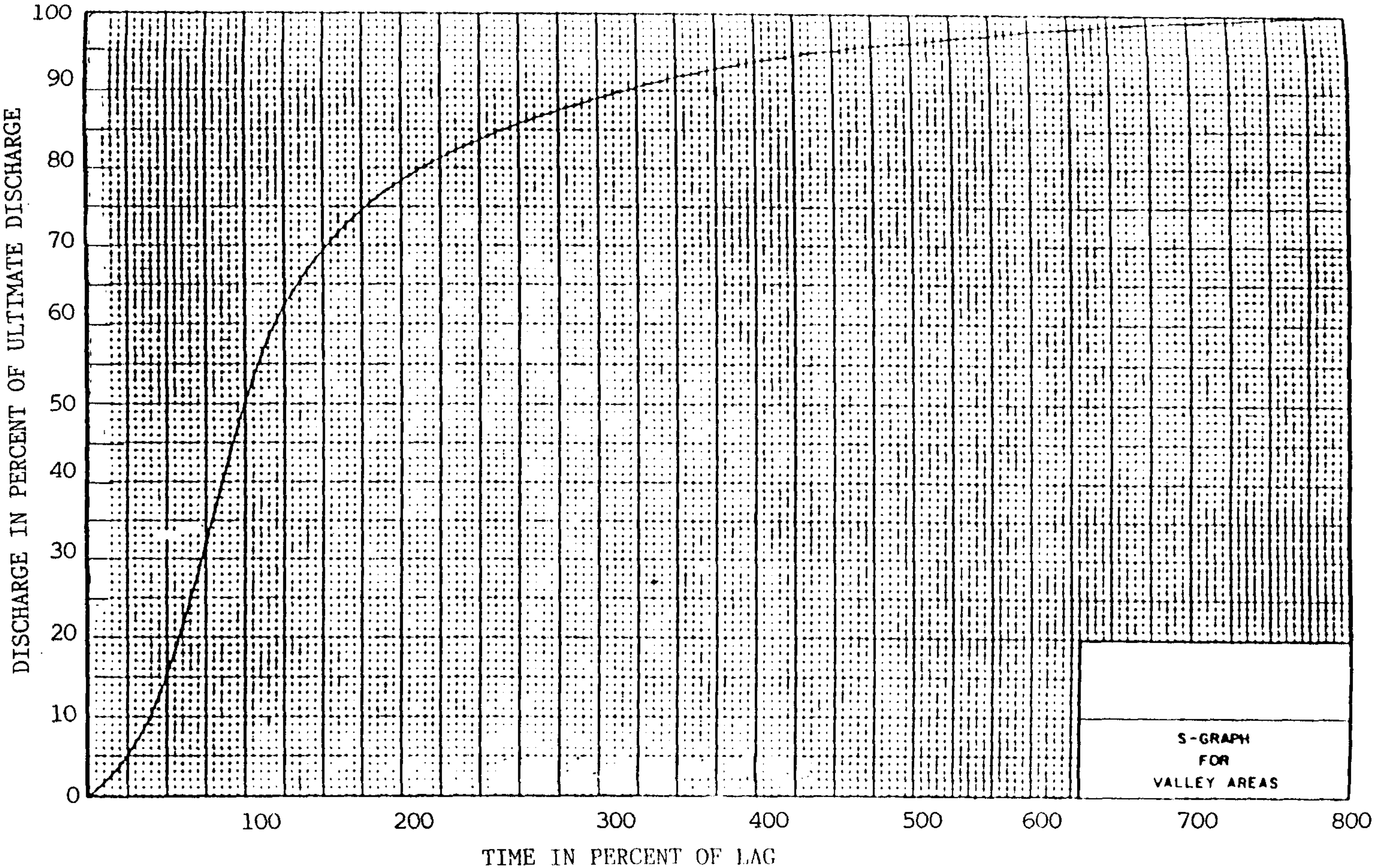


Figure 10. S-graph for valley area.

hydrograph, the volume of runoff equals 1 inch. A tabulation of the SCS dimensionless unit-hydrograph ratios is given in Table 6. From the table, it is evident that whatever the condition or classification of the watershed, the ratio of the time to peak to total unit-hydrograph duration is a constant. Additionally, it is evident that the volume of runoff under the rising limb of the unit hydrograph is a constant 37.5% of the total runoff volume and that the peak-rate factor associated to the unit rainfall is a constant value of 484. The variability of the SCS unit hydrograph with respect to other peak-rate factors was investigated by McCuen and Bondelid [18]. In their study, methods to estimate a peak-rate factor for ungaged watershed are proposed using a gamma function distribution for the unit hydrograph and the estimation of approximate watershed storage effects in the determination of the volume of runoff defined under the rising limb. As with many other unit-hydrograph schemes, the assumption of linearity is once again utilized to develop a runoff hydrograph. Watershed peak-rate factors can be estimated by time-area analysis and attempts to duplicate runoff hydrographs.

Table 6
Ratios for the SCS Unit Hydrograph

Time Ratio	Discharge Ratio	Mass-Curve Ratio
0.00	0.000	0.00
0.10	0.030	0.001
0.20	0.100	0.006
0.30	0.190	0.012
0.40	0.310	0.035
0.50	0.470	0.065
0.60	0.660	0.107
0.70	0.820	0.163
0.80	0.930	0.228
0.90	0.990	0.300
1.00	1.000	0.375
1.10	0.990	0.450
1.20	0.930	0.522
1.30	0.860	0.589
1.40	0.780	0.650
1.50	0.680	0.700
1.60	0.560	0.751
1.70	0.460	0.790
1.80	0.390	0.822
2.00	0.280	0.871
2.20	0.207	0.908
2.40	0.147	0.934
2.60	0.107	0.953
2.80	0.077	0.967
3.00	0.055	0.977
3.20	0.040	0.984
3.40	0.029	0.989
3.60	0.021	0.993
3.80	0.015	0.995
4.00	0.011	0.997
4.50	0.005	0.999
5.00	0.000	1.000

Variations on the SCS Unit Hydrograph

The concept of time to peak (Tp) is geometrically related to the watershed lag time. Here lag time follows the definition of lag, which is the time from the center of mass of the effective rainfall to the peak of the runoff hydrograph. It has been shown that the time to peak Tp and watershed lag can be related by the general formula

$$\mathrm{Tp} = C_0 \text{ lag} \tag{24}$$

where C_0 is a constant. Studies have shown that C_0 varies from about 0.6 to 1.25. For example, the County of San Diego, California [28] uses the empirical watershed lag formulas of Equations 20 and 21 plus the relationship

$$\mathrm{Tp} = 0.862 \text{ lag} \tag{25}$$

Equation 25 is based on the geometric relationships for the unit hydrographs shown in Figure 11, which include the triangular version of the SCS curvilinear unit hydrograph. From Figure 11, it is assumed that

$$\mathrm{Tr} = 1.67\mathrm{Tp} \tag{26}$$

$$\mathrm{Tb} = 2.67\mathrm{Tp} \text{ (triangular unit hydrograph)}$$

The Espey-Altman Unit Hydrograph

Additional insight into the mechanics of the unit hydrograph is gained by examining the Espey and Altman approach. This version of the unit hydrograph involves a set of five parameters which

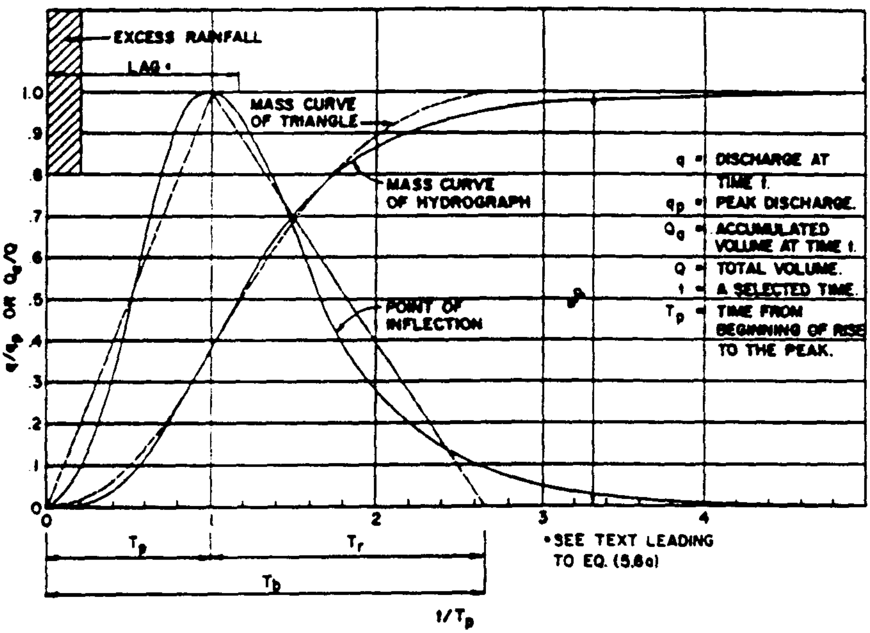


Figure 11. SCS unit-hydrograph definitions.

are defined according to Figure 12. The variance relationships are

$$
\begin{aligned}
Tp &= 3.1L^{0.23}S^{-0.25-0.18}K^{1.57} \\
T_B &= 125{,}890AQ^{-0.95} \\
Q &= 31{,}620A^{0.96}Tp^{-1.07} \\
W_{50} &= 16{,}220A^{0.93}Q^{-0.92} \\
W_{75} &= 3{,}240A^{0.79}Q^{-0.78}
\end{aligned}
\tag{27}
$$

where L = main channel length, ft
H = drop in elevation along channel
S = channel slope
I = impervious area fraction
A = watershed area, square miles
Tp = time to peak, min
TB = base time, min
K = dimensionless channel conveyance factor
W50 and W75 = hydrograph widths at 50% and 75% of the peak discharge, respectively

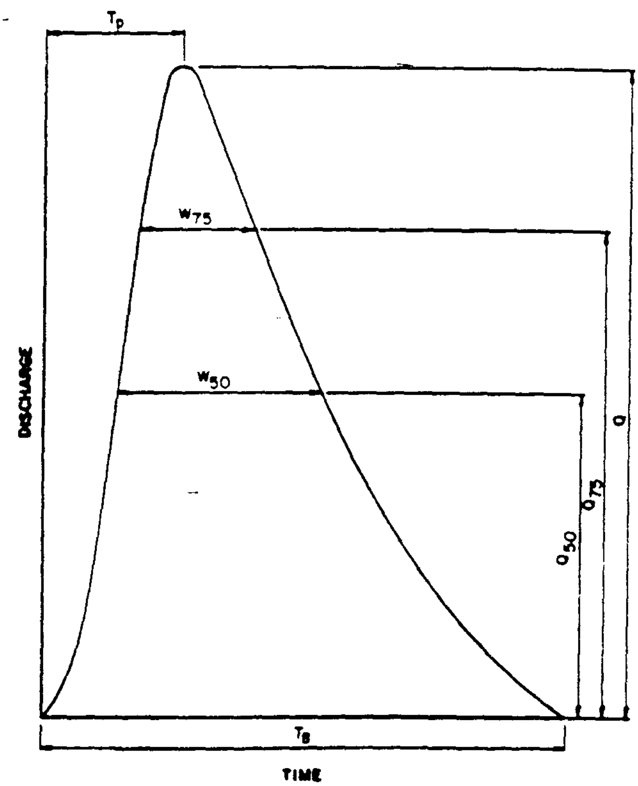


Figure 12. Espey-Altman unit-hydrograph definitions.

The Espey unit hydrograph is sensitive to the channel conveyance factor (K) in a fashion analogous to the sensitivity of the unit hydrograph lag parameter to the basin n* factor. Each of the factors used in Equations 27 require calibration in order to properly evaluate their respective correlation to local watershed characteristics. Additionally, the K factor requires calibration using local rainfall-runoff data in order to develop definitions similar to those used for the basin factor of Table 5.

Synthetic Runoff-Hydrograph Development

The various modeling elements of design storm pattern, point rainfall determination, watershed lag estimation, and the unit-hydrograph approximation are combined in the convolution of effective rainfall with the unit hydrograph. Figure 13 illustrates the convolution procedure for developing a synthetic runoff hydrograph. Each unit interval (or unit period) of rainfall from the design storm pattern is split into the effective unit rainfall and the corresponding watershed unit loss. The resulting effective rainfall storm pattern is then combined with the assumed watershed unit hydrograph by convolution to produce the synthetic runoff hydrograph. Figure 13 shows a 12-unit-period storm pattern and the corresponding unit-period runoff hydrographs. Summing together the 12-unit-period runoff hydrographs (i.e., assuming the system to respond linearly to the design storm input) results in the design storm runoff hydrograph.

It should be noted that in the convolution approach, each unit-period runoff hydrograph has the same base period regardless of the magnitude of the associated unit effective rainfall. Also, each unit-period runoff hydrograph has a constant time-to-peak (Tp). Each of the unit-period runoff hydrographs are directly related to the volume of the associated effective rainfalls. Additionally, in the summation process, it is assumed that there are negligible restrictions to flow for the summed runoff hydrographs (these restrictions would occur due to channel capacity problems and watershed detention effects). Each of these concerns should be addressed after the watershed runoff hydrograph is generated in order to ascertain whether the computed results are reasonable for the watershed conditions being studied. In many study situations, a single runoff-hydrograph generation would be inappropriate due to additional watershed storage conditions, and a more complex watershed model is required that includes submodels for simulating channel and basin routing and storage effects.

Other contributions to the runoff hydrograph such as baseflow and snowmelt can be summed with the synthetic design storm runoff hydrograph. Generally, such considerations are neglected in the study of urban watersheds; however, detailed discussions of related hydrologic processes are contained in standard texts.

The Rational Method as a Unit-Hydrograph Method

The rational method can be interpreted as a design storm unit-hydrograph method. The design storm pattern is developed by using a selected return frequency rainfall intensity-duration curve. At a point of concentration with time of concentration, Tc, the rational-method design storm pattern is constructed from an intensity-duration curve by first determining the total amount of rainfall (i.e., unit rainfalls) that falls in several successive unit periods, each of duration Tc. The next step is to arrange these several unit rainfalls into the rational-method design storm pattern by placing the largest unit rainfall as the first unit, followed by the second largest unit rainfall, and so forth until a sufficiently long design storm pattern is developed (usually about 1 hour in total length, but it may be longer depending on the various stream confluence Tc values).

Using the area-averaged loss rate, Fm, the design storm unit effective rainfalls are calculated by subtracting the appropriate proportion of Fm from each unit rainfall. It is noted that the design storm unit rainfalls are given in units of inches of precipitation, whereas Fm is given as a rate (in./hr).

Rational-Method Unit-Hydrograph Application

The unit hydrograph corresponding to the rational method is a triangle with base 2Tc and a peak occurring at time Tc (see Figure 14). For a unit period of duration equal to Tc and a unit

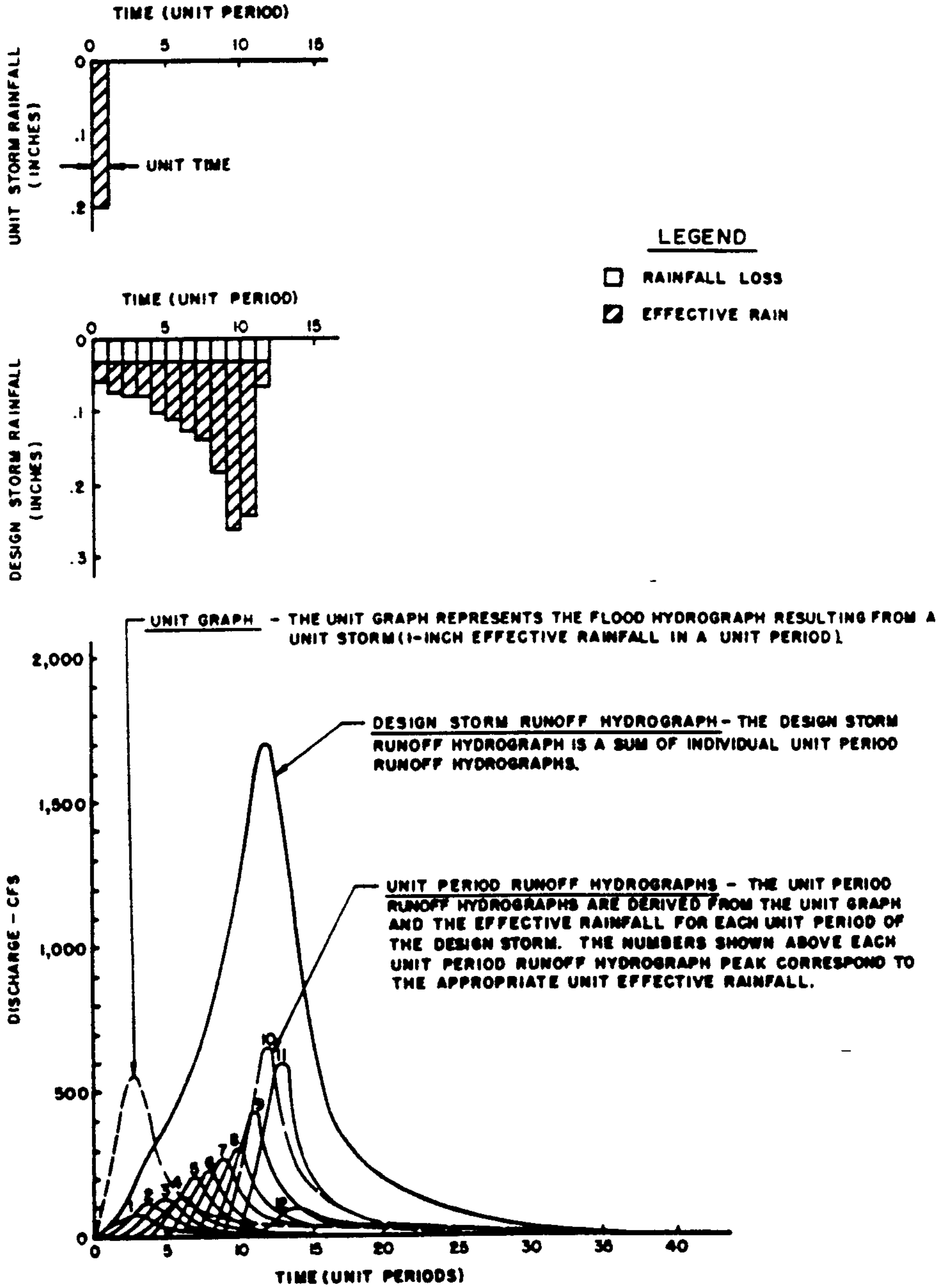


Figure 13. The convolution process.

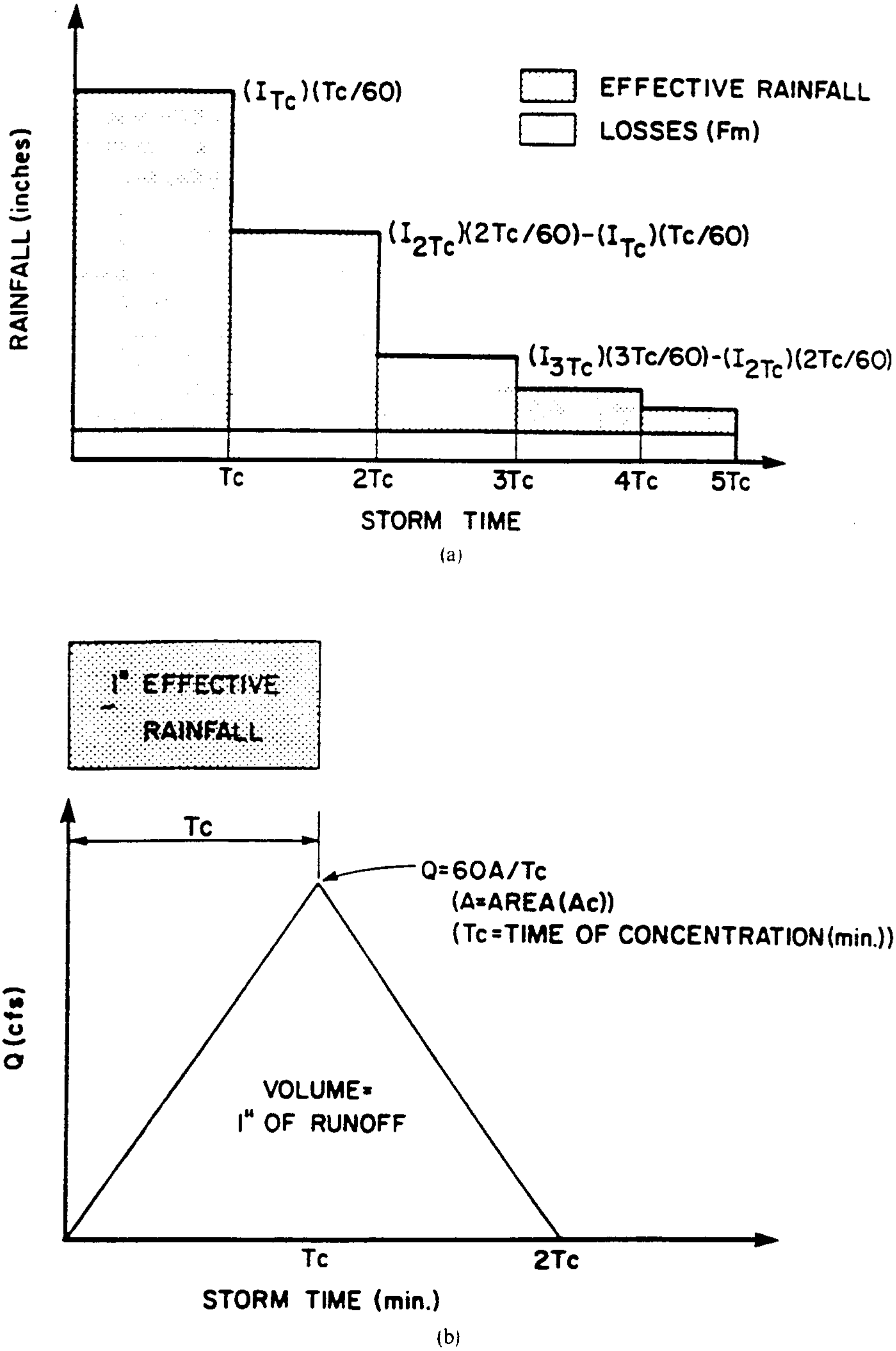


Figure 14. Rational-method unit-hydrograph application: (a) rational-method design storm; (b) rational-method unit hydrograph.

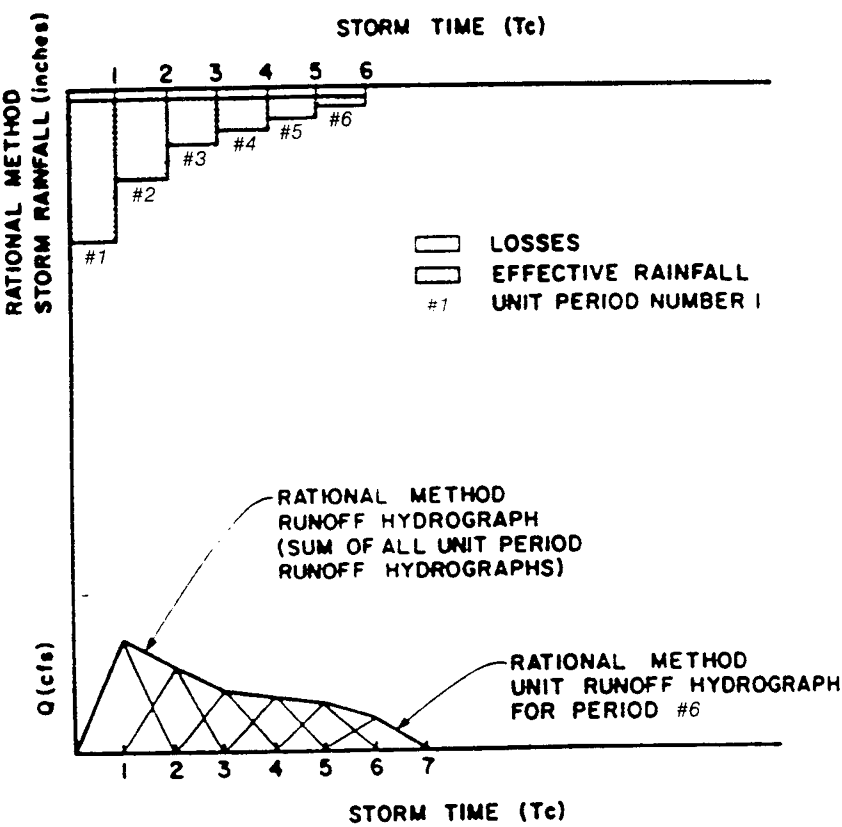


Figure 15. Rational-method runoff hydrograph.

effective rainfall of 1 inch, the associated unit-period runoff hydrograph must have a peak flowrate of (60/Tc) cfs per acre, where Tc is given in minutes. Similarly, a unit-period effective rainfall of only $\frac{1}{2}$ in. must have an associated unit-period runoff hydrograph with a base of 2Tc and a peak flowrate of ($\frac{1}{2}$)(60/Tc) cfs per acre. The runoff hydrographs associated with each unit effective rainfall are determined similarly and then arranged as shown in Figure 15 so that the resulting individual unit-period runoff hydrographs correspond in timing to the proper unit-period effective rainfalls. The runoff hydrograph is developed by adding the flow contributions from the several unit-period runoff hydrographs. Should the rational-method design storm pattern be rearranged into another sequence of unit effective rainfalls, the resulting runoff hydrograph would reflect the change in timing. Such a procedure is useful for small area studies.

Presentation of Product

An important element in the preparation of hydrology studies is the presentation of the results. Figure 16 illustrates a typical format for preparing watershed studies and includes the general organization of the study results, assumed data, and estimation procedures and calculations.

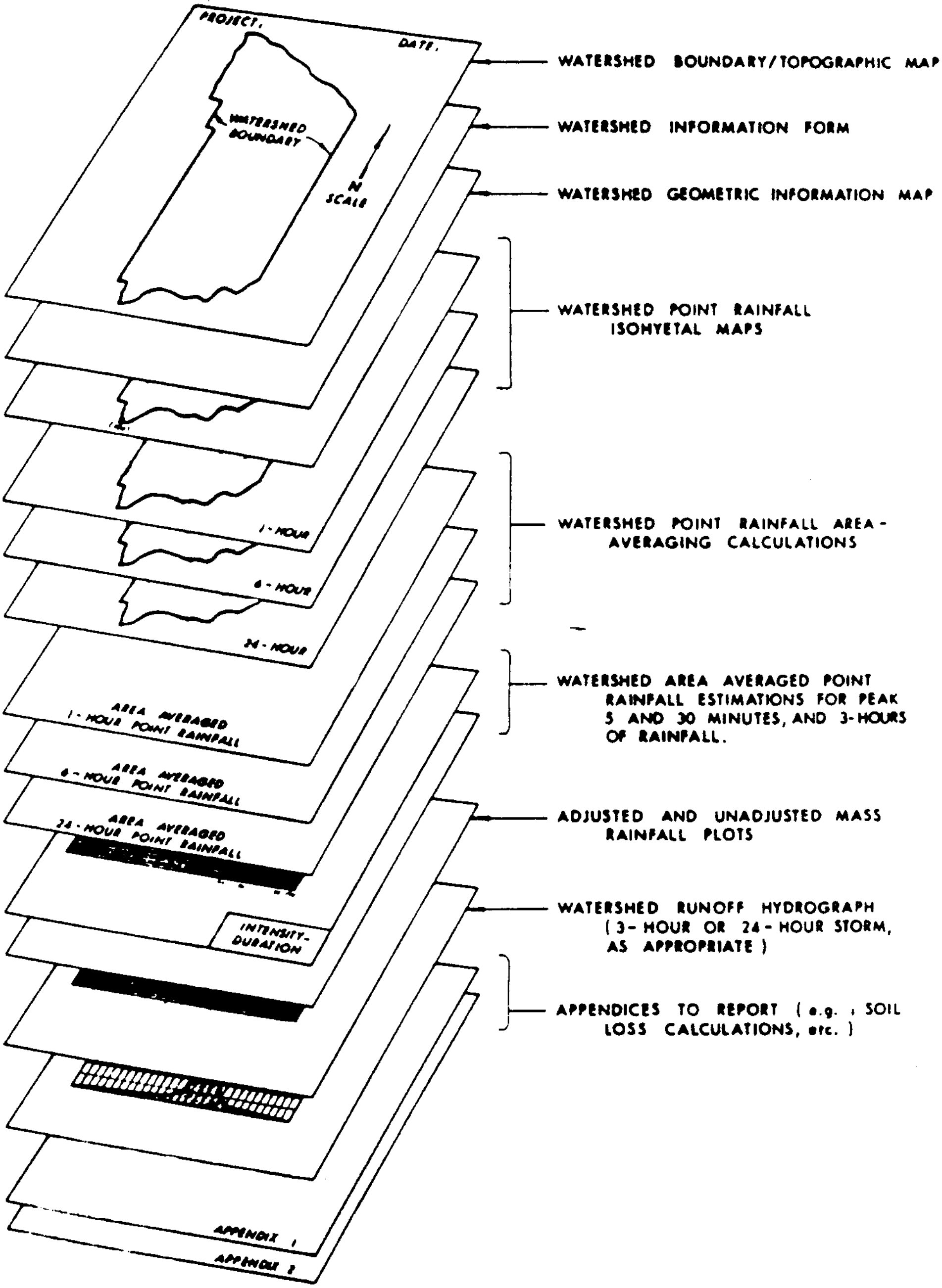


Figure 16. Runoff-hydrograph study submittal format.

FLOOD-FREQUENCY ESTIMATION

The uses of flood-flow-frequency data range from the specification of flood insurance risk relationships to the commonly occurring problem of designing flood-control facilities. Typically, however, stream-gage data are unavailable at the study site: consequently, some type of method is needed to synthesize a flood-frequency curve for ungaged streams.

Procedures used to develop flood-flow-frequency estimates at ungaged locations can be grouped in the following categories: (a) data-transfer methods, (b) statistical methods, (c) empirical equations, and (d) simulation models.

Because flood-flow-frequency information may be used for a variety of purposes, the hydrologist must be aware of the limitations associated with each of the groupings of methods. For example, flood-flow-frequency estimates used for design of flood-control facilities often are conservative in that the high design discharges are used for the corresponding return frequency. In this fashion, the designer compensates for the uncertainty in reliability of the design flowrate and provides a factor of safety. For flood-insurance studies, however, use of the computed flood-flow-frequency estimate may be desirable in order to avoid excessively high costs for the corresponding benefit. This procedure is described in [34].

Detailed discussions of the several categories of flood-flow-frequency analysis procedures are contained in [34]. In that publication, the four groupings of methods are further designated into eight categories as follows:

1. statistical estimation of peak flowrates
2. statistical estimation of moments
3. index flood-estimation methods
4. transfer methods
5. empirical equations
6. single-event methods
7. multiple-discrete-event methods
8. continuous simulation methods

Advantages and disadvantages of these methods are as follows:

- *Category 1:* Statistical estimation of peak flowrate (Q_p) methods use regression equations for determining a specific return frequency of flowrate by correlating stream-gage data to watershed characteristics. Ungaged stream flowrate estimates can then be obtained from the regression equations.
- *Category 2:* Statistical estimation of moments procedure extends the procedures of Category 1 by correlating the statistical moments of the frequency function developed from stream-gauge data to watershed characteristics.
- *Category 3:* Index flood estimation methods are analogous to the above two categories except that a selected index flood, such as the mean annual event, is used for the development of the necessary statistical relationships for events other than the index event.
- *Category 4:* Transfer methods usually refer to the relationships used to estimate flowrates immediately upstream or downstream of a stream gage location. However, the Corps of Engineers broadens this category to include procedures for the direct transfer of peak flood-flow-frequency values for frequency functions from similar gage locations to the subject study point.
- *Category 5:* Empirical equations are often used for the estimation of peak flowrates. The well-known rational method is an important example of this category.
- *Category 6:* Single-event methods are the most widely used approach for developing runoff hydrographs, which are subsequently used to develop a flood-flow-frequency curve. Incorporated in this category are the design storm methods, which attempt to relate runoff and rainfall frequency curves.
- *Category 7:* By considering a series of important record storm events with a single-event method, an approximate flood-frequency curve can be developed. The multiple-discrete-event category of models serves as a blend of the single-event category of models and the concept of continuous simulation.

- *Category 8:* Continuous-simulation (or continuous-record) models attempt to develop a continuous streamflow record based on a continuous rainfall record. Although in concept this category of models appears to be plausible, the success of these methods has not been clearly established due to the lack of evidence that this approach performs the much simpler and more often used unit hydrograph procedure of Category 6.

WATERSHED-MODELING UNCERTAINTY

Watershed runoff is a function of rainfall intensity, storm duration, infiltration capacity of the soil, cover of the soil, type of vegetation, area of the watershed and related shape factors, distribution of the storm with respect to space and time, watershed stream system topology, connectivity and branching, watershed geometry, stream-system hydraulics, overland flow characteristics, and several other factors. Because of the dozens of variables included in a completely deterministic model of watershed runoff and due to the uncertainty associated to the spatial and temporal values of each of the variable mathematical definitions, urban hydrologists often use statistical methods in predicting surface-runoff quantities.

With the widespread use of minicomputers and inexpensive microcomputers, the use of deterministic models is commonplace. These models attempt to simulate several of the most important hydrologic variables that strongly influence the watershed runoff quantities produced from severe design storm events. Generally speaking, the design storm and simulation models include approximations for runoff-hydrograph generation (coupled with models for estimating interception, evapotranspiration, interflow, and infiltration), channel routing, and detention-basin routing. The program user then combines these processes into a link-node schematic of the watershed. Because each of the hydrologic processes involve several parameters, the resulting output of the model (the runoff hydrograph) may be a function of several dozen parameters. In a procedure called *calibration*, many or all of the parameters are estimated by attempts to duplicate significant historical runoff hydrographs. However, Wood [43] notes that the watershed-model parameter interaction can result in considerable difficulty in optimizing the parameter set. In a similar deterministic modeling approach for soil systems and soil water movement, Guymon et al. [8] found that the normal range of uncertainty associated with laboratory measurement of groundwater flow hydraulic parameters alone can produce considerable variation in the model output. A detailed analysis of the sensitivity relationships for a watershed model is given by Mein and Brown [19].

The complexity of the uncertainty problem can be envisioned by formulating an abstract representation of the watershed. Figure 17 shows a link-node model of a simple watershed configuration. Also shown in the figure is a rain gauge and stream gauge (both located outside of the catchment). Consequently, a hydrologic model, when calibrated to the available rainfall-runoff data, is only a statistical correlation between the rain-gauge and stream-gauge data. It typically represents the correlation of the available data rather than the relationship between the actual rainfall distributions over the catchment and resulting runoff. The output of the model is the runoff hydrography, defined by

$$R(t, x_i) = R(t, x(m_i, s_i)) \tag{28}$$

where t = model time
x_i = model process function with statistical mean m_i and standard deviation s_i

The process variables used in Equation 28 include

- *Storm-pattern arrangement:* The design storm pattern can be thought of as a finite series of unit rainfalls (e.g., 5-, 10-, and 60-minute unit durations) where the rainfall intensity is assumed constant during each unit rainfall duration. The set of possible permutations of the set is (n!) where n is the number of unit periods, and each member of the set may have an assumed probability of occurrence. By examining a long history of local storm patterns, a general shape of the storm pattern may be statistically described by some type of shape factor (for example, the center of

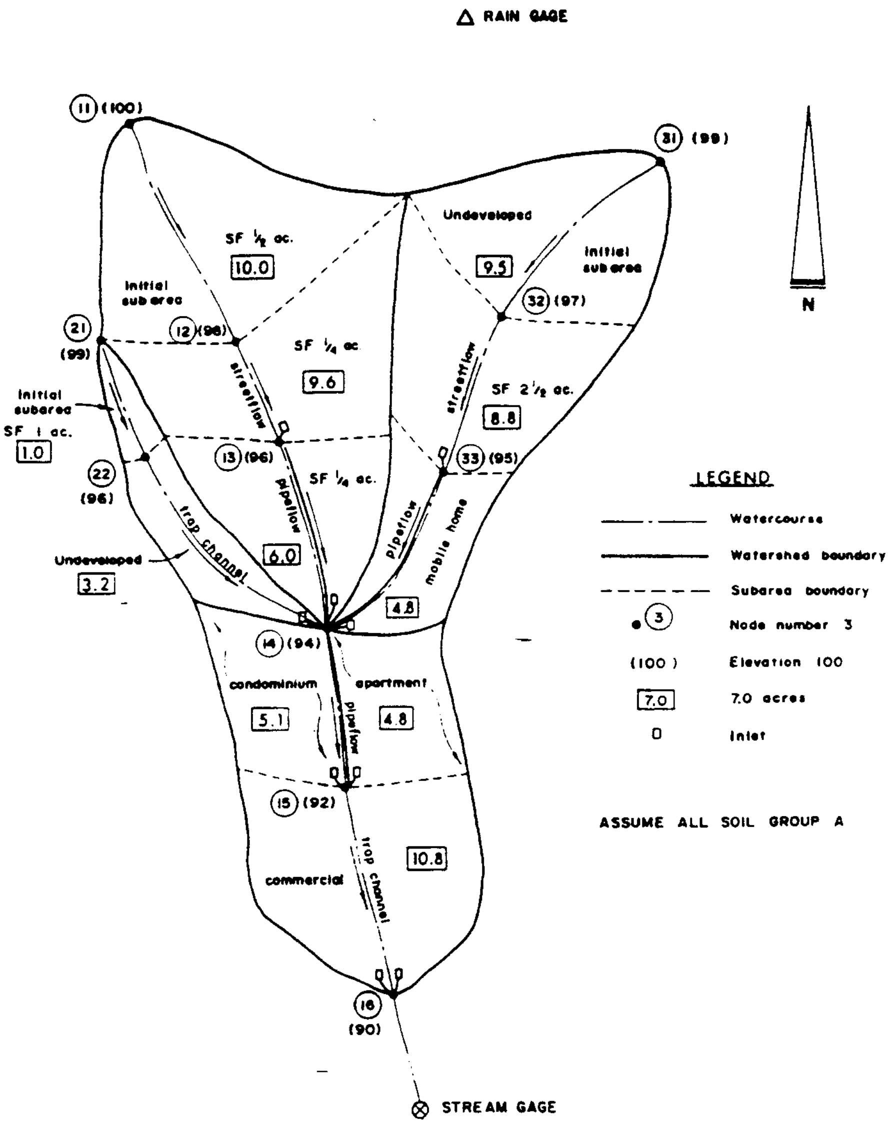


Figure 17. Example catchment link-node model schematic.

mass location as a percentage of total storm duration). Studying historical storm patterns with respect to a defined shape factor results in a statistical distribution with a corresponding mean and standard deviation.

- *Rainfall intensity:* The statistical representations of peak unit rainfall intensities result in several mathematical interpretations. Rain-gauge data are usually insufficient to extrapolate to a severe storm event (e.g., a 100-year return frequency) to within a precise confidence limit. Consequently, each unit rainfall has a statistical function associated with its average rainfall intensity and its probability occurrence with respect to the other rainfalls.

- *Point rainfall measurement:* In addition to the uncertainty in estimating severe storm rainfall magnitudes, there exists the uncertainty associated with the measuring and analysis of rain-gauge data. Rain-gauge networks are usually sparse, and many intensities occur at some location other than at a gauge. Additionally, measurement error is introduced due to winds, gauge malfunctions, and human errors. By studying the rain-gauge-network point rainfall estimates and comparing them to actual storm histories, a probabilistic function for this type of error may be estimated.
- *Point rainfall depth-area adjustment:* Factors to adjust point rainfall values to compensate for watershed size involve another probabilistic function, which relates the depth-area factors to the storm return frequency, storm duration, location of watershed, and other factors.
- *Storm movement:* In studies of large watersheds, movement of the storm may be a factor in producing peak-runoff flowrate quantities. Associated with this factor is the error in the assumed storm movement and its corresponding probability distribution. For small watersheds, this factor may usually be ignored.
- *Antecedent moisture condition:* The degree of soil saturation at the initiation of the storm event has a probabilistic distribution relating the moisture content of the soil to the design storm's return frequency. This function is further complicated by the differences in recovery rates of various soil types for various soil-cover conditions.
- *Infiltration capacity:* This variable alone leads to considerable uncertainty in watershed runoff determination. Errors arise due to the imprecise knowledge of soil cover, condition, landscape practices, compaction, and cultural practices, among other factors. Additional uncertainty occurs from laboratory measurement error, soil-type nonhomogeneity, plant root densities, position of the water table, and soil layering characteristics. Guymon et al. [8] noted that the doubts associated with deterministic models of soil water flow alone introduce considerable pessimism about deterministic models of more complex processes that involve soil water-flow models as a submodel. Additional uncertainty is generated by the variations in the infiltration capacity of a soil complex, the relationship of infiltration capacity to the rainfall intensity and antecedent moisture condition, and the proper definition of the capacity function itself.
- *Groundcover condition:* Associated with the condition of the vegetative cover is a probability distribution with respect to severe storm occurrence. For example, a burned watershed will produce much higher runoff than the unburned watershed in good condition.
- *Depression storage capacity:* Each watershed contains local depressions that temporarily store runoff and reduce the effective rainfall. This depression storage has a probabilistic function that relates the remaining depression storage capacity to the occurrence of the severe storm event.
- *Runoff hydrograph model uncertainty:* This error function attempts to relate several uncertainty factors, including overland flow, the unit-hydrograph shape, the linearity assumption used with the unit hydrograph, lag time (or time to peak) estimation, and related watershed parameters such as basin factors, and slope.
- *Channel-routing models:* An exact channel-routing deterministic model is currently unavailable. However, by accepting a level of error, reasonable approximate routing methods can be used. Because the methods are inexact, each has an associated probabilistic error function.
- *Baseflow and interflow:* The interaction between groundwater flow effects and the stream channels introduce several uncertainty variables, each with an unknown error function.
- *Basin-routing models:* Analogous to channel-routing models, detention-basin- and reservoir-routing models involve a mathematical interpretation of a physical process that has an associated error in precision. Also involved are errors in defining outflow versus depth, and basin storage versus depth relations. Considerable error can be associated with backwater effects in the basin outlet system due to unknown downstream hydraulics.
- *Submodel combination:* Besides the individual process-error functions, an error probability exists in the interaction between the multiple processes being modeled. For example, errors in channel routing may result in the magnitude of the peak flow being improperly defined and offset in time with respect to the peak of a combining channel flow, thus causing an error in the specification of the overall storm peak-flow magnitude.

- *Discretization errors:* The philosophy regarding the selection of nodal points and corresponding linkage processes varies among hydrologists, and consequently different link-node models of a watershed will be produced by different hydrologists. Because each of the submodels are imprecise, the total error function is also a function of the number of processes involved and the number and spacing of nodal points. This error includes the error in the hydrologist's interpretation of the watershed runoff characteristics.

Several other errors can be included in the above list, such as hydraulic-modeling errors (backwater effects, stream-flow measurement errors, stream-gage data collection and synthesis errors, etc.). Due to the great degree of uncertainty associated with watershed-simulation approaches, a truly precise deterministic model is unattainable. For severe design storm conditions, however, many of the above errors become negligible due to the overtaxing of the corresponding physical system. For example, the local design storm condition may include sufficient rainfall to completely fill the depression storage before the highest intensity storm rainfall begins. Additionally, due to the difficulty in defining the objective function (a severe design storm runoff hydrograph), the uncertainty involved in watershed modeling is usually acceptable in urban hydrology design studies as long as the results are conservative. Obviously, overestimation should be reduced whenever reasonable; however, the unknown objective function usually necessitates that reasonably conservative methods and parameters be utilized in order to obtain a higher probability of achieving the accepted level of public protection.

CHOICE OF MODELING CATEGORY

Due to the need for developing runoff hydrographs for design purposes, statistical methods such as contained in model Categories 1 through 5 are usually precluded in watershed hydrologic studies. Consequently, the categories of models available are essentially restricted to Categories 6, 7, and 8. Single-event models directly transform a design storm (hypothetical causative input) into a flood hydrograph. The multiple-discrete-even models transform an annual series of selected discrete rainfall events (usually one storm for each year) into an annual series of runoff hydrographs whose peak flowrates are used for subsequent statistical analysis. The continuous-record or continuous-simulation model results in a continuous record of synthetic runoff hydrographs for statistical synthesis. Each of the above three categories of deterministic models contain various versions and modifications that range widely in complexity, data requirements, and computational effort.

In general, the widely used unit hydrograph-design storm approach has continued widespread support among practitioners and governmental agencies involved in flood-control design. Such general-purpose models include the SCS model [39] and the hydrologic computer program package HEC-1 [35]. In a recent survey of hydrologic model usage by Federal and State governmental agencies and private engineering firms, it was found that "practically no use is made of watershed models for discrete event and continuous hydrograph simulation." In comparison, however, design-storm methods were used from 24 to 35 times more frequently than discrete-event or continuous-simulation models by Federal agencies and the private sector, respectively. The frequent use of design-storm methods appear to be due to several reasons: (a) design storm methods are considerably simpler to use than discrete-event and continuous-simulation models, (b) it has not been established in general that the more complex models provide an improvement in computational accuracy over design storm models, and (c) the level of complexity typically embodied in the continuous simulation class of models does not appear to be appropriate for the catchment rainfall-runoff data that are typically available. Consequently, the design storm approach continues to be the most often selected for flood control and drainage design studies.

A criterion for classifying a model as being simple or complex is given by Beard and Chang [2] as the "difficulty or reliability of model calibration." They state that "perhaps the simplest type of model that produces a flood hydrograph is the unit hydrograph model" which "can be derived to some extent from physical drainage features but fairly easily and fairly reliably calibrated through successive approximations by relating the time distribution of average basin rainfall excess to the time distribution of the runoff." In comparison, the "most complicated type of model is one that represents each significant element of the hydrologic process by a mathematical algorithm. This is represented by the Stanford Watershed model and requires extensive data and effort to calibrate."

The literature contains numerous reports of problems in using complex models, especially in parameter optimization. Additionally, it has not been clearly established whether complex models, such as in the continuous-simulation or discrete-event classes of models, provide an increase in accuracy over a standard design storm unit-hydrograph model. There are only a few papers and reports in the literature that provide a comparison of hydrologic model performance. From these references, it appears that a simple unit-hydrograph model provides as good as or better results than quasiphysically based (see [17]) or complex models.

Beard and Chang [2] state that in the case of the unit-hydrograph model, "the function of runoff versus rainfall excess is considered to be linear, whereas it usually is not in nature. Also, the variations in shapes of unit hydrographs are not derivable directly from physical factors. However, models of this general nature are usually as representative of physical conditions as can reasonably be validated by available data, and sophistication beyond validation capability." It is suggested that if data from 50 to 100 years of streamflow are available for a specified condition of watershed development, a frequency curve of flows should be constructed from a properly selected set of flows.

Schilling and Fuchs [29] write "the spatial resolution of rain data input is of paramount importance to the accuracy of the simulated hydrograph" due to "the high spatial variability of storms" and "amplification of rainfall sampling errors by the nonlinear transformation" of rainfall into runoff. Their recommendations are that a model should employ a simplified surface-flow model if there are many subbasins; a simple runoff coefficient loss rate; and a diffusion (zero inertia) or storage-channel routing technique. Hornberger et al. [10] state that "even the most physically based models . . . cannot reflect the true complexity and heterogeneity of the processes occurring in the field. Catchment hydrology is still very much an empirical science."

In attempting to define the modeling processes by the available field data forms, Hornberger et al. find that "hydrologic quantities measured in the field tend to be either integral variables (e.g., stream discharge, which reflects an integrated catchment response) or point estimates of variables that are likely to exhibit marked spatial and/or temporal variation (e.g., soil hydraulic conductivity)." Hence, the precise definition of the physics in a modeling sense becomes a problem that is "poorly posed in the mathematical sense." Typically, subbasin parameters cannot be estimated precisely due to the large associated estimation error. "Such difficulties often indicate that the structural complexity of the model is greater than is warranted on the basis of the calibration data set."

Schilling and Fuchs [29] note that errors in simulation occur for several reasons, including

1. The input data, consisting of rainfall and antecedent conditions, vary throughout the watershed and cannot be precisely measured.
2. The physical laws of fluid motion are simplified.
3. Model parameter estimates may be in error.

Schilling and Fuchs [29] also state that "it is inappropriate to use a sophisticated runoff model to achieve a desired level of modeling accuracy if the spatial resolution of rain input is low" (in their study, the rain-gage densities considered for an 1,800-acre catchment were 81, 9, and a single centered gage).

Garen and Burges [5] noted the difficulties in rainfall measurement for use in the Stanford Watershed Model, because the K1 parameter (rainfall adjustment factor) and UZSN parameter (upper-level storage) had the dominant impact on the model sensitivity. This is especially noteworthy because Dawdy and O'Donnell [3] concluded that insensitive model coefficients could not be calibrated accurately. Thus, they could not be used to measure physical effects of watershed changes.

In the extensive study by Loague and Freeze [17], three event-based rainfall-runoff models (a regression model, a unit-hydrograph model, and a kinematic-wave model) were used on three data sets of 269 storm events from three small upland catchments. In their paper, the term "quasi-physically based" (QPB) is used for the kinematic-wave model. The three catchments were 25 acres, 35 acres, and 2.8 square miles in size and were extensively monitored with rain gage, stream gage, neutron probe, and soil parameter site-testing instruments.

For example, for the 25-acre site, instrumentation consisted of 35 neutron probe access sites, 26 soil parameter sites (all equally spaced), an on-site rain gage, and a stream gage. The QPB model utilized 22 overland flow planes and four channel segments. In comparative tests between the three modeling approaches to measured rainfall-runoff data, it was concluded that all models performed poorly and that the QPB performance was only slightly improved by calibration of its most sensitive parameter, hydraulic conductivity. Loague and Freeze [17] write that the "conclusion one is forced to draw . . . is that the QPB model does not represent reality very well: in other words, there is considerable model error present. We suspect this is the case with most, if not all conceptual models currently in use." Additionally, "the fact that simpler, less data intensive models provided as good or better predictions than a QPB is food for thought."

Based on the literature, the main difficulty in the use, calibration, and development of complex models appears to be the lack of precise rainfall data and the high model sensitivity to (and magnification of) rainfall measurement errors. Nash and Sutcliffe [21] state that "as there is little point in applying exact laws to approximate boundary conditions, this, and the limited ranges of the variables encountered, suggest the use of simplified empirical relations."

In the absence of more encouraging results in the use of complex hydrology models, the widespread use and continued acceptance of design storm unit-hydrograph methods for the estimation of watershed runoff quantities is understandable. For a new rainfall-runoff modeling approach to achieve widespread acceptance, it must clearly demonstrate a superiority in performance. For example, Hall [9] writes that some predetermined criterion of goodness-of-fit is typically used to assess a new model's capability in reproducing historic storm-event runoff quantities. The new model is first calibrated to observed rainfall-runoff data and then verified using storm events excluded from the calibration storm event data set. This type of split-sample testing (e.g., see [17]) has become standard for evaluation of rainfall-runoff model performance.

A second set of criteria must be evaluated when using a new rainfall-runoff model for design storm flood estimation. Model parameters must be correlated to watershed characteristics, or regional values of the parameters must be established. More specifically, the model parameters used as the dependent variables must provide a relationship between the return frequency of runoff and the return frequency of the input rainfall. Acceptance of any new modeling technique typically depends upon the ease of use of the model by hydrologists. Hall [9] concluded that "until the additional steps required to develop a rainfall-runoff model into a flood estimation method are more widely appreciated, this apparent reluctance to accept innovation is liable to remain a feature of design practice."

The issue as to whether complex models provide better results than simpler models in developing watershed runoff quantities has motivated the proliferation of dozens of complex, conceptual, or so-called physically-based models. However, based upon the literature, the weight of evidence indicates that use of simpler models (such as the unit-hydrograph approach) will continue to be the most widely used modeling technique. Consequently, it is clear that so far, complex watershed models have not proven themselves to provide better results than the standard, simple unit-hydrograph models. In a study of stochastic hydrologic methods, Klemes and Bulu [16] write that often modelers "sidestep the real problem of modeling—the problem of how well a model is likely to reflect the future events—and divert the user to a more tractable, though less useful, problem of how to construct a model that will reproduce the past events. In so doing they expect, and perhaps rightly so, that by the time the prospective modeler has dug himself out of the heaps of technicalities, he either will have forgotten what the true purpose of modeling is or will have invested so much effort into the modeling game that he would prefer to avoid questions about its relevance." According to Gburek [6], "a model system is merely a researcher's idea of how a physical system interacts and behaves, and in the case of watershed research, watershed models are usually extremely simplified mathematical descriptions of a complex situation . . . . Until each internal submodel of the overall model can be independently verified, the model remains strictly a hypothesis with respect to its internal locations and transformation."

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