Chapter 8

Surface Runoff

8.1 DRAINAGE BASINS AND STORM HYDROGRAPHS

8.1.1 Drainage Basins and Runoff

As defined in Chapter 7, *drainage basins, catchments*, and *watersheds* are three synonymous terms that refer to the topographic area that collects and discharges surface streamflow through one outlet or mouth. The study of topographic maps from various physiographic regions reveals that there are several different types of drainage patterns (Figure 8.1.1). *Dendritic patterns* occur where rock and weathered mantle offer uniform resistance to erosion. Tributaries branch and erode headward in a random fashion, which results in slopes with no predominant direction or orientation. *Rectangular*

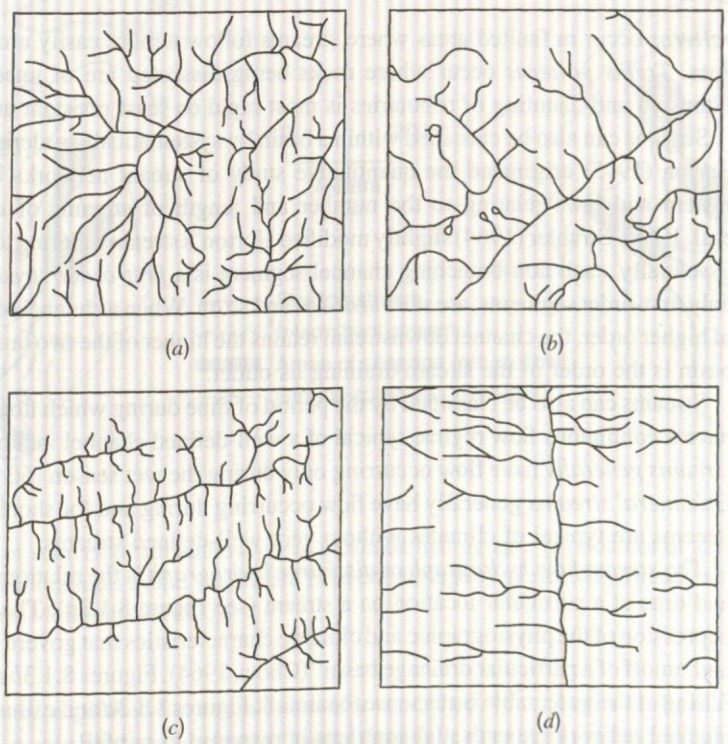


Figure 8.1.1 Common drainage patterns: (*a*) Dendritic; (*b*) Rectangular; (*c*) Trellis on folded terrain; (*d*) Trellis on mature, dissected coastal plain (from Hewlett and Nutter (1969)).

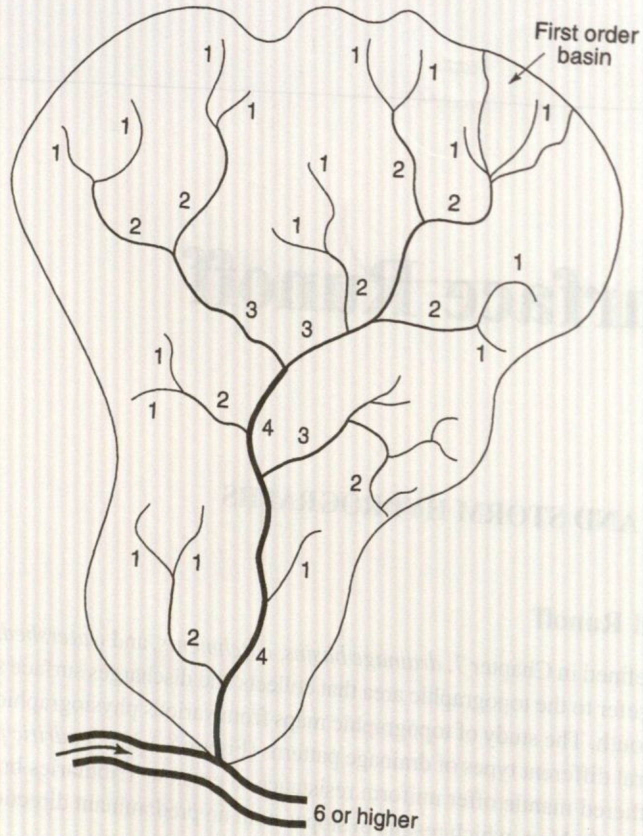


Figure 8.1.2 Stream orders (from Hewlett and Nutter (1969)).

patterns occur in faulted areas where streams follow a more easily eroded fractured rock in fault lines. *Trellis patterns* occur where rocks being dissected are of unequal resistance so that the extension and daunting of tributaries is most rapid on least resistant areas.

Streams can also be classified within a basin by systematically ordering the network of branches. Horton (1945) originated the quantitative study of stream networks by developing an ordering system and laws relating to the number and length of streams of different order (see Chow, et al. 1988). Strahler (1964) slightly modified Horton's stream ordering to that shown in Figure 8.1.2. Essentially, each non-branching channel segment is a *first-order stream*. Streams, which receive only first-order segments, are *second order*, and so on. When a channel of lower order joins a channel of higher order, the channel downstream retains the higher of the two orders. The order of a drainage basin is the order of the stream draining its outlet.

Streams can also be classified by the period of time during which flow occurs. *Perennial streams* have a continuous flow regime typical of a well-defined channel in a humid climate. *Intermittent streams generally* have flow occurring only during the wet season (50 percent of the time or less). *Ephemeral streams* generally have flow occurring during and for short periods after storms. These streams are typical of climates without very well-defined streams.

The *stream flow hydrograph* or *discharge hydrograph* is the relationship of flow rate (discharge) and time at a particular location on a stream (see Figure 8.1.3*a*). The *hydrograph* is "an integral expression of the physiographic and climatic characteristics that govern the relation between rainfall and runoff of a particular drainage basin" (Chow, 1964). Figures 8.1.3*b* and *c* illustrate the rising and falling of the water table in response to rainfall. Figures 8.1.3*d* and *e* illustrate that the flowing stream channel network expands and contracts in response to rainfall.

The spatial and temporal variations of rainfall and the concurrent variation of the abstraction processes define the runoff characteristics from a given storm. When the local abstractions have been

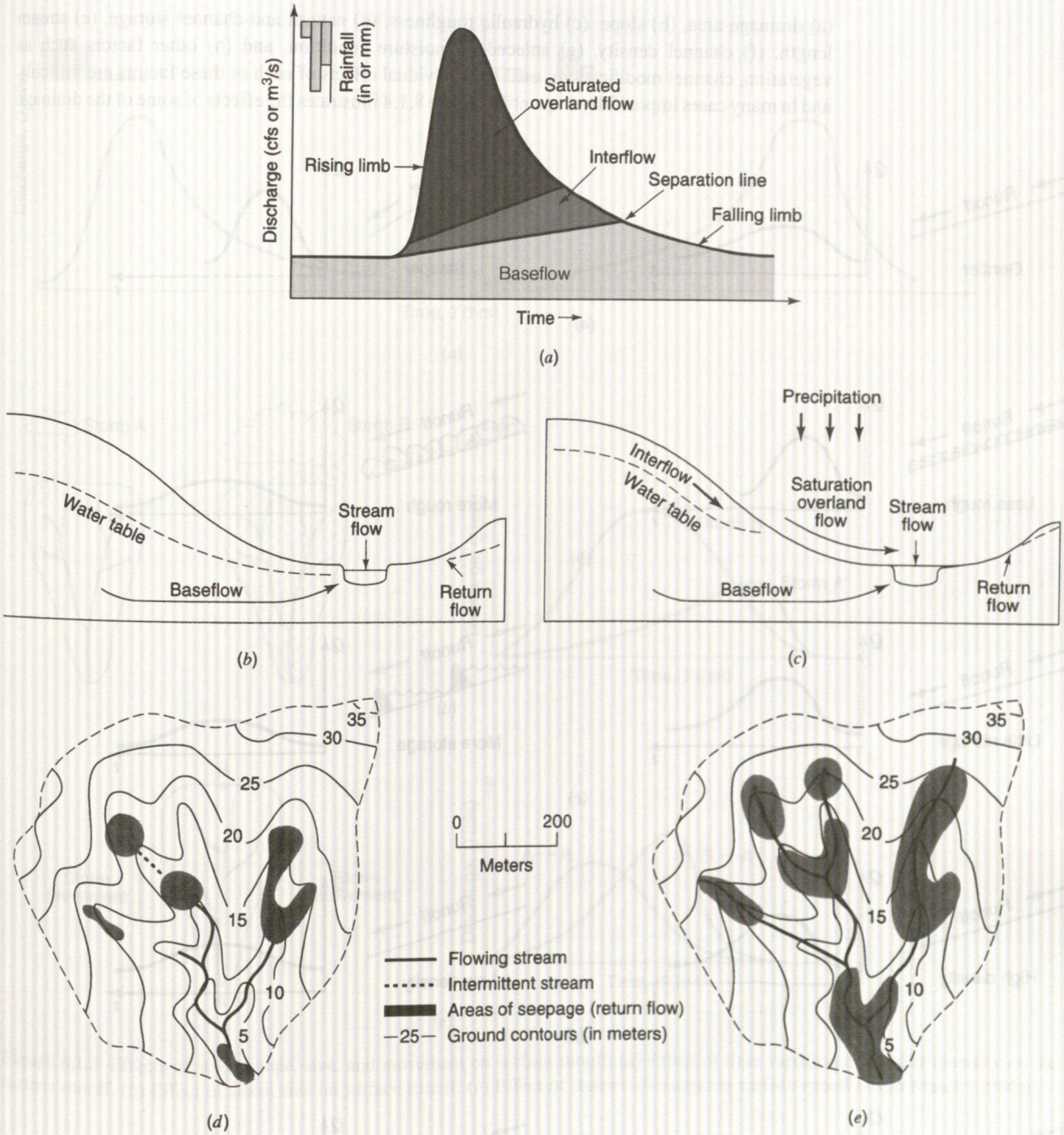


Figure 8.1.3 (*a*) Separatism of sources of streamflow on an idealized hydrograph; (*b*) Sources of streamflow on a hillslope profile during a dry period; (*c*) During a rainfall event; (*d*) Stream network during dry period; (*e*) Stream network extended during and after rainfall (from Mosley and McKerchar (1993)).

accomplished for a small area of a watershed, water begins to flow overland as *overland flow* and eventually into a drainage channel (in a gulley or stream valley). When this occurs, the hydraulics of the natural drainage channels have a large influence on the runoff characteristics from the watershed. Some of the factors that determine the hydraulic character of the natural drainage system include:

(a) drainage area, (b) slope, (c) hydraulic roughness, (d) natural and channel storage, (e) stream length, (f) channel density, (g) antecedent moisture condition, and (h) other factors such as vegetation, channel modifications, etc. The individual effects of each of these factors are difficult, and in many cases impossible, to quantify. Figure 8.1.4 illustrates the effects of some of the drainage

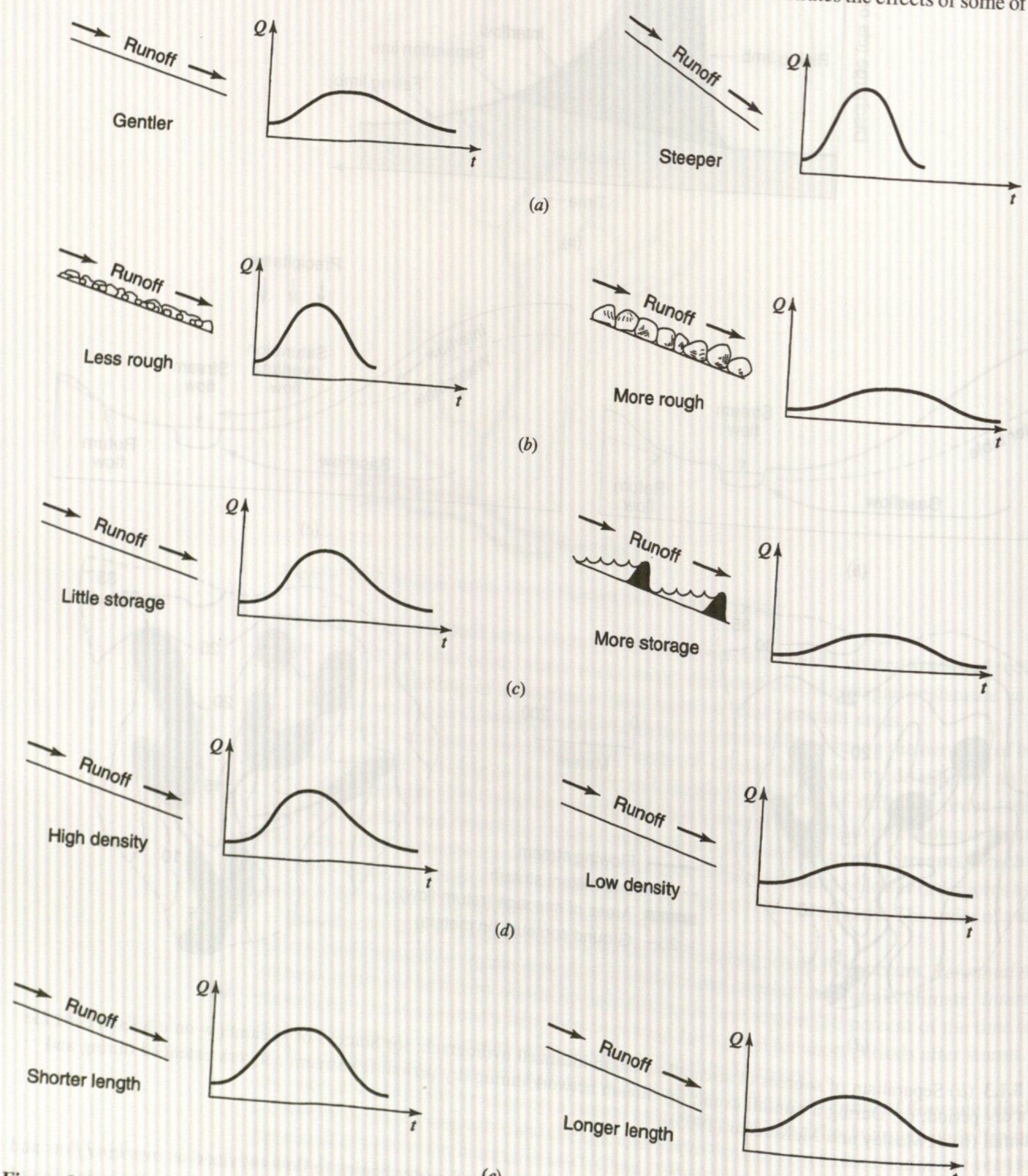


Figure 8.1.4 Effects of basin characteristics on the flood hydrograph. (*a*) Relationship of slope to peak discharge. (*b*) Relationship of hydraulic roughness to runoff. (*c*) Relationship of storage to runoff. (*d*) Relationship of drainage density to runoff. (*e*) Relationship of channel length to runoff (from Masch (1984)).

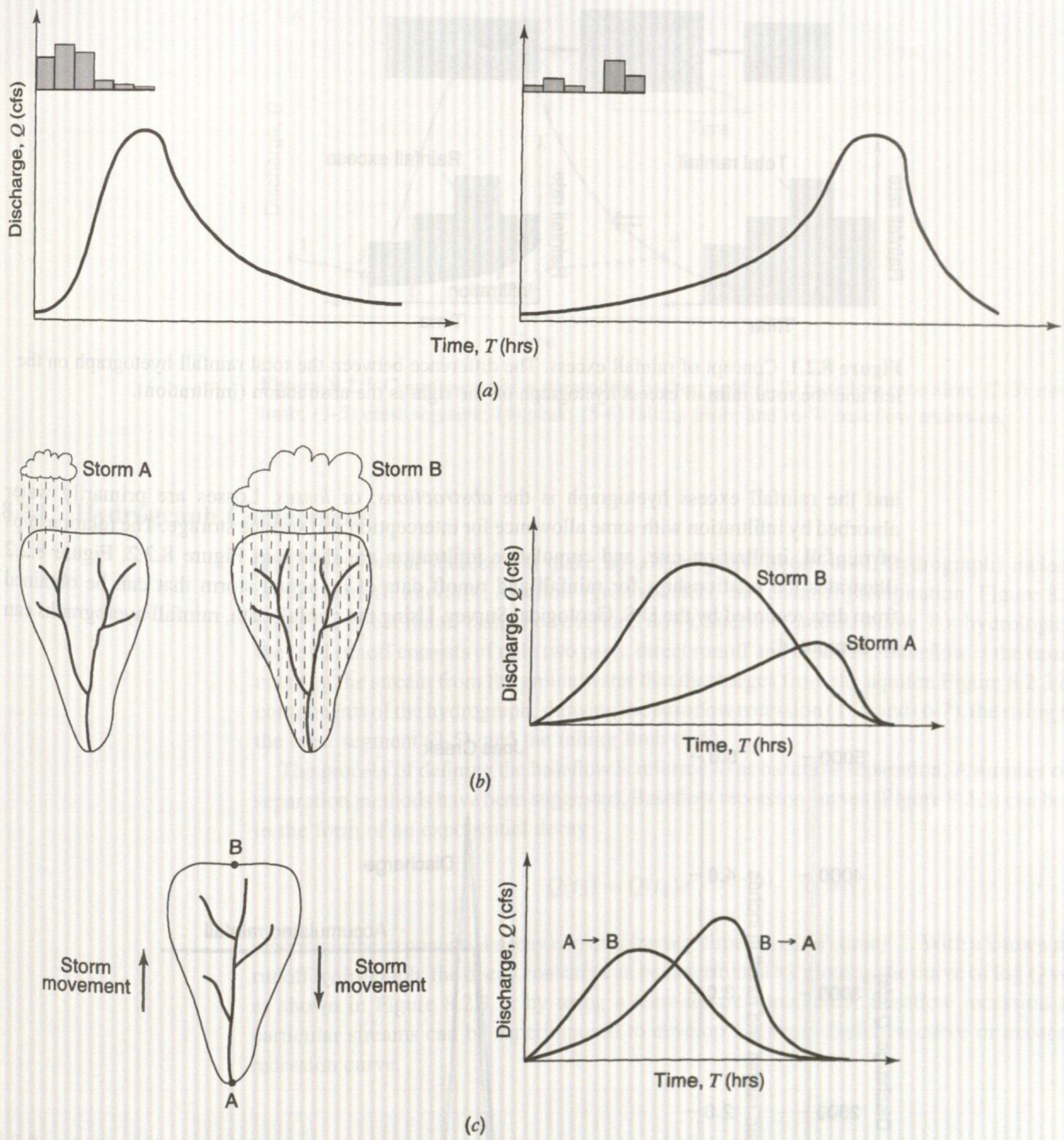


Figure 8.1.5 Effects of storm shape, size, and movement on surface runoff. (*a*) Effect of time variation of rainfall intensity on the surface runoff. (*b*) Effect of storm size on surface runoff. (*c*) Effect of storm movement on surface runoff (from Masch (1984)).

basin characteristics on the surface runoff (discharge hydrographs) and Figure 8.1.5 illustrates the effects of storm shape, size, and movement on surface runoff.

8.2 HYDROLOGIC LOSSES, RAINFALL EXCESS, AND HYDROGRAPH COMPONENTS

Rainfall excess, or *effective rainfall*, is that rainfall that is neither retained on the land surface nor infiltrated into the soil. After flowing across the watershed surface, rainfall excess becomes direct runoff at the watershed outlet. The graph of rainfall excess versus time is the rainfall excess hyetograph. As shown in Figure 8.2.1, the difference between the observed total rainfall hyetograph

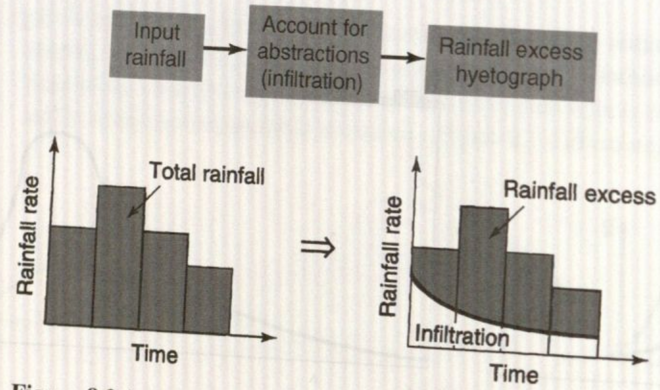


Figure 8.2.1 Concept of rainfall excess. The difference between the total rainfall hyetograph on the left and the total rainfall excess hyetograph on the right is the abstraction (infiltration).

and the rainfall excess hyetograph is the *abstractions*, or *losses*. Losses are primarily water absorbed by infiltration with some allowance for interception and surface storage. The relationships of rainfall, infiltration rate, and cumulative infiltration are shown in Figure 8.2.2. Figure 8.2.2 illustrates the relationships for rainfall and runoff data of an actual storm that can be obtained from data recorded by the U.S. Geological Survey. Using the rainfall data, rainfall hyetographs can be computed.

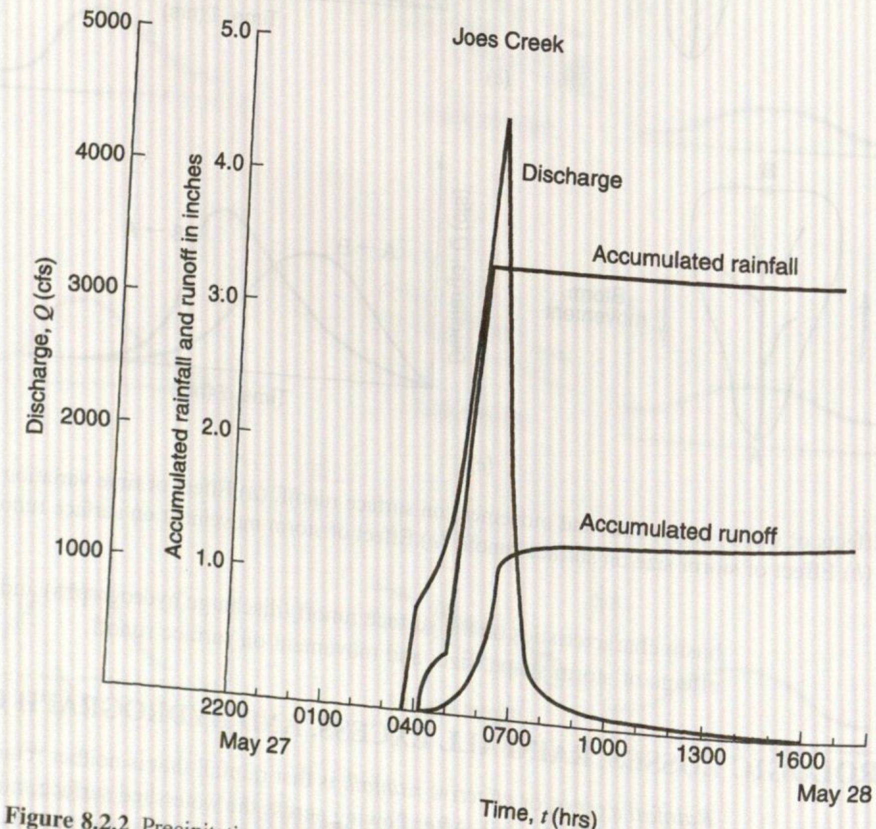


Figure 8.2.2 Precipitation and runoff data for Joes Creek, storm of May 27–28, 1978 (from Masch (1984)).

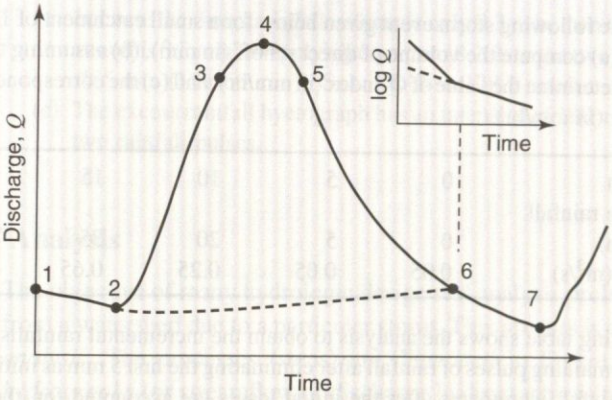


Figure 8.2.3 Components of a streamflow hydrograph: (1-2) baseflow recession; (2-3) rising limb; (3-5) crest segment; (4) peak; (5-6) falling limb; and (6-7) baseflow recession.

8.2.1 Hydrograph Components

There are several sources that make up a hydrograph (total runoff hydrograph) including direct surface runoff, interflow, baseflow (groundwater), and channel precipitation. Figure 8.1.3*a* illustrates the direct runoff (saturated overland flow), interflow, and baseflow. For hydrologic purposes, the total runoff consists of only two parts, direct runoff and baseflow. Baseflow is the result of water entering the stream from the groundwater that discharges from the aquifer. Figure 8.2.3 defines the components of the hydrograph, showing the baseflow recession (1-2) and (6-7), the rising limb (2-3), the crest segment (3-5), and the falling limb (5-6).

The process of defining the baseflow is referred to as baseflow separation. A number of baseflow separation methods have been suggested. Baseflow recession curves (Figure 8.2.3) can be described in the form of an exponential decay

$$Q(t_2) = Q(t_2)e^{-k(t_2 - t_1)}, \quad t_2 > t_1 \tag{8.2.1}$$

where k is the exponential decay constant having dimensions of $(\text{time})^{-1}$. With a known streamflow runoff hydrograph, the decay constant can be determined by plotting the curve of log Q versus time as shown in Figure 8.2.3 or by using a least-squares procedure. Baseflow recession curves for particular streams can be superimposed to develop a normal depletion curve or master baseflow recession curve.

8.2.2 Φ-Index Method

The Φ-index is a constant rate of abstractions (in/hr or cm/hr) that can be used to approximate infiltration. Using an observed rainfall pattern and the resulting known volume of direct runoff, the Φ-index can be determined. Using the known rainfall pattern, Φ is determined by choosing a time interval Δt, identifying the number of rainfall intervals N of rainfall that contribute to the direct runoff volume, and then subtracting $\Phi \cdot \Delta t$ from the observed rainfall in each time interval. The values of Φ and N will need to be adjusted so that the volume of direct runoff (r_d) and excess rainfall are equal

$$r_d = \sum_{n=1}^{N} (R_n - \Phi \cdot \Delta t) \tag{8.2.2}$$

where R_n is the observed rainfall (in or cm) in time interval n.

EXAMPLE 8.2.1

Consider the following storm event given below for a small catchment of 120 hectares. For a baseflow of 0.05 m^3/s, (a) compute the volume of direct runoff (in mm), (b) assuming the initial losses (abstractions) are 5 mm, determine the value of Φ-index (in mm/hr), and (c) the corresponding effective rainfall intensity hyetograph (in mm/hr).

Time (min)	0	5	10	15	20	25	30
Cumulative rainfall Depth (mm)	0	5	20	35	45		
Discharge (m^3/s)	0.05	0.05	0.25	0.65	0.35	0.15	0.05

SOLUTION

The following table shows the analysis to obtain the incremental rainfalls and rainfall intensities. There are three remaining pulses of rainfall after eliminating the first 5 mm as initial abstractions. For each of the first two rainfall increments after the initial losses are accounted for, the incremental rainfall volume is $\Phi \cdot \Delta t = \Phi(5\text{ min})(1\text{ hr}/60\text{ min}) = 15$ mm, so solving $\Phi = 180$ mm/hr. For the third interval, $\Phi \cdot \Delta t = \Phi(5\text{ min})(1\text{ hr}/60\text{ min}) = 10$, so the rainfall intensity is 120 mm/hr.

Time (min)	Discharge (m^3/sec)	Direct runoff (m^3/sec)	Cumulative rainfall (mm)	Incremental rainfall (mm)	Rainfall intensity (mm/hr)
0	0.05	0	0		
5	0.05	0	5 (initial loss)	0	
10	0.25	0.20	20	15	180
15	0.65	0.60	35	15	180
20	0.35	0.30	45	10	120
25	0.15	0.10			
30	0.05	0			

(a) The direct runoff volume $= (0.2 + 0.6 + 0.3 + 0.1)\ m^2/\text{sec}\ (5\text{ min.})\ (60\text{ sec/min}) = 360\ m^3$, which converts to $r_d = 360\ m^3/(120\text{ hectares} \times 10{,}000\ m^2/\text{hectare}) = 0.3$ mm.

(b) There are three pulses of rainfall after eliminating the first 5 mm as initial abstractions. Considering the two largest rainfall pulses, the rainfall volume above the 120 mm/hr level is 10 mm, and

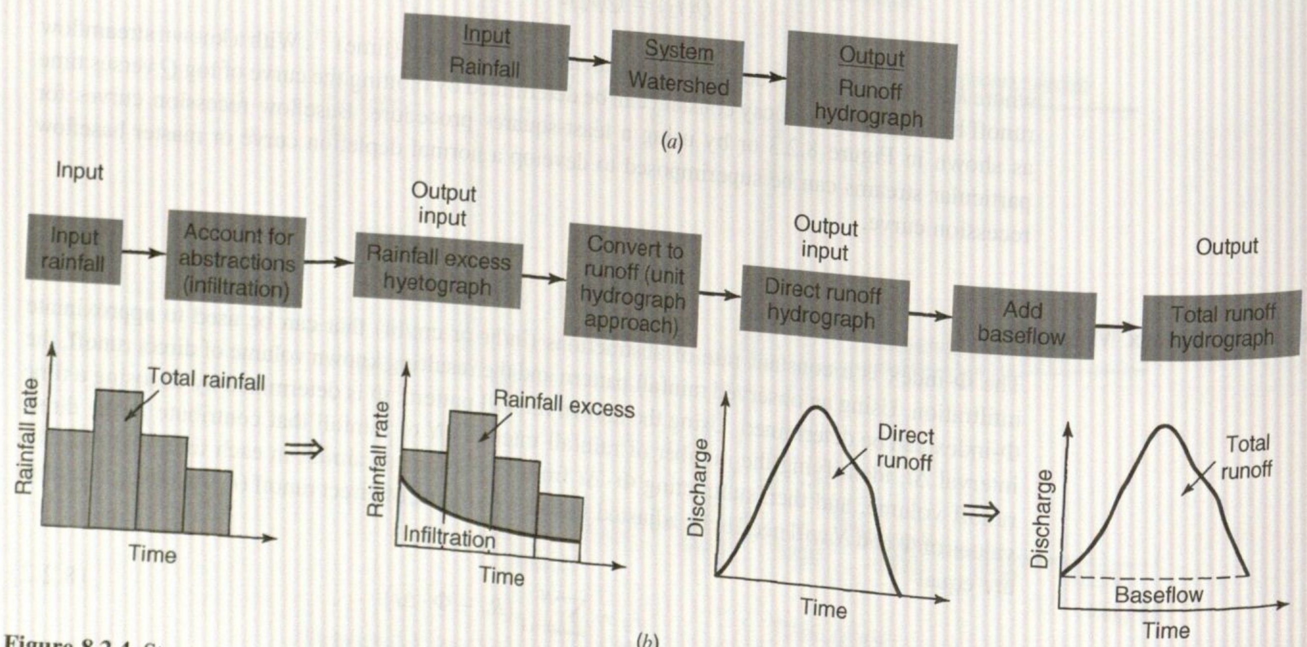


Figure 8.2.4 Storm runoff hydrographs. (*a*) Rainfall-runoff modeling; (*b*) Steps to define storm runoff.

because the direct rainfall volume is only 0.3 mm, then Φ is above the 120 mm/hr level. The direct runoff is 0.3 mm, so applying equation (8.2.2) to the two largest pulses, $r_d = 0.3$ mm $= 2[(15$ mm $- \Phi(5$ min$)(1$ hr$/60$ min$)]$. Solving, $\Phi = 178.2$ mm/hr.

(c) The excess rainfall hyetograph has an intensity of 180 mm/hr − 178.2 mm/hr = 1.8 mm/hr for the two rainfall pulses.

8.2.3 Rainfall-Runoff Analysis

The objective of many hydrologic design and analysis problems is to determine the surface runoff from a watershed due to a particular storm. This process is commonly referred to as *rainfall-runoff analysis*. The processes (steps) are illustrated in Figure 8.2.4 to determine the *storm runoff hydrographs* (or streamflow or discharge hydrograph) using the unit hydrograph approach.

8.3 RAINFALL-RUNOFF ANALYSIS USING UNIT HYDROGRAPH APPROACH

The objective of *rainfall-runoff analysis* is to develop the runoff hydrograph as illustrated in Figure 8.2.4*a*, where the system is a watershed or river catchment, the input is the rainfall hyetograph, and the output is the runoff or discharge hydrograph. Figure 8.2.4*b* defines the processes (steps) to determine the runoff hydrograph from the rainfall input using the *unit hydrograph approach*.

A *unit hydrograph* is the direct runoff hydrograph resulting from 1 in (or 1 cm in SI units) of excess rainfall generated uniformly over a drainage area at a constant rate for an effective duration. The unit hydrograph is a simple linear model that can be used to derive the hydrograph resulting from any amount of excess rainfall. The following basic assumptions are inherent in the unit hydrograph approach:

1. The excess rainfall has a constant intensity within the effective duration.
2. The excess rainfall is uniformly distributed throughout the entire drainage area.
3. The base time of the direct runoff hydrograph (i.e., the duration of direct runoff) resulting from an excess rainfall of given duration is constant.
4. The ordinates of all direct runoff hydrographs of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.
5. For a given watershed, the hydrograph resulting from a given excess rainfall reflects the unchanging characteristics of the watershed.

The following *discrete convolution equation* is used to compute direct runoff hydrograph ordinates Q_n, given the rainfall excess values P_m and given the unit hydrograph ordinates U_{n-m+1} (Chow et al., 1988):

$$Q_n = \sum_{m=1}^{n \le M} P_m U_{n-m+1} \quad \text{for } n = 1, 2, \ldots, N \tag{8.3.1}$$

where n represents the direct runoff hydrograph time interval and m represents the precipitation time interval ($m = 1, \ldots, n$).

The reverse process, called *deconvolution*, is used to derive a unit hydrograph given data on P_m and Q_n. Suppose that there are M pulses of excess rainfall and N pulses of direct runoff in the storm considered; then N equations can be written for Q_n, $n = 1, 2 \ldots, N$, in terms of $N - M + 1$ unknown values of the unit hydrograph, as shown in Table 8.3.1. Figure 8.3.1 diagramatically illustrates the calculation and the runoff contribution by each rainfall input pulse.

Once the unit hydrograph has been determined, it may be applied to find the direct runoff and streamflow hydrographs for given storm inputs. When a rainfall hyetograph is selected, the abstractions are subtracted to define the excess rainfall hyetograph. The time interval used in

Table 8.3.1 The Set of Equations for Discrete Time Convolution

Q_1	$= P_1U_1$
Q_2	$= P_2U_1 + P_1U_2$
Q_3	$= P_3U_1 + P_2U_2 + P_1U_3$
$\cdots$	
Q_M	$= P_MU_1 + P_{M-1}U_2 + \cdots + P_1U_M$
Q_{M+1}	$= \quad 0 + P_MU_2 \quad + \cdots + P_2U_M + P_1U_{M+1}$
$\cdots$	
Q_{N-1}	$= \quad 0 + \quad 0 \quad + \cdots + \quad 0 \quad + \quad 0 \quad + \cdots + P_MU_{N-M} + P_{M-1}U_{N-M+1}$
Q_N	$= \quad 0 + \quad 0 \quad + \cdots + \quad 0 \quad + \quad 0 \quad + \cdots + \quad 0 \qquad + P_MU_{N-M+1}$

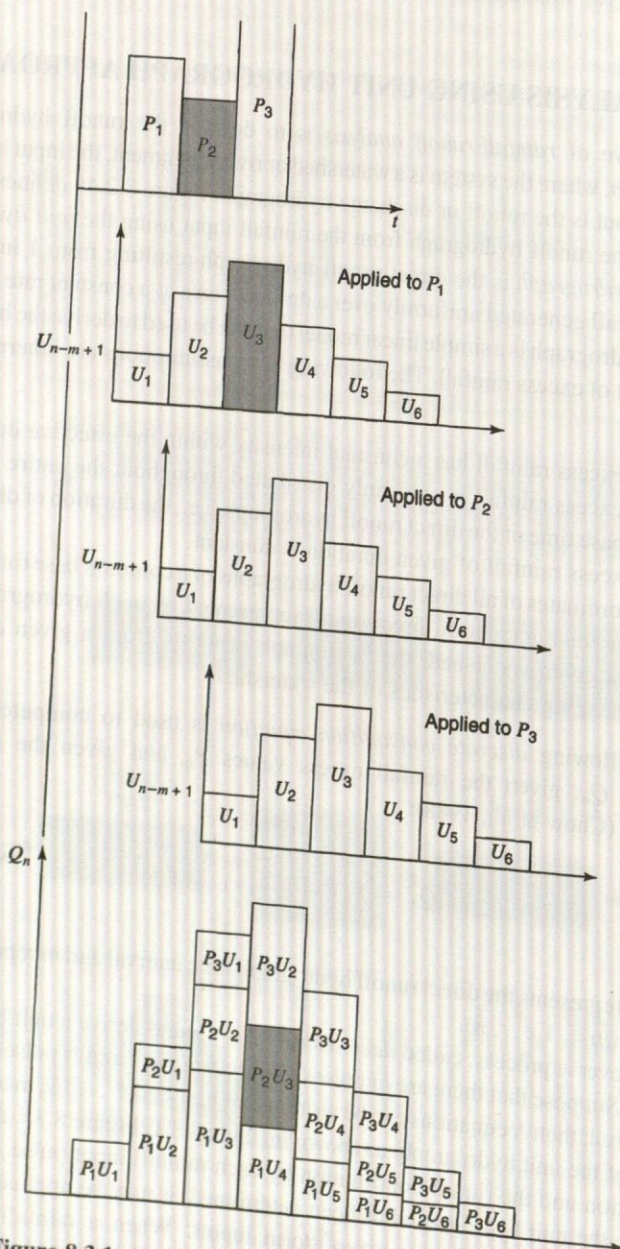


Figure 8.3.1 Application of unit hydrograph to rainfall input.

defining the excess rainfall hyetograph ordinates must be the same as that for which the unit hydrograph is specified.

EXAMPLE 8.3.1

The 1-hr unit hydrograph for a watershed is given below. Determine the runoff from this watershed for the storm pattern given. The abstractions have a constant rate of 0.3 in/h.

Time (h)	1	2	3	4	5	6
Precipitation (in)	0.5	1.0	1.5	0.5		
Unit hydrograph (cfs)	10	100	200	150	100	50

SOLUTION

The calculations are shown in Table 8.3.2. The 1-hr unit hydrograph ordinates are listed in column 2 of the table; there are $L = 6$ unit hydrograph ordinates, where $L = N - M + 1$. The number of excess rainfall intervals is $M = 4$. The excess precipitation 1-hr pulses are $P_1 = 0.2$ in, $P_2 = 0.7$ in, $P_3 = 1.2$ in, and $P_4 = 0.2$ in, as shown at the top of the table. For the first time interval $n = 1$, the discharge is computed using equation (8.3.1):

$$Q_1 = P_1 U_1 = 0.2 \times 10 = 2 \text{ cfs}$$

For the second time interval, $n = 2$,

$$Q_2 = P_1 U_2 + P_2 U_1 = 0.2 \times 100 + 0.7 \times 10 = 27 \text{ cfs}$$

and similarly for the remaining direct runoff hydrograph ordinates. The number of direct runoff ordinates is $N = L + M - 1 = 6 + 4 - 1 = 9$; i.e., there are nine nonzero ordinates, as shown in Table 8.3.2. Column 3 of Table 8.3.2 contains the direct runoff corresponding to the first rainfall pulse, $P_1 = 0.2$ in, and column 4 contains the direct runoff from the second rainfall pulse, $P_2 = 0.2$ in, etc. The direct runoff hydrograph, shown in column 7 of the table, is obtained, from the principle of superposition, by adding the values in columns 3–6.

Table 8.3.2 Calculation of the Direct Runoff Hydrograph

(1)	(2)	(3)	(4)	(5)	(6)	(7)
		Total precipitation (in)				
		0.5	1	1.5	0.5	
		Excess precipitation (in)				
Time (hr)	Unit hydrograph (cfs/in)	0.2	0.7	1.2	0.2	Direct runoff (cfs)
0	0	0	0			0
1	10	2	0	0		2
2	100	20	7	0	0	27
3	200	40	70	12	0	122
4	150	30	140	120	2	292
5	100	20	105	240	20	385
6	50	10	70	180	40	300
7	0	0	35	120	30	185
8			0	60	20	80
9				0	10	10
10					0	0

EXAMPLE 8.3.2

Determine the 1-hr unit hydrograph for a watershed using the precipitation pattern and runoff hydrograph below. The abstractions have a constant rate of 0.3 in/hr, and the baseflow of the stream is 0 cfs.

Time (h)	1	2	3	4	5	6	7	8	9	10
Precipitation (in)	0.5	1.0	1.5	0.5						
Runoff (cfs)	2	27	122	292	385	300	185	80	10	0

SOLUTION

Using the deconvolution process, we get $Q_1 = P_1U_1$

so that for $P_1 = 0.5 - 0.3 = 0.2$ in and $Q_1 = 2$ cfs,

$U_1 = Q_1/P_1 = 2/0.2 = 10\,\text{cfs}$.

$Q_2 = P_1U_2 + P_2U_1$, so that

$U_2 = (Q_2 - P_2U_1)/P_1$

where

$P_2 = 1.0 - 0.3 = 0.7$ in and $Q_2 = 27$ cfs.

$U_2 = (27 - 0.7(10))/0.2 = 100$ cfs and

$Q_3 = P_1U_3 + P_2U_2 + P_3U_1$

then

$U_3 = (Q_3 - P_2U_2 - P_3U_1)/P_1$, so that

$U_3 = (122 - 0.7(100) - 1.2(10))/0.2 = 200$ cfs.

The rest of the unit hydrograph ordinates can be calculated in a similar manner.

8.4 SYNTHETIC UNIT HYDROGRAPHS

8.4.1 Snyder's Synthetic Unit Hydrograph

When observed rainfall-runoff data are not available for unit hydrograph determination, a *synthetic unit hydrograph* can be developed. A unit hydrograph developed from rainfall and streamflow data in a watershed applies only to that watershed and to the point on the storm where the streamflow data were measured. Synthetic unit hydrograph procedures are used to develop unit hydrographs for other locations on the stream in the same watershed or other watersheds that are of similar character.

One of the most commonly used synthetic unit hydrograph procedures is Snyder's synthetic unit hydrograph. This method relates the time from the centroid of the rainfall to the peak of the unit hydrograph to geometrical characteristics of the watershed. To determine the regional parameters C_t and C_p, one can use values of these parameters determined from similar watersheds. C_t can be determined from the relationship for the *basin lag*:

$$t_p = C_1C_t(L \cdot L_c)^{0.3} \tag{8.4.1}$$

where C_1, L, and L_c are defined in Table 8.4.1. Solving equation (8.4.1) for C_t gives

$$C_t = \frac{t_p}{C_1(L \cdot L_c)^{0.3}} \tag{8.4.2}$$

To compute C_t for a gauged basin, L and L_c are determined for the gauged watershed and t_p from the derived unit hydrograph for the gauged basin.

To compute the other required parameter C_p, the expression for peak discharge of the standard unit hydrograph can be used:

$$Q_p = \frac{C_2 C_p A}{t_p} \tag{8.4.3}$$

or for a unit discharge (discharge per unit area)

$$q_p = \frac{C_2 C_p}{t_p} \tag{8.4.4}$$

Solving equation (8.4.4) for C_p gives

$$C_p = \frac{q_p t_p}{C_2} \tag{8.4.5}$$

This relationship can be used to solve for C_p for the ungauged watershed, knowing the terms in the right-hand side. Table 8.4.1 defines the steps for this procedure.

Section 8.8 discusses the SCS-unit hydrograph procedure.

EXAMPLE 8.4.1

A watershed has a drainage area of 5.42 mi^2; the length of the main stream is 4.45 mi, and the main channel length from the watershed outlet to the point opposite the center of gravity of the watershed is 2.0 mi. Using $C_t = 2.0$ and $C_p = 0.625$, determine the standard synthetic unit hydrograph for this basin. What is the standard duration? Use Snyder's method to determine the 30-min unit hydrograph parameter.

SOLUTION

For the standard unit hydrograph, equation (8.4.1) gives

$t_p = C_1 C_t (LL_c)^{0.3} = 1 \times 2 \times (4.45 \times 2)^{0.3} = 3.85\ \text{hr}$

The standard rainfall duration $t_r = 3.85/5.5 = 0.7$ hr. For a 30-min unit hydrograph, $t_R = 30\ \text{min} = 0.5$ hr. The basin lag $t_{pR} = t_p - (t_r - t_R)/4 = 3.85 - (0.7 - 0.5)/4 = 3.80$ hr. The peak flow for the required unit hydrograph is $= q_p t_p / t_{pR}$, and substituting equation (8.4.4) in the previous equation, $q_{pR} = q_p t_p / t_{pR} = (C_2 C_p / t_p) t_p / t_{pR} = C_2 C_p / t_{pR}$, so that $q_{pR} = 640 \times 0.625/3.80 = 105.26\ \text{cfs}\ (\text{in} \cdot \text{mi}^2)$, and the peak discharge is $Q_{pR} = q_{pR} A = 105.26 \times 5.42 = 570\ \text{cfs/in}$.

The widths of the unit hydrograph are computed next. At 75 percent of the peak discharge, $W_{75} = C_{W_{75}} q_{pR}^{-1.08} = 440 \times 105.26^{-1.08} = 2.88$ hr. At 50 percent of the peak discharge, $W_{50} = C_{W_{50}} q_{pR}^{-1.08} = 770 \times 105.26^{-1.08} = 5.04$ hr.

The base time t_b may be computed assuming a triangular shape. This, however, does not guarantee that the volume under the unit hydrograph corresponds to 1 in (or 1 cm, for SI units) of excess rainfall. To overcome this, the value of t_b may be exactly computed taking into account the values of W_{50} and W_{75} by solving the equation in step 5 of Table 8.4.1 for t_b:

$t_b = 2581\, A/Q_{pR} - 1.5\, W_{50} - W_{75}$

so that, with $A = 5.42\ \text{mi}^2$, $W_{50} = 5.04$ hr, $W_{75} = 2.88$ hr, and $Q_{pR} = 570$ cfs/in, $T_b = 2581(5.42)/570 - 1.5 \times 5.04 - 2.88 = 14.1$ hr.

8.4.2 Clark Unit Hydrograph

The Clark unit hydrograph procedure (Clark, 1945) is based upon using a time-area relationship of the watershed that defines the cumulative area of the watershed contributing runoff to the watershed outlet as a function of time. Ordinates of the time-area relationship are converted to a volume of runoff per second for an excess (1 cm or 1 in) and interpolated to the given time interval to define a

Table 8.4.1 Steps to Compute Snyder's Synthetic Unit Hydrograph

Step 0 Measured information from topography map of watershed

- L = main channel length in mi (km)
- L_c = length of the main stream channel from outflow point of watershed to a point opposite the centroid of the watershed in mi (km)
- A = watershed area in mi^2 (km^2)

Regional parameters C_t and C_p determined from similar watersheds.

Step 1 Determine time to peak (t_p) and duration (t_r) of the standard unit hydrograph:

$$t_p = C_1 C_t (L \cdot L_c)^{0.3} \quad \text{(hours)}$$

$$t_r = t_p / 5.5 \quad \text{(hours)}$$

where $C_1 = 1.0$ (0.75 for SI units)

Step 2 Determine the time to peak t_{pR} for the desired duration t_R:

$$t_{pR} = t_p + 0.25(t_R - t_r) \quad \text{(hours)}$$

Step 3 Determine the peak discharge, Q_{pR}, in cfs/in ((m^3/s)/cm in SI units)

$$Q_{pR} = \frac{C_2 C_p A}{t_{pR}}$$

where $C_2 = 640$ (2.75 for SI units)

Step 4 Determine the width of the unit hydrograph at $0.5Q_{pR}$ and $0.75Q_{pR}$. W_{50} is the width at 50% of the peak given as

$$W_{50} = \frac{C_{50}}{(Q_{pR}/A)^{1.08}}$$

where $C_{50} = 770$ (2.14 for SI units). W_{75} is the width at 75% of the peak given as

$$W_{75} = \frac{C_{75}}{(Q_{pR}/A)^{1.08}}$$

where $C_{75} = 440$ (1.22 for SI units)

Step 5 Determine the base, T_b, such that the unit hydrograph represents 1 in (1 cm in SI units) of direct runoff volume:

$$1 \text{ in} = \left[\left(\frac{W_{50}+T_b}{2}\right)(0.5Q_{pR}) + \left(\frac{W_{75}+W_{50}}{2}\right)(0.25Q_{pR}) + \frac{1}{2}W_{75}(0.25Q_{pR})\right]\left(\text{hr} \times \frac{\text{ft}^3}{\text{sec}}\right)$$

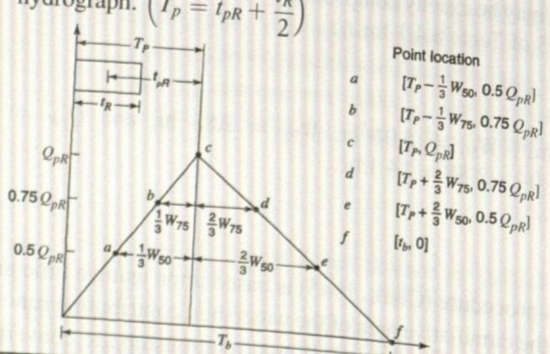
$$\left(\frac{1}{A(\text{mi})^2} \times \frac{1 \text{ mi}^2}{(5{,}280)^2 \text{ ft}^2} \times \frac{12 \text{ in}}{\text{ft}} \times \frac{3{,}600 \text{ sec}}{\text{hr}}\right)$$

Solving for T_b, we get

$$T_b = 2{,}581\frac{A}{Q_{pR}} - 1.5W_{50} - W_{75}$$

for A in mi^2, Q_{pR} in cfs, W_{50} and W_{75} in hours.

Step 6 Define known points of the unit hydrograph. $\left(T_p = t_{pR} + \frac{t_R}{2}\right)$



translation hydrograph. The assumption is a pure translation of the rainfall excess without storage effects of the watershed to define a translation hydrograph. This translation hydrograph is routed through a linear reservoir ($S = RQ$) in order to simulate the effects of storage of the watershed, where R is the storage coefficient. The resulting routed hydrograph for the instantaneous excess is averaged to produce the unit hydrograph for the excess (1 cm or 1 inch) occurring in the given time interval.

Synthetic time-area relationships can be expressed in the following form such as that used by the U.S. Army Corps of Engineers (1990)

$$A/A_c = 1.414(t/T_c)^{1.5} \qquad \text{for } 0 \le t/T_c \le 0.5 \tag{8.4.6a}$$

and

$$A/A_c = 1 - 1.414(1 - t/T_c)^{1.5} \qquad \text{for } 0.5 \le t/T_c \le 1.0 \tag{8.4.6b}$$

where A is the contributing area at time t, A_c is the total watershed area, and T_c is the time of concentration of the watershed area. Some investigators such as Ford et al. (1980) indicate that a detailed time-area curve usually is not necessary for accurate synthetic unit hydrograph estimation. A comparison of the HEC (Hydrologic Engineering Center) default relation found in HEC-1 and HEC-HMS to that used in Phoenix, Arizona is given in Table 8.4.2.

The average instantaneous flow over time interval t to $t + \Delta t$, defining the translation hydrograph is denoted as $I_{ave,t}$. To compute $I_{ave,t}$, assuming a pure translation over a Δt hr time period, the flowing equations are used. For 1 cm (0.01 m), the $I_{ave,t}$ in m^3/s is expressed as

$$I_{ave,t} = (0.01 \text{ m})(\Delta A \text{ km}^2)(10^6 \text{ m}^2/\text{km}^2)(1/\Delta t \text{ hr})(1 \text{ hr}/3600 \text{ sec}) \tag{8.4.7}$$

where ΔA is the incremental area in km^2 between runoff isochrones (lines of equal runoff at a certain time) and Δt is the time increment in hours. For 1 in in the $I_{ave,t}$ in ft^3/s is expressed as

$$I_{ave,t} = (1 \text{ in})(1 \text{ ft}/12 \text{ in})(\Delta A \text{ mi}^2)(5280 \text{ ft}^2/\text{mi}^2)(1/\Delta t \text{ hr})(1 \text{ hr}/3600 \text{ sec}) \tag{8.4.8}$$

Storage effects in the watershed are incorporated by routing the translation hydrograph through a linear reservoir using the continuity equation

$$I_{ave,t} - 0.5(Q_t + Q_{t+\Delta t}) = (S_t + S_{t+\Delta t})/\Delta t \tag{8.4.9}$$

Table 8.4.2 Synthetic Dimensionless Time-Area Relations

	Contributing area, as a percent of total area		
Time as a percent of T_c	Urban* watersheds	Natural* watersheds	HEC default
0	0	0	0.0
10	5	3	4.5
20	16	5	12.6
30	30	8	23.2
40	65	12	35.8
50	77	20	50.0
60	84	43	64.2
70	90	75	76.8
80	94	90	87.4
90	97	96	95.5
100	100	100	100

*Flood Control District of Maricopa County, Phoenix, AZ (1995)

$I_{ave,t}$ is the average instantaneous inflow over time interval t to $t + \Delta t$, defining the translation hydrograph, Q is the outflow from the linear reservoir, and S is the storage in the linear reservoir. In the linear reservoir assumption, storage S_t is assumed to be linearly proportional to Q_t

$$S_t = RQ_t \tag{8.4.10}$$

in which R is the proportionality constant (watershed storage coefficient) with units of time. Combining equations (8.4.9) and (8.4.10) the routing equation is

$$Q_{t+\Delta t} = CI_{ave,t} + (1 - C)Q_t \tag{8.4.11}$$

where $I_{ave,t}$ is the average translated runoff (inflow rate to the linear reservoir) during time increment, and

$$C = 2\,\Delta t/(2R + \Delta t) \tag{8.4.12}$$

The discharge, Q_t, from the linear reservoir now includes the effects of the storage of the watershed. This hydrograph is an instantaneous (duration = 0 hr) unit hydrograph, which is converted to the desired unit hydrograph of duration τ by averaging the ordinates over the time interval

$$U_\tau(t) = 0.5[Q_t + Q_{t-\tau}] \tag{8.4.13}$$

The above equations are the basis for the Clark unit hydrograph procedure.

EXAMPLE 8.4.2

A small watershed has an area of 10 km^2 and a time of concentration of 1.5 hr. The watershed storage coefficient is 0.75 hr. The time-area relationship is the HEC default values in Table 8.4.2. Compute the 1-hr unit hydrograph for this small watershed. Use a time interval of 0.5-hr for the computations.

SOLUTION

First the incremental areas of the watershed are determined using the HEC time-area relationship in Table 8.4.2. The translation hydrograph is then computed by applying equation (8.4.7) to each ΔA. Next the translation hydrograph is routed through a linear reservoir using the given watershed storage coefficient. Compute the routing coefficient $C = 2(0.5)/[2(0.75) + 0.5] = 0.5$, so the linear reservoir routing equation is $Q_{t+\Delta t} = 0.5I_{ave,t} + (1 - 0.5)Q_t = 0.5I_{ave,t} + 0.5Q_t$. The unit hydrograph ordinates are computed using equation (8.4.13) with $\tau = 1$ hr. For example, the unit hydrograph ordinate for time 0.5 hr is $(0 + 7.56)/2 = 3.78$ m^3/s, for time 1.0 hr is $(0 + 16.4)/2 = 8.20$ m^3/s and for 1.5 hr is $(7.56 + 15.8)/2 = 11.7$ m^3/s.

t (hr)	t/T_c	A/A_c	A (km^2)	ΔA (km^2)	$I_{ave,t}$ (m^3/s)	$Q_{t+\Delta t}$ (m^3/s)	$U_\tau(t)$ (m^3/s)
0.0							0.0
0.5	0.333	0.272	2.72	2.72	15.1	7.56	3.78
1.0	0.667	0.728	7.28	4.56	25.3	16.4	8.20
1.5	1.0	1.0	10.0	2.72	15.1	15.8	11.7
2.0					0.0	7.89	12.2
2.5						3.94	9.86
3.0						1.97	4.93
3.5						0.986	2.46
4.0						0.493	1.23
4.5						0.247	0.616
5.0						0.123	0.308

The use of the model HEC-HMS (HEC-1) requires the time of concentration, T_c, and the storage coefficient R. Various locations have developed relationships for these parameters to make the methods more accurate and easier to use. Straub et al. (2000) developed the following equations for small rural watersheds (0.02-2.3 mi^2) in Illinois

$$T_c = 1.54L^{0.875}\,S_o^{-0.181} \tag{8.4.14}$$

and

$$R = 16.4L^{0.342}\,S_o^{-0.790} \tag{8.4.15}$$

where L is the stream length measured along the main channel from the watershed outlet to the watershed divide in miles, and S_o is the main-channel slope determined from elevations at points that represent 10 and 85 percent of the distance along the channel from the watershed outlet to the watershed divide in ft/mi.

Others have used time of concentration equations that have included additional parameters. For example Phoenix, Arizona (Flood Control District of Maricopa County) uses the following time of concentration equations developed by Papadakis and Kazan (1987) for urban areas

$$T_c = 11.4L^{0.50}\,K_b^{0.52}\,S_o^{-0.31}\,i^{-0.38} \tag{8.4.16}$$

where T_c is the time of concentration in hours, L is the length of the longest flow path in miles, K_b is a watershed resistance coefficient ($K_b = -0.00625 \log A + 0.04$) for commercial and residential areas, A is the watershed area in acres, S is the slope of the flow path in ft/mi, and i is the rainfall intensity in in/hr. The storage coefficient is

$$R = 0.37T_c^{1.11}\,A^{-0.57}\,L^{0.80} \tag{8.4.17}$$

where A is the watershed area in mi^2.

.5 S-HYDROGRAPHS

In order to change a unit hydrograph from one duration to another, the *S-hydrograph method*, which is based on the principle of superposition, can be used. An S-hydrograph results theoretically from a continuous rainfall excess at a constant rate for an indefinite period. This curve (see Figure 8.5.1) has an S-shape with the ordinates approaching the rate of rainfall excess at the time of equilibrium.

Basically the S-curve (hydrograph) is the summation of an infinite number of t_R duration unit hydrographs, each lagged from the preceding one by the duration of the rainfall excess, as illustrated in Figure 8.5.2.

A unit hydrograph for a new duration t_R' is obtained by: (1) lagging the S-hydrograph (derived with the t_R duration unit hydrographs) by the new (desired) duration t_R', (2) *subtracting* the two S-hydrographs from one another, and (3) *multiplying* the resulting hydrograph ordinates by the ratio t_R/t_R'. Theoretically the S-hydrograph is a smooth curve because the input rainfall excess is assumed to be a constant, continuous rate. However, the numerical processes of the procedures may result in an undulatory form that may require smoothing or adjustment of the S-hydrograph.

XAMPLE 8.5.1

Using the 2-hr unit hydrograph in Table 8.5.1, construct a 4-hr unit hydrograph (adapted from Sanders (1980)).

LUTION

See the computations in Table 8.5.1.

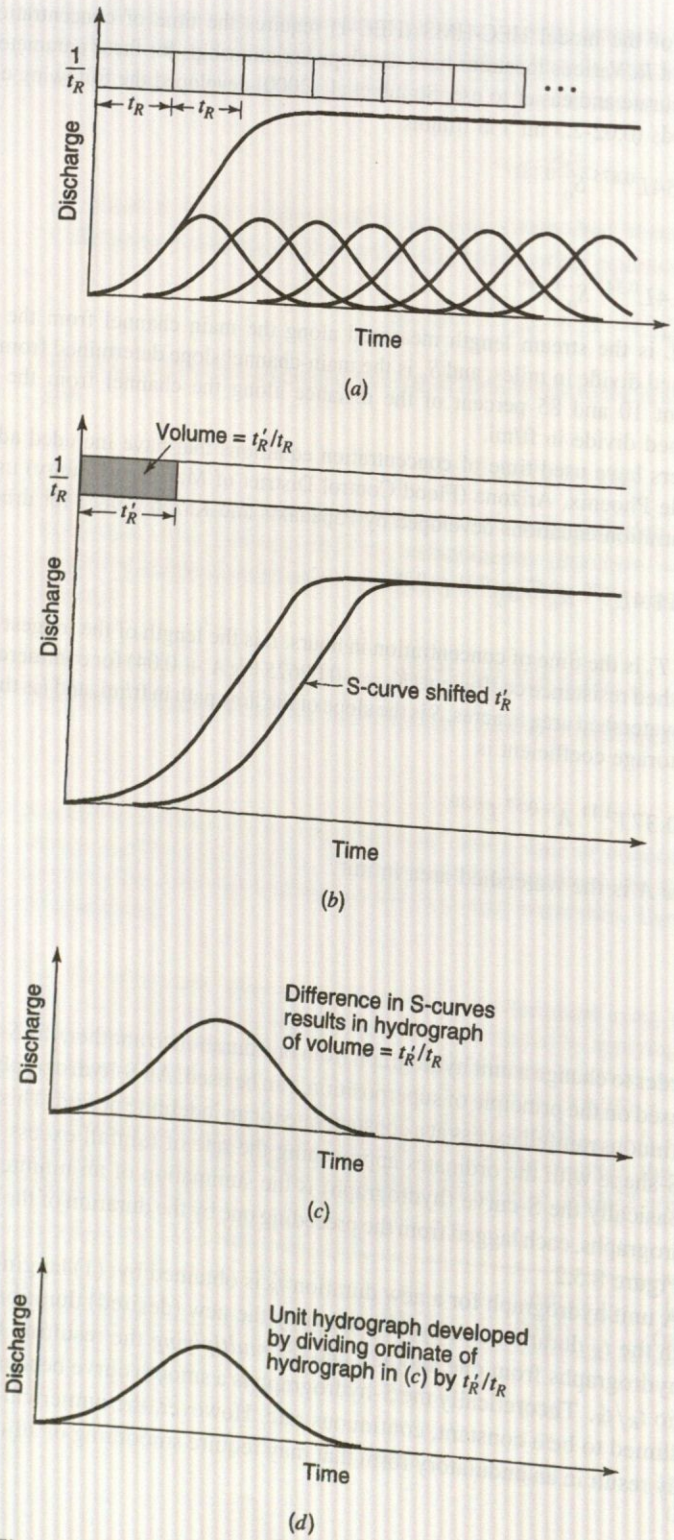


Figure 8.5.1 Development of a unit hydrograph for duration t_R' from a unit hydrograph for duration t_R.

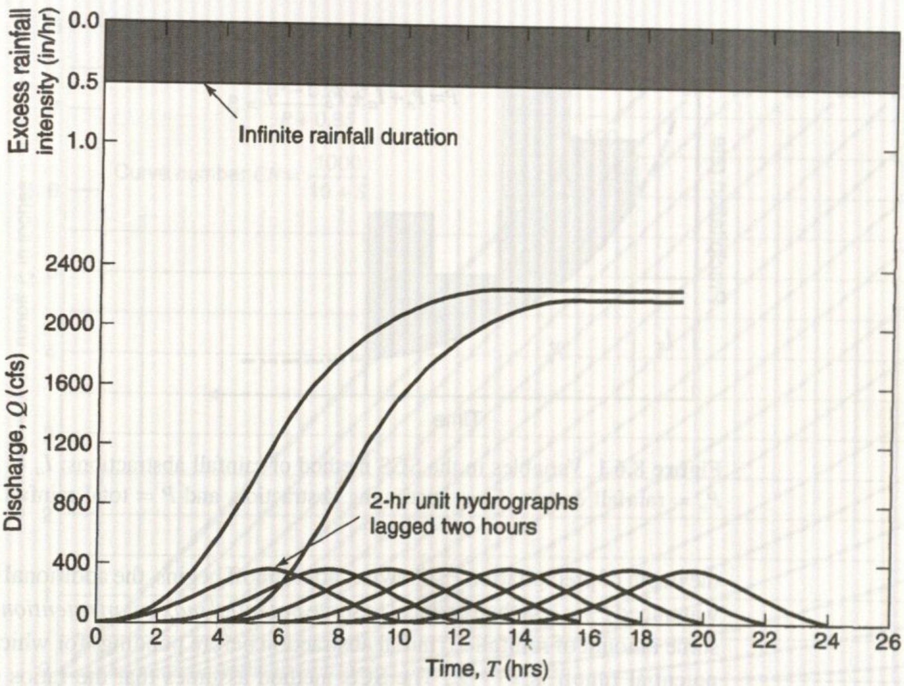


Figure 8.5.2 Graphical illustration of the S-curve construction (from Masch (1984)).

Table 8.5.1 S-Curve Determined from a 2-hr Unit Hydrograph to Estimate a 4-hr Unit Hydrograph

Time (hr)	2-hr unit hydrograph (cfs/in)	Lagged 2-hr unit hydrograph (cfs/in)		S-curve	Lagged S-curve	4-hr hydrograph	4-hr-unit hydrograph (cfs/in)
0	0			0	—	0	0
2	69	0		69	—	69	34
4	143	69	0 . . .	212	0	212	106
6	328	143	69 . . .	540	69	471	235
8	389	328	143 . . .	929	212	717	358
10	352	389	328	1281	540	741	375
12	266	352	389	1547	929	618	309
14	192	266	352	1739	1281	458	229
16	123	192	.	1862	1547	315	158
18	84	123	.	1946	1739	207	103
20	49	84	.	1995	1862	133	66
22	20	49	.	2015	1946	69	34
24	0	20	.	*2015	1995	20	10
26	0	0	. . . .	*2015	2015	0	0

*Adjusted values

Source: Sanders (1980).

NRCS (SCS) RAINFALL-RUNOFF RELATION

The U.S. Department of Agriculture Soil Conservation Service (SCS) (1972), now the National Resources Conservation Service (NRCS), developed a rainfall-runoff relation for watershed. For the storm as a whole, the depth of excess precipitation or direct runoff P_e is always less than or equal to

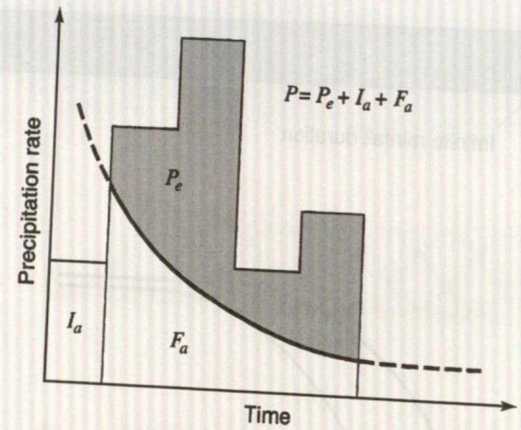


Figure 8.6.1 Variables in the SCS method of rainfall abstractions: I_a = initial abstraction, P_e = rainfall excess, F_a = continuing abstraction, and P = total rainfall.

the depth of precipitation P; likewise, after runoff begins, the additional depth of water retained in the watershed F_a is less than or equal to some *potential maximum retention* S (see Figure 8.6.1). There is some amount of rainfall I_a (initial abstraction before ponding) for which no runoff will occur, so the potential runoff is $P - I_a$. The SCS method assumes that the ratios of the two actual to the two potential quantities are equal, that is,

$$\frac{F_a}{S} = \frac{P_e}{P - I_a} \quad \frac{\text{Actual}}{\text{Potential}} \tag{8.6.1}$$

From continuity,

$$P = P_e + I_a + F_a \tag{8.6.2}$$

so that combining equations (8.6.1) and (8.6.2) and solving for P_e gives

$$P_e = \frac{(P - I_a)^2}{P - I_a + S} \tag{8.6.3}$$

which is the basic equation for computing the depth of excess rainfall or direct runoff from a storm by the SCS method.

From the study of many small experimental watersheds, an empirical relation was developed for I_a:

$$I_a = 0.2S \tag{8.6.4}$$

so that equation (8.6.3) is now expressed as

$$P_e = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{8.6.5}$$

Empirical studies by the SCS indicate that the potential maximum retention can be estimated as

$$S = \frac{1000}{CN} - 10 \tag{8.6.6}$$

where CN is a runoff curve number that is a function of land use, antecedent soil moisture, and other factors affecting runoff and retention in a watershed. The curve number is a dimensionless number defined such that $0 \le CN \le 100$. For impervious and water surfaces $CN = 100$; for natural surfaces $CN < 100$.

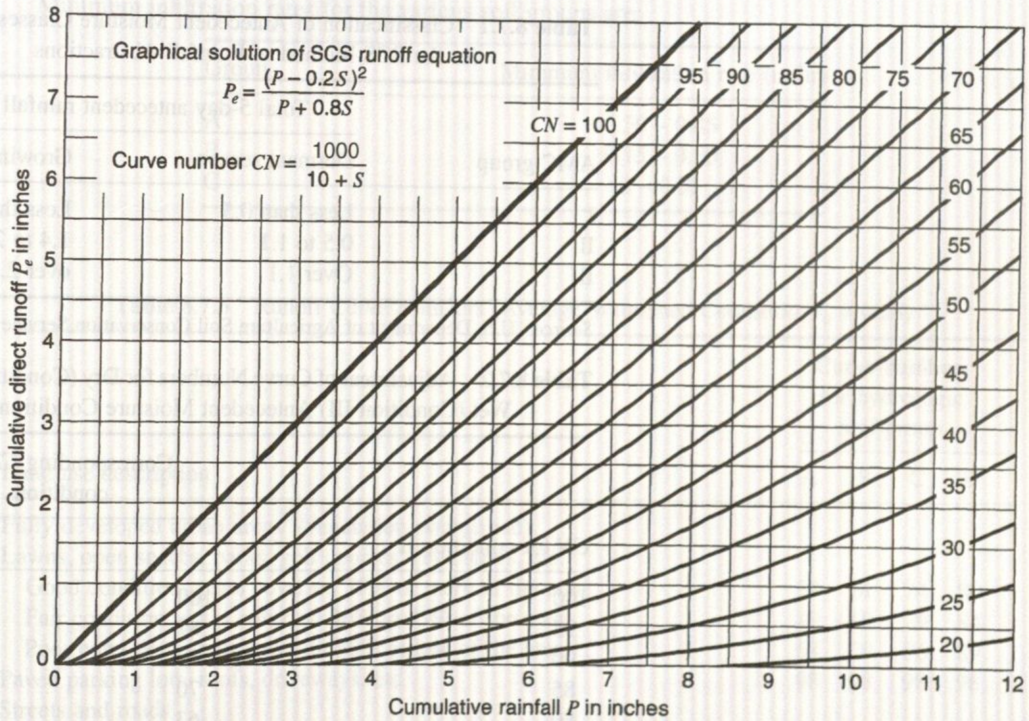


Figure 8.6.2 Solution of the SCS runoff equations (from U.S. Department of Agriculture Soil Conservation Service (1972)).

The SCS rainfall-runoff relation (equation (8.6.5)) can be expressed in graphical form using the curve numbers as illustrated in Figure 8.6.2. Equation (8.6.5) or Figure 8.6.2 can be used to estimate the volume of runoff when the precipitation volume P and the curve number CN are known.

7 CURVE NUMBER ESTIMATION AND ABSTRACTIONS

7.1 Antecedent Moisture Conditions

The curve numbers shown in Figure 8.6.2 apply for normal *antecedent moisture conditions* (AMC II). Antecedent moisture conditions are grouped into three categories:

AMC I—Low moisture
AMC II—Average moisture condition, normally used for annual flood estimates
AMC III—High moisture, heavy rainfall over the preceding few days

For dry conditions (AMC I) or wet conditions (AMC III), equivalent curve numbers can be computed using

$$CN(\mathrm{I}) = \frac{4.2CN(\mathrm{II})}{10 - 0.058CN(\mathrm{II})} \tag{8.7.1}$$

and

$$CN(\mathrm{III}) = \frac{23CN(\mathrm{II})}{10 + 0.13CN(\mathrm{II})} \tag{8.7.2}$$

The range of antecedent moisture conditions for each class is shown in Table 8.7.1. Table 8.7.2 lists the adjustment of curve numbers to conditions I and III for known II conditions.

Table 8.7.1 Classification of Antecedent Moisture Classes (AMC) for the SCS Method of Rainfall Abstractions

AMC group	Total 5-day antecedent rainfall (in)	
	Dormant season	Growing season
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	over 2.1

Source: U.S. Department of Agriculture Soil Conservation Service (1972).

Table 8.7.2 Adjustment of Curve Numbers for Dry (Condition I) and Wet (Condition III) Antecedent Moisture Conditions

CN for condition II	Corresponding *CN* for condition	
	I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

Source: U.S. Department of Agriculture Soil Conservation Service (1972).

8.7.2 Soil Group Classification

Curve numbers have been tabulated by the Soil Conservation Service on the basis of soil type and land use in Table 8.7.3. The four soil groups in Table 8.7.3 are described as:

Group A: Deep sand, deep loess, aggregated silts
Group B: Shallow loess, sandy loam
Group C: Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay
Group D: Soils that swell significantly when wet, heavy plastic clays, and certain saline soils

The values of *CN* for various land uses on these soil types are given in Table 8.7.3. For a watershed made up of several soil types and land uses, a composite *CN* can be calculated.

Minimum infiltration rates for the various soil groups are:

Group	Minimum infiltration rate (in/hr)
A	0.30 – 0.45
B	0.15 – 0.30
C	0 – 0.05

Table 8.7.3 Runoff Curve Numbers (Average Watershed Condition, $I_a = 0.2S$)

Land use description		Average % impervious[b]	Curve numbers for hydrologic soil group A	B	C	D
Fully developed urban areas[a] (vegetation established)						
Lawns, open spaces, parks, golf courses, cemeteries, etc.						
Good condition; grass cover on 75% or more of the area			39	61	74	80
Fair condition; grass cover on 50% to 75% of the area			49	69	79	84
Poor condition; grass cover on 50% or less of the area			68	79	86	89
Paved parking lots, roofs, driveways, etc.			98	98	98	98
Streets and roads						
Paved with curbs and storm sewers			98	98	98	98
Gravel			76	85	89	91
Dirt			72	82	87	89
Paved with open ditches			83	89	92	93
Commercial and business areas		85	89	92	94	95
Industrial districts		72	81	88	91	93
Row houses, town houses, and residential with lot sizes 1/8 acre or less		65	77	85	90	92
Residential: average lot size						
1/4 acre		38	61	75	83	87
1/3 acre		30	57	72	81	86
1/2 acre		25	54	70	80	85
1 acre		20	51	68	79	84
2 acre		12	46	65	77	82
Developing urban areas[c] (no vegetation established)						
Newly graded area			77	86	91	94
Cover						
Land use	Treatment of practice	Hydrologic condition[d]				
Cultivated agricultural land						
Fallow	Straight row		77	86	91	94
	Conservation tillage	Poor	76	85	90	93
	Conservation tillage	Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90

(*Continued*)

Table 8.7.3 (*Continued*)

Cover: Land use	Cover: Treatment of practice	Hydrologic condition[d]	Curve numbers for hydrologic soil group: A	B	C	D
	Conservation tillage	Good	64	75	82	85
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured and conservation tillage	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
	Contoured and terraces	Good	62	71	78	81
	Contoured and terraces and conservation tillage	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
	Conservation tillage	Good	60	72	80	84
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured and conservation tillage	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
	Contoured and terraces	Good	59	70	78	81
	Contoured and terraces and conservation tillage	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded legumes or rotation meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
	Contoured and terraces	Good	51	67	76	80
Noncultivated agricultural land, pasture or range	No mechanical treatment	Poor	68	79	86	89
	No mechanical treatment	Fair	49	69	79	84
	No mechanical treatment	Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		—	30	58	71	78
Forested—grass or orchards—evergreen or deciduous		Poor	55	73	82	86
		Fair	44	65	76	82
		Good	32	58	72	79
Brush		Poor	48	67	77	83
		Good	20	48	65	73
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		—	59	74	82	86
Forest-range Herbaceous		Poor		79	86	92
		Fair		71	80	89
		Good		61	74	84

Table 8.7.3 (*Continued*)

Cover			Curve numbers for hydrologic soil group			
Land use	Treatment of practice	Hydrologic condition[d]	A	B	C	D
Oak–aspen		Poor		65	74	
		Fair		47	57	
		Good		30	41	
Juniper–grass		Poor		72	83	
		Fair		58	73	
		Good		41	61	
Sage–grass		Poor		67	80	
		Fair		50	63	
		Good		35	48	

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

[b]Includes paved streets.

[c]Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area *CN* and Figure 8.7.1*a* or *b*, the composite *CN* can be computed for any degree of development.

[d]For conservation tillage in poor hydrologic condition, 5 percent to 20 percent of the surface is covered with residue (less than 750-lb/acre row crops or 300-lb/acre small grain).

For conservation tillage in good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 750-lb/acre row crops or 300-lb/acre small grain).

[e]Close-drilled or broadcast.

For noncultivated agricultural land:
Poor hydrologic condition has less than 25 percent ground cover density.
Fair hydrologic condition has between 25 percent and 50 percent ground cover density.
Good hydrologic condition has more than 50 percent ground cover density.

For forest–range:
Poor hydrologic condition has less than 30 percent ground cover density.
Fair hydrologic condition has between 30 percent and 70 percent ground cover density.
Good hydrologic condition has more than 70 percent ground cover density.

Source: U.S. Department of Agriculture Soil Conservation Service (1986).

8.7.3 Curve Numbers

Table 8.7.3 gives the curve numbers for average watershed conditions, $I_a = 0.2S$, and antecedent moisture condition II. For watersheds consisting of several subcatchments with different *CN*s, the area-averaged composite *CN* can be computed for the entire watershed. This analysis assumes that the impervious areas are directly connected to the watershed drainage system (Figure 8.7.1*a*). If the percent imperviousness is different from the value listed in Table 8.7.3 or if the impervious areas are not directly connected, then Figures 8.7.1*a* or *b*, respectively, can be used. The pervious *CN* used in these figures is equivalent to the open-space *CN* in Table 8.7.3. If the total impervious area is less than 30 percent, Figure 8.7.1*b* is used to obtain a composite *CN*. For natural desert landscaping and newly graded areas, Table 8.7.3 gives only the *CN*s for pervious areas.

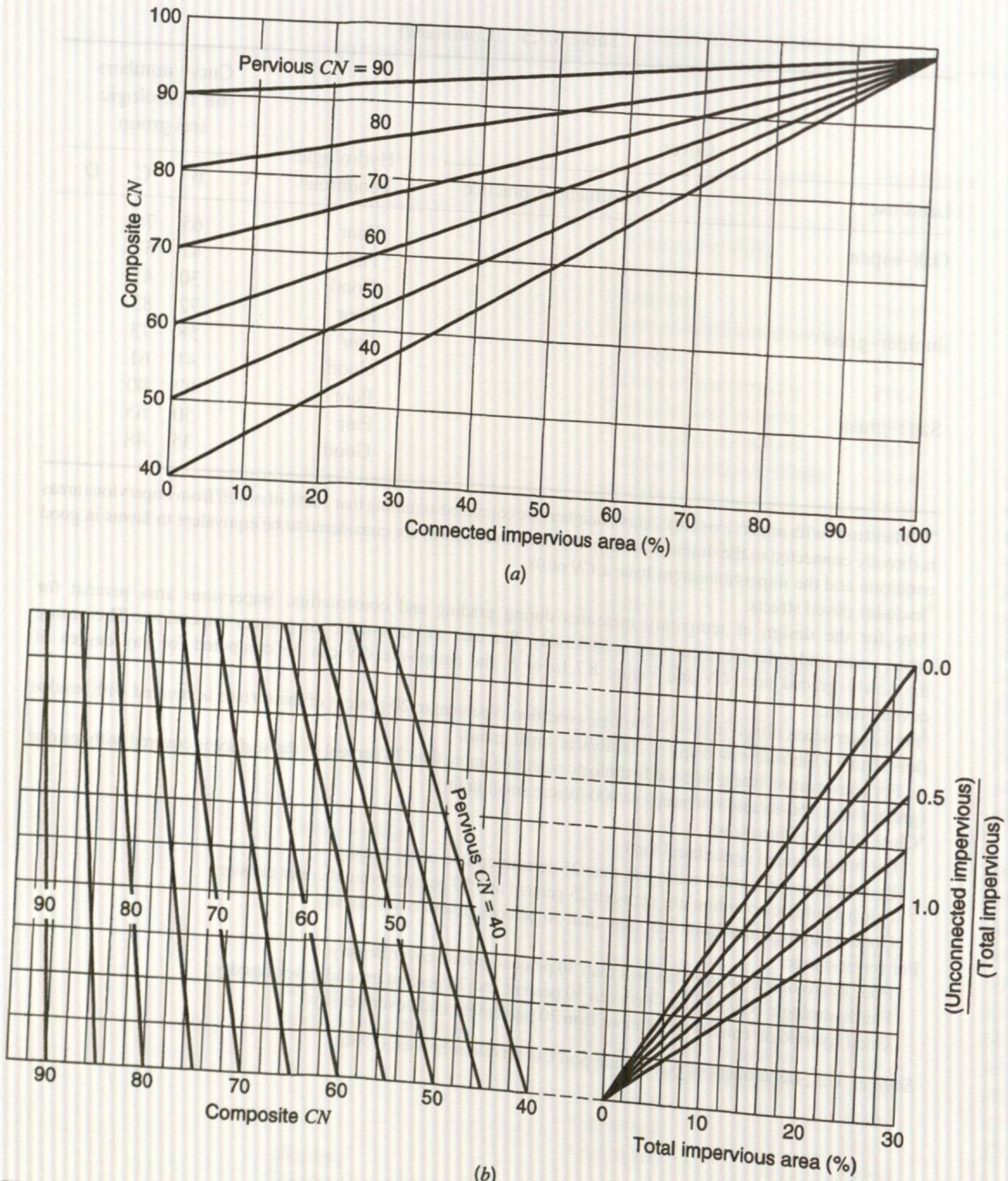


Figure 8.7.1 Relationships for determining composite *CN*. (*a*) Connected impervious area; (*b*) Unconnected impervious area (from U.S. Department of Agriculture Soil Conservation Service (1986)).

EXAMPLE 8.7.1

Determine the weighted curve numbers for a watershed with 40 percent residential (1/4-acre lots), 25 percent open space, good condition, 20 percent commercial and business (85 percent impervious), and 15 percent industrial (72 percent impervious), with corresponding soil groups of C, D, C, and D.

SOLUTION

The corresponding curve numbers are obtained from Table 8.7.3:

Land use (%)	Soil group	Curve number
40	C	83
25	D	80
20	C	94
15	D	93

The weighted curve number is

$$\begin{aligned} CN &= 0.40(83) + 0.25(80) + 0.20(94) + 0.15(93) \\ &= 33.2 + 20 + 18.8 + 13.95 \\ &= 85.95 (\text{use } 86) \end{aligned}$$

EXAMPLE 8.7.2

The watershed in example 8.7.1 experienced a rainfall of 6 in. What is the runoff volume?

SOLUTION

Using equation (8.6.5), P_e = runoff volume is

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

where S is computed with the weighted curve number of 86 from example 8.7.1:

$$S = \frac{1000}{86} - 10 = 1.63$$

So

$$\begin{aligned} P_e &= \frac{[6 - 0.2(1.63)]^2}{6 + 0.8(1.63)} = \frac{32.19}{7.3} \\ &= 4.41 \text{ in of runoff} \end{aligned}$$

EXAMPLE 8.7.3

For the watershed in examples 8.7.1 and 8.7.2, the 6-in rainfall pattern was 2 in the first hour, 3 in the second hour, and 1 in the third hour. Determine the cumulative rainfall and cumulative rainfall excess as functions of time.

SOLUTION

The initial abstractions are computed as $I_a = 0.2S$ with $S = 1.63$ from example 8.7.2, so $I_a = 0.2(1.63) =$ 033 in. The remaining losses for time period (the first hour) are computed using the following equation, derived by combining equations (8.6.1) and (8.6.2):

$$F_{a,t} = \frac{S(P_t - I_a)}{P_t - I_a + S} = \frac{1.63(P_t - 0.33)}{P_t - 0.33 + 1.63} = \frac{1.63(P_t - 0.33)}{P_t + 1.3}$$

$$F_{a,1} = \frac{1.63(2 - 0.33)}{2 + 1.3} = 0.82 \text{ in}$$

The total loss for the first hour is 0.33 + 0.82 = 1.15 in, and the excess is

$$P_{e1} = P_1 - I_a - F_{a,1} = 2 - 0.33 - 0.82 = 0.85 \text{ in}$$

For the second hour, $P_t = 2 + 3 = 5$ in, so

$$F_{a,2} = \frac{1.63(5 - 0.33)}{5 + 1.3} = 1.21 \text{ in}$$

and the cumulative rainfall excess is $P_{e_2} = 5 - 0.33 - 1.21 = 3.46$ in.

For the third hour, $P_3 = 2 + 3 + 1 = 6$ in, so

$$F_{a,3} = \frac{1.63(6 - 0.33)}{6 + 1.3} = 1.27 \text{ in}$$

and $P_{e_3} = 6 - 0.33 - 1.27 = 4.40$ in (which compares well with the results of example 8.7.2). The results are summarized below, along with the rainfall excess hyetograph.

Time (h)	Cumulative rainfall P_t (in)	Cumulative abstractions I_a (in)	Cumulative abstractions $F_{a,t}$ (in)	Cumulative rainfall excess P_e (in)	Rainfall excess hyetograph (in)
1	2	0.33	0.82	0.85	0.85
2	5	0.33	1.21	3.46	2.61
3	6	0.33	1.27	4.40	0.94

8.8 NRCS (SCS) UNIT HYDROGRAPH PROCEDURE

The SCS dimensionless unit hydrograph and mass curve are shown in Figure 8.8.1 and tabulated in Table 8.8.1. The SCS dimensionless equivalent triangular unit hydrograph is also shown in Figure 8.8.1. The following section discusses how to develop a unit hydrograph from these dimensionless unit hydrographs.

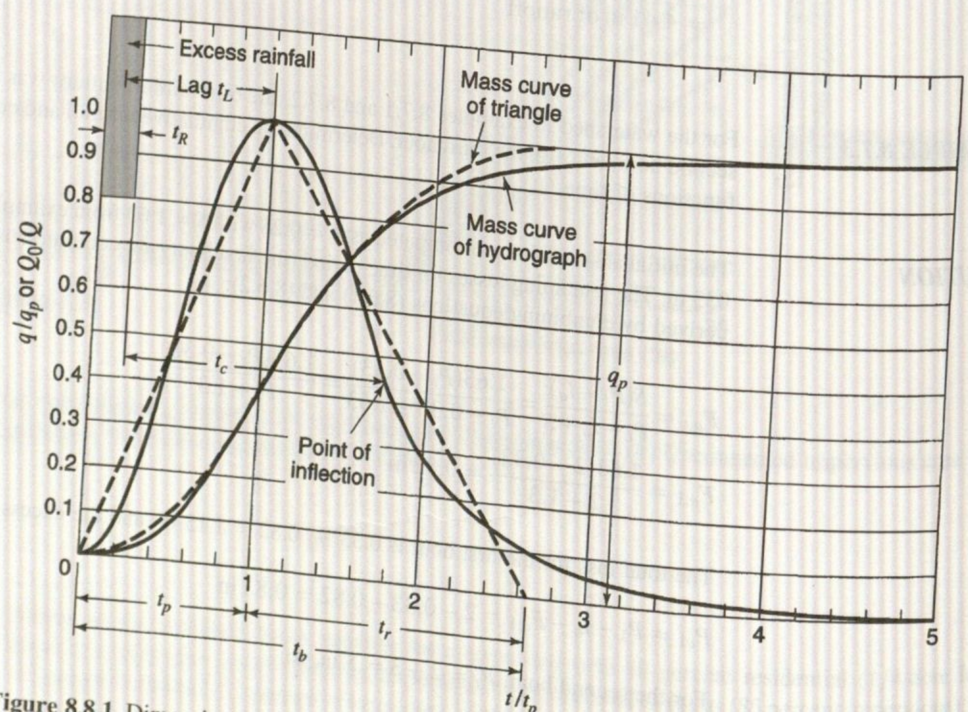


Figure 8.8.1 Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph (from U.S. Department of Agriculture Soil Conservation Service (1986)).

Table 8.8.1 Ratios for Dimensionless Unit Hydrograph and Mass Curve

Time ratios t/t_p	Discharge ratios q/q_p	Mass curve ratios Q_a/Q
0	0.000	0.000
0.1	0.030	0.001
0.2	0.100	0.006
0.3	0.190	0.012
0.4	0.310	0.035
0.5	0.470	0.065
0.6	0.660	0.107
0.7	0.820	0.163
0.8	0.930	0.228
0.9	0.990	0.300
1.0	1.000	0.375
1.1	0.990	0.450
1.2	0.930	0.522
1.3	0.860	0.589
1.4	0.780	0.650
1.5	0.680	0.700
1.6	0.560	0.751
1.7	0.460	0.790
1.8	0.390	0.822
1.9	0.330	0.849
2.0	0.280	0.871
2.2	0.207	0.908
2.4	0.147	0.934
2.6	0.107	0.953
2.8	0.077	0.967
3.0	0.055	0.977
3.2	0.040	0.984
3.4	0.029	0.989
3.6	0.021	0.993
3.8	0.015	0.995
4.0	0.011	0.997
4.5	0.005	0.999
5.0	0.000	1.000

Source: U.S. Department of Agriculture Soil Conservation Service (1972).

8.8.1 Time of Concentration

The *time of concentration* for a watershed is the time for a particle of water to travel from the hydrologically most distant point in the watershed to a point of interest, such as the outlet of the watershed. SCS has recommended two methods for time of concentration, the *lag method* and the *upland*, or *velocity method.*

The lag method relates the *time lag* (t_L), defined as the time in hours from the center of mass of the rainfall excess to the peak discharge, to the slope (Y) in percent, the hydraulic length (L) in feet, and the potential maximum retention (S), expressed as

$$t_L = \frac{L^{0.8}(S+1)^{0.7}}{1900Y^{0.5}} \tag{8.8.1}$$

The SCS uses the following relationship between the time of concentration (t_c) and the lag (t_L):

$$t_c = \frac{5}{3} t_L \tag{8.8.2}$$

or

$$t_c = \frac{L^{0.8}(S+1)^{0.7}}{1140Y^{0.5}} \tag{8.8.3}$$

where t_c is in hours. Refer to Figure 8.8.1 to see the SCS definition of t_c and t_L.

The velocity (upland) method is based upon defining the time of concentration as the ratio of the hydraulic flow length (L) to the velocity (V):

$$t_c = \frac{L}{3600V} \tag{8.8.4}$$

where t_c is in hours, L is in feet, and V is in ft/s. The velocity can be estimated knowing the land use and the slope in Figure 8.8.2. Alternatively, we can think of the concentration as being the sum of

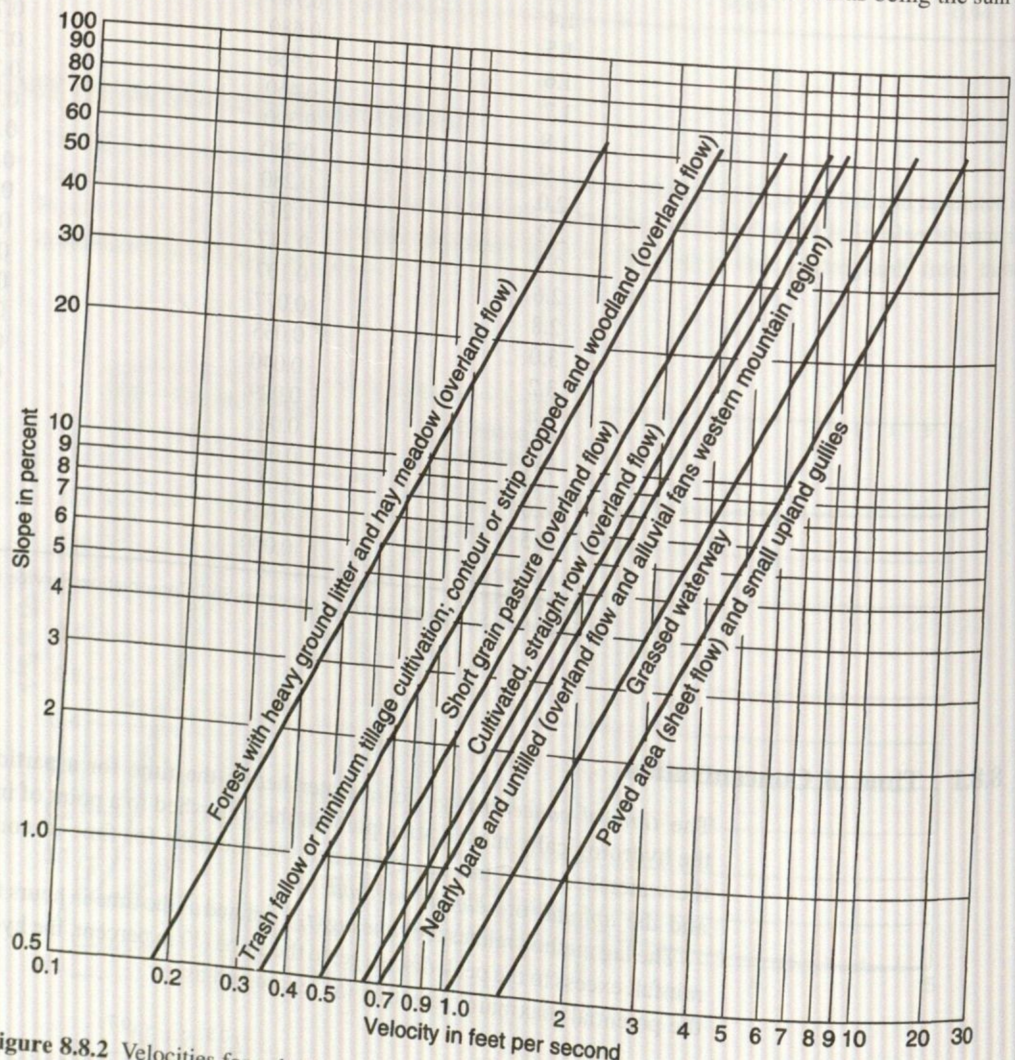


Figure 8.8.2 Velocities for velocity upland method of estimating t_c (from U.S. Department of Agriculture Soil Conservation Service (1986)).

travel times for different segments

$$t_c = \frac{1}{3600}\sum_{i=1}^{k}\frac{L_i}{V_i} \tag{8.8.5}$$

for k segments, each with different land uses.

8.8.2 Time to Peak

Time to peak (t_p) is the time from the beginning of rainfall to the time of the peak discharge (Figure 8.8.1)

$$t_p = \frac{t_R}{2} + t_L \tag{8.8.6}$$

where t_p is in hours, t_R is the duration of the rainfall excess in hours, and t_L is the lag time in hours. The SCS recommends that t_R be 0.133 of the time of concentration of the watershed, t_c:

$$t_R = 0.133t_c \tag{8.8.7}$$

and because $t_L = 0.6t_c$ by equation (8.8.2), then by equation (8.8.6) we get

$$\begin{aligned} t_p &= \frac{0.133t_c}{2} + 0.6t_c \\ t_p &= 0.67t_c \end{aligned} \tag{8.8.8}$$

8.8.3 Peak Discharge

The area of the unit hydrograph equals the volume of direct runoff Q, which was estimated by equation (8.6.5). With the equivalent triangular dimensionless unit hydrograph of the curvilinear dimensionless unit hydrograph in Figure 8.8.1, the time base of the dimensionless triangular unit hydrograph is 8/3 of the time to peak t_p, as compared to $5t_p$ for the curvilinear. The areas under the rising limb of the two dimensionless unit hydrographs are the same (37 percent).

Based upon geometry (Figure 8.8.1), we see that

$$Q = \frac{1}{2}q_p(t_p + t_r) \tag{8.8.9}$$

for the direct runoff Q, which is 1 in where t_r is the recession time of the dimensionless triangular unit hydrograph and q_p is the peak discharge. Solving equation (8.8.9) for q_p gives

$$q_p = \frac{Q}{t_p}\left[\frac{2}{1 + t_r/t_p}\right] \tag{8.8.10}$$

Letting $K = \left[\frac{2}{1 + t_r/t_p}\right]$, then

$$q_p = \frac{KQ}{t_p} \tag{8.8.11}$$

where Q is the volume, equals to 1 in for a unit hydrograph.

The above equation can be modified to express q_p in ft^3/sec, t_p in hours, and Q in inches:

$$q_p = 645.33K\frac{AQ}{t_p} \tag{8.8.12}$$

The factor 645.33 is the rate necessary to discharge 1 in of runoff from 1 mi^2 in 1 hr. Using $t_r = 1.67t_p$ gives $K = [2/(1 + 1.67)] = 0.75$; then equation (8.8.12) becomes

$$q_p = \frac{484AQ}{t_p} \tag{8.8.13}$$

For SI units,

$$q_p = \frac{2.08AQ}{t_p} \tag{8.8.14}$$

where A is in square kilometers.

The steps in developing a unit hydrograph are:

Step 1 Compute the time of concentration using the lag method (equation (8.8.3)) or the velocity method (equation (8.8.4) or (8.8.5)).

Step 2 Compute the time to peak $t_p = 0.67t_c$ (equation (8.8.8)) and then the peak discharge q_p using equation (8.8.13) or (8.8.14).

Step 3 Compute time base t_b and the recession time t_r:
Triangular hydrograph: $t_b = 2.67t_p$
Curvilinear hydrograph: $t_b = 5t_p$
$t_r = t_b - t_p$

Step 4 Compute the duration $t_R = 0.133\ t_c$ and the lag $t_L = 0.6\ t_c$ by using equations (8.8.7) and (8.8.2), respectively.

Step 5 Compute the unit hydrograph ordinates and plot. For the triangular only t_p, q_p, and t_r are needed. For the curvilinear, use the dimensionless ratios in Table 8.8.1.

EXAMPLE 8.8.1

For the watershed in example 8.7.1, determine the triangular SCS unit hydrograph. The average slope of the watershed is 3 percent and the area is 3.0 mi^2. The hydraulic length is 1.2 mi.

SOLUTION

Step 1 The time of concentration is computed using equation (8.8.1), with S = 1.63 from example 8.7.2:

$$t_L = \frac{(6336)^{0.8}(1.63+1)^{0.7}}{1900\sqrt{3}} = 0.66 \text{ hr}$$

and $t_c = \frac{5}{3}t_L = 1.1$ hr

Step 2 The time to peak $t_p = 0.67t_c = 0.67(1.1) = 0.74$ hr.

Step 3 The time base is $t_b = 2.67t_p = 1.97$ hr.

Step 4 The duration is $t_R = 0.133t_c = 0.133(1.1) = 0.15$ hr, and t_L is 0.66 hr.

Step 5 The peak is (for $Q = 1$ in)

$$q_p = \frac{484AQ}{t_p} = \frac{484(3)(1)}{0.74} = 1962 \text{ cfs}.$$

In summary, the triangular unit hydrograph has a peak of 1962 cfs at the time to peak of 0.74 hr with a time base of 1.97 hr. This is a 0.15-hr duration unit hydrograph.

8.9 KINEMATIC-WAVE OVERLAND FLOW RUNOFF MODEL

Hortonian overland flow occurs when the rainfall rate exceeds the infiltration capacity and sufficient water ponds on the surface to overcome surface tension effects and fill small depressions. Overland flow is surface runoff that occurs in the form of sheet flow on the land surface without concentrating

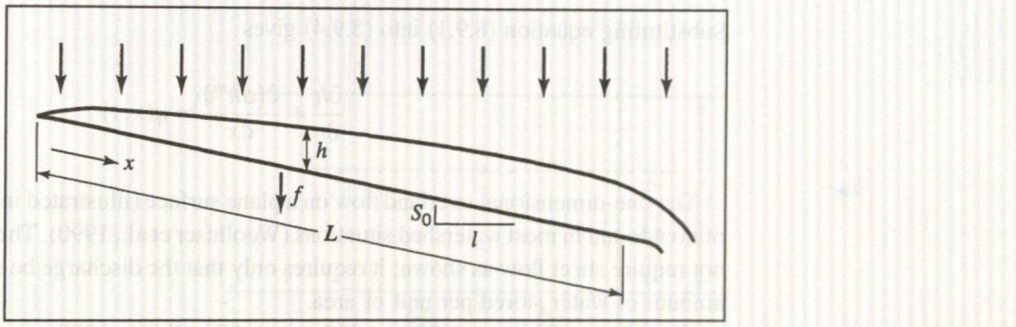


Figure 8.9.1 Definition sketch of overland flow on a plane as a one-dimensional flow (from Woolhiser et al. (1990)).

in clearly defined channels (Ponce, 1989). For the purposes of rainfall-runoff analysis, this flow can be viewed as a one-dimensional flow process (Figure 8.9.1) in which the flux is proportional to some power of the storage per unit area, expressed as (Woolhiser et al., 1990):

$$Q = \alpha h^m \tag{8.9.1}$$

where Q is the discharge per unit width, h is the storage of water per unit area (or depth if the surface is a plane), and α and m are parameters related to slope, surface roughness, and whether the flow is laminar or turbulent.

The mathematical description of overland flow can be accomplished through the continuity equation in one-dimensional form and a simplified form of the momentum equation. This model is referred to as the *kinematic wave model. Kinematics* refers to the study of motion exclusive of the influence of mass and force. A *wave* is a variation in flow, such as a change in flow rate or water surface elevation. *Wave celerity* is the velocity with which this variation travels. *Kinematic waves* govern flow when inertial and pressure forces are negligible.

The *kinematic wave equations* (also see Chapter 9) for a one-dimensional flow are expressed as follows:

Continuity:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q(x, t) \tag{8.9.2}$$

Momentum:

$$S_0 - S_f = 0 \tag{8.9.3}$$

where A is the cross-sectional area of flow, Q is the discharge, t is time, x is the spatial coordinate, $q(x, t)$ is the lateral inflow rate, S_0 is the overland flow slope, and S_f is the friction slope.

Equation (8.9.3) indicates that the gravity and friction forces are balanced, so that flow does not accelerate appreciably. The inertial (local and convective acceleration) term and pressure term are neglected in the kinematic wave model (refer to Section 9.4). Eliminating these terms eliminates the mechanism to describe backwater effects and flood wave peak attenuation.

Considering that h is the storage per unit area or depth, then $A = h$, so that equation (8.9.2) becomes

$$\frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = q(x, t) \tag{8.9.4}$$

Substituting equation (8.9.1) into (8.9.4) gives

$$\frac{\partial h}{\partial t}+\frac{\partial(\alpha h^m)}{\partial x}=q(x,t) \tag{8.9.5}$$

The one-dimensional overland flow on a plane surface (illustrated in Figure 8.9.1) is not the type of flow found in most watershed situations (Woolhiser et al., 1990). The kinematic assumption does not require sheet flow as shown; it requires only that the discharge be some unique function of the amount of water stored per unit of area.

Woolhiser and Liggett (1967) and Morris and Woolhiser (1980) showed that the kinematic-wave formulation is an excellent approximation for most overland flow conditions. Keep in mind that these equations are a simplification of the Saint-Venant equations (see Chapter 9).

The kinematic-wave equation (8.9.5) for overland flow can be solved numerically using a four-point implicit method where the finite-difference approximations for the spatial and temporal derivatives are, respectively,

$$\frac{\partial h}{\partial x}=\theta\frac{h_{i+1}^{j+1}-h_i^{j+1}}{\Delta x}+(1-\theta)\frac{h_{i+1}^{j}-h_i^{j}}{\Delta x} \tag{8.9.6}$$

and

$$\frac{\partial h}{\partial t}=\frac{1}{2}\left[\frac{h_i^{j+1}-h_i^j}{\Delta t}+\frac{h_{i+1}^{j+1}-h_{i+1}^{j}}{\Delta t}\right]$$

or

$$\frac{\partial h}{\partial t}=\frac{h_i^{j+1}+h_{i+1}^{j+1}-h_i^j-h_{i+1}^j}{2\Delta t} \tag{8.9.7}$$

and

$$q=\frac{1}{2}\left(\bar{q}_{i+1}+\bar{q}_i\right) \tag{8.9.8}$$

where θ is a weighting parameter for spatial derivative, $\theta=\Delta' t/\Delta t$ (see Figure 8.9.2). The derivative $\partial h/\partial t$ is the average of the temporal derivatives at locations i and $i+1$ or for the midway locations between i and $i+1$, and $\bar{q}_i$ and $\bar{q}_{i+1}$ are the average lateral inflows at i and $i+1$, respectively. Notation for the finite-difference grid is shown in Figure 8.9.2. Substituting these finite difference expressions (8.9.6), (8.9.7), and (8.9.8) into (8.9.5) and simplifying results in the following finite-difference equation:

$$\begin{aligned}&h_{i+1}^{j+1}-h_{i+1}^{j}+h_i^{j+1}-h_i^j\\&+\frac{2\Delta t}{\Delta x}\left\{\theta\left[\alpha_{i+1}^{j+1}(h_{i+1}^{j+1})^m-\alpha_i^{j+1}(h_i^{j+1})^m\right]+(1-\theta)\left[\alpha_{i+1}^{j}(h_{i+1}^{j})^m-\alpha_i^{j}(h_i^{j})^m\right]\right\}\\&-\Delta t(\bar{q}_{i+1}+\bar{q}_i)=0\end{aligned} \tag{8.9.9}$$

The only unknown in the above equation is h_{i+1}^{j+1}, which must be solved by using Newton's method (see Appendix A). Using Manning's equation to express equation (8.9.1), $Q=\alpha h^m$, we

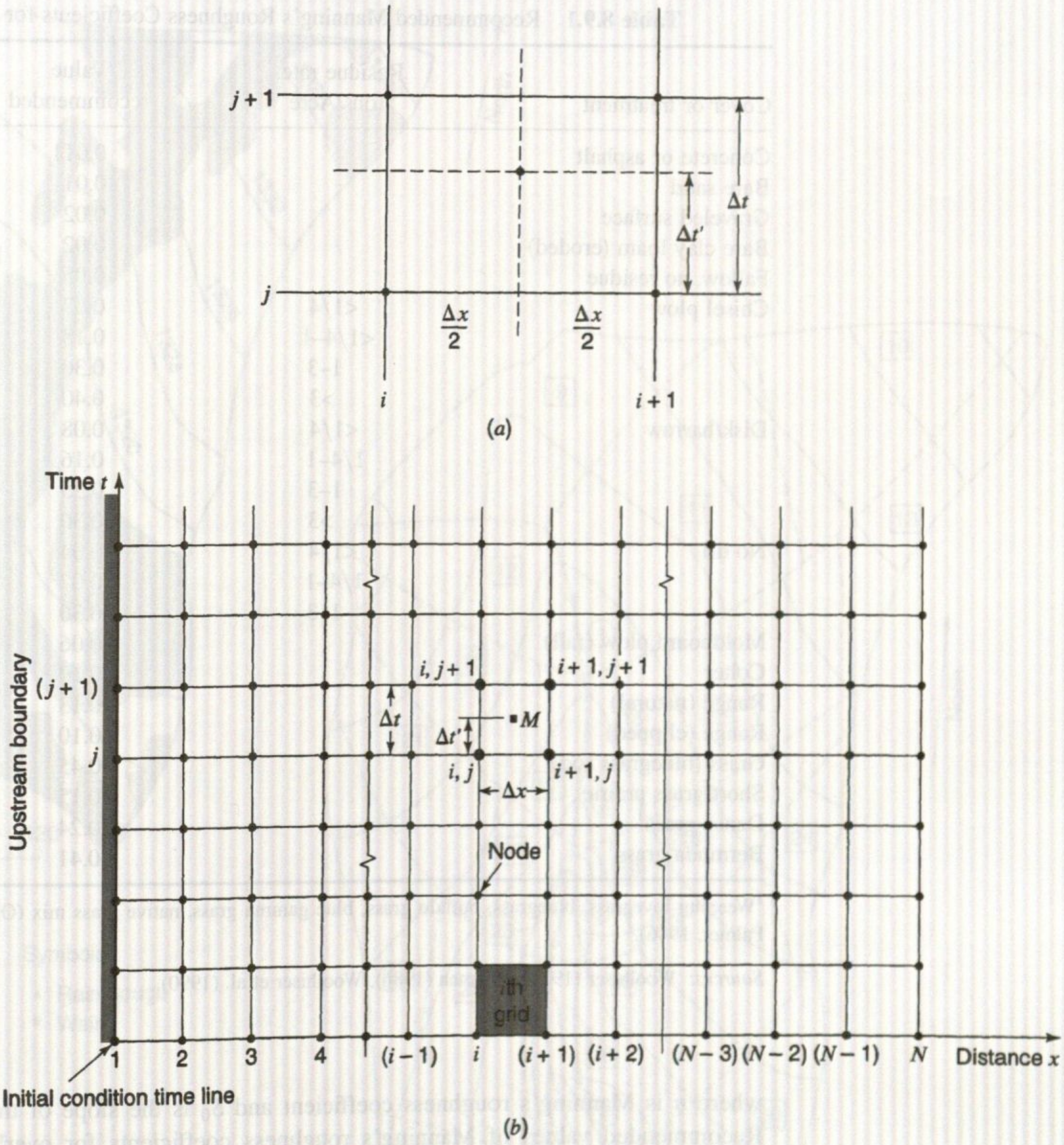


Figure 8.9.2 The *x*-*t* solution plane: The finite-difference forms of the Saint-Venant equations are solved at a discrete number of points (values of the independent variables *x* and *t*) arranged to the time axis represent locations along the plane, and those parallel to the distance axis represent times. (*a*) Four points of finite-difference grid; (*b*) Finite difference grid.

find

$$Q = \left[\frac{1.49S^{1/2}}{n}\right] h^{5/3} \tag{8.9.10}$$

where

$$\alpha = \frac{1.49S_0^{1/2}}{n}, \ m = 5/3 \tag{8.9.11}$$

Table 8.9.1 Recommended Manning's Roughness Coefficients for Overland Flow

Cover or treatment	Residue rate, Tons/Acre	Value recommended	Range
Concrete or asphalt		0.011	0.010–0.013
Bare sand		0.01	0.010–0.016
Graveled surface		0.02	0.012–0.03
Bare clay loam (eroded)		0.02	0.012–0.033
Fallow, no residue		0.05	0.006–0.16
Chisel plow	<1/4	0.07	0.006–0.17
	<1/4–1	0.18	0.07–0.34
	1–3	0.30	0.19–0.47
	>3	0.40	0.34–0.46
Disk/harrow	<1/4	0.08	0.008–0.41
	1/4–1	0.16	0.10–0.25
	1–3	0.25	0.14–0.53
	>3	0.30	—
No till	<1/4	0.04	0.03–0.07
	1/4–1	0.07	0.01–0.13
	1–3	0.30	0.16–0.47
Moldboard plow (fall)		0.06	0.02–0.10
Colter		0.10	0.05–0.13
Range (natural)		0.13	0.01–0.32
Range (clipped)		0.10	0.02–0.24
Grass (bluegrass sod)		0.45	0.39–0.63
Short grass prairie		0.15	0.10–0.20
Dense grass[1]		0.24	0.17–0.30
Bermuda grass[1]		0.41	0.30–0.48

[1]Weeping lovegrass, bluegrass, buffalo grass, blue gamma grass, native grass mix (OK), alfalfa, lespedeza (from Palmer, 1946).

Sources: Woolhiser (1975), Engman (1986), Woolhiser et al. (1990).

where n is Manning's roughness coefficient and S_0 is the slope of the overland flow plane. Recommended values of Manning's roughness coefficients for overland flow are given in Table 8.9.1. The *time to equilibrium* of a plane of length L and slope S_0 can be derived using Manning's equation as

$$t_c = \frac{nL}{1.49 S_0^{1/2} h^{2/3}} \tag{8.9.12}$$

The U.S. Department of Agriculture Agricultural Research Service (Woolhiser et al., 1990) has developed a KINematic runoff and EROSion model referred to as KINEROS. This model is event-oriented, i.e., it is a physically based model describing the processes of interception, infiltration, surface runoff, and erosion from small agricultural and urban watersheds. The model is distributed because flows are modeled for both the watershed and the channel elements, as illustrated in Figures 8.9.3 and 8.9.4. The model is *event-oriented* because it does not have components describing evapotranspiration and soil water movement between storms. In other words, there is no hydrologic balance between storms.

Figures 8.9.3 and 8.9.4 illustrate that the approach to describing a watershed is to divide it into a branching system of channels with plane elements contributing lateral flow to channels. The

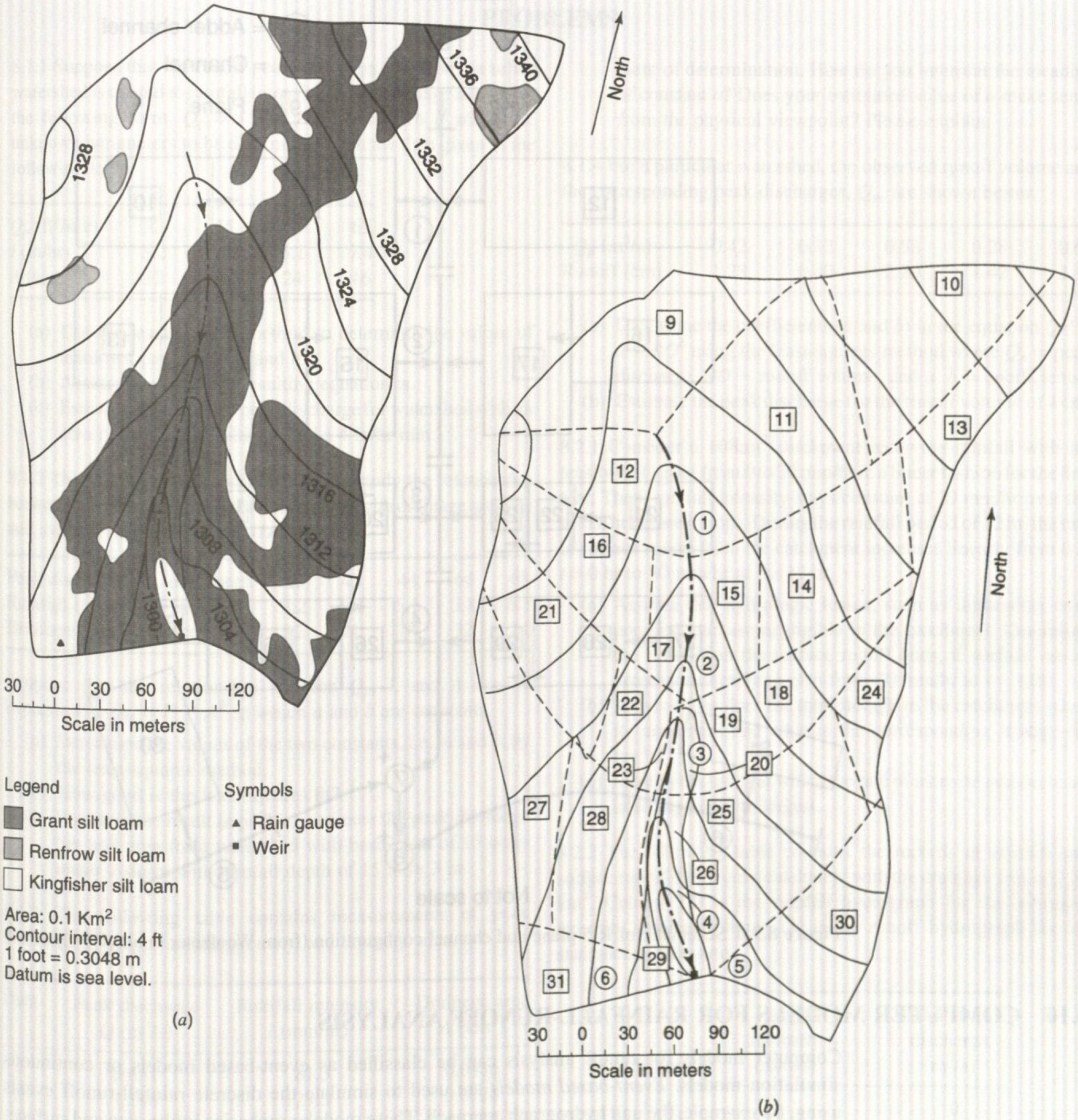


Figure 8.9.3 R-5 catchment, Chickasha, OK. (*a*) Contour map; (*b*) Division into plane and channel elements (from Woolhiser et al. (1990)).

KINEROS model takes into account interception, infiltration, overland flow routing, channel routing, reservoir routing, erosion, and sediment transport. Overland flow routing has been described in this section. Channel routing is performed using the kinematic-wave approximation described in Chapter 9. The reservoir routing in KINEROS is basically a level-pool routing procedure, as described in Chapter 9.

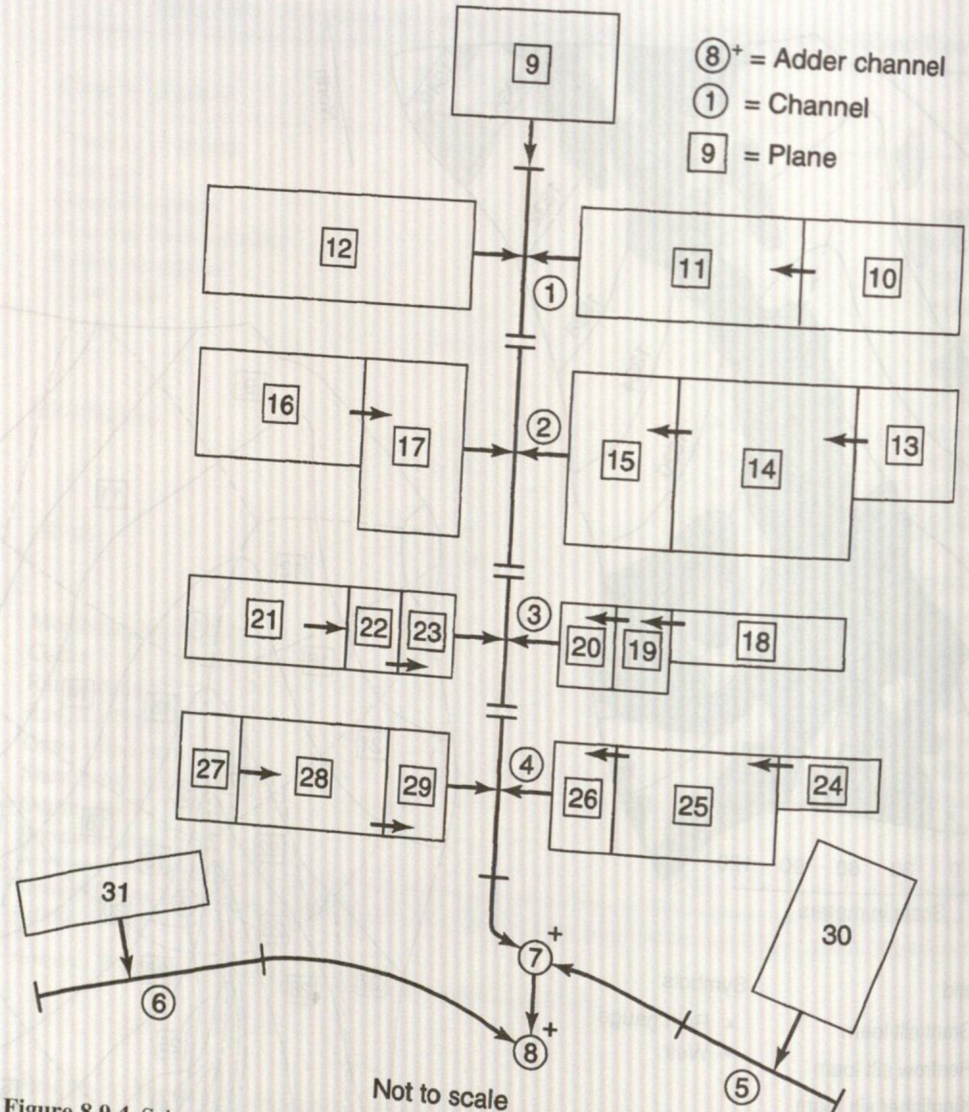


Figure 8.9.4 Schematic of R-5 plane and channel configuration (from Woolhiser et al. (1990)).

8.10 COMPUTER MODELS FOR RAINFALL-RUNOFF ANALYSIS

Computer models for runoff analysis can be classified as event-based models or continuous simulation models. *Event-based models* are used to simulate the discrete rainfall-runoff events using, for example, the unit hydrograph approach. These models emphasize infiltration and surface-runoff components with the objective of determining direct runoff, and are applicable to excess-water flow calculations in cases where direct runoff is the major contributor to streamflow. The HEC-HMS (HEC-1) and the TR-20 and TR-55 models are single-event models. The KINEROS model (Woolhiser et al., 1990) described in the previous section is an overland flow model based on the kinematic-wave routing. This model is a *distributed event-based model*. Other examples include the kinematic-wave model for overland flow routing in the HEC-HMS (HEC-1) model.

Continuous-simulation models account for the overall moisture balance of the basin, including moisture accounting between storm events. These models explicitly account for all runoff components including surface flow and indirect runoff such as interflow and baseflow, and are well suited for *long-term runoff forecasting*.

PROBLEMS

8.1.1 Suppose that the runoff peak discharge (Q_p) from an urban watershed is related to rainfall intensity (I) and drainage area (A) in the following form, $Q_p = a \times I^b \times A^b$, in which a and b are unknown parameters to be determined from the data given in the following table.

Q_p (ft^3/sec)	23	45	68	62
I (in/hr)	3.2	4.6	6.1	7.4
A (acres)	12	21	24	16

(a) Use the least-squares method to determine the values of unknown parameters a and b.
(b) Assess how good the resulting equation is.
(c) Estimate the expected peak discharge for watershed with an area of 20 acres resulting from a 4-in/hr rain.

8.1.2 The following data were obtained to study the relationship between peak discharge (Q_p) of a watershed with drainage area (A) and average rainfall intensity (I).

Peak discharge (Q_p in ft^3/sec)	23	45	44	64	68
Rainfall intensity (I in in/hr)	3.2	4.2	5.1	3.8	6.1
Drainage area (A in acres)	12	21	18	32	24

Suppose that the relationship between Q_p, I, and A can be expressed as $Q_p = I^a \times A^b$ in which a and b are constants.

(a) Determine the values of the two constants, i.e., a and b, by the least-squares method.
(b) How good is the least-squares fit?
(c) Based on the result in part (a), estimate the peak discharge per unit area for a watershed with basin area of 15 acres under a storm with rainfall depth of 15 in in 3 hr.

8.1.3 The following table contains measurements of peak discharge, average rainfall intensity, and drainage area.

Data	Peak discharge Q (m^3/s)	Rainfall intensity I (mm/hr)	Drainage area A (km^2)
1	0.651	81.3	0.0486
2	1.246	129.5	0.0728
3	1.812	96.5	0.1295
4	1.926	154.9	0.0971

(a) Determine the coefficients a and b in the equation $Q = a \times (I \times A)^b$ using the least-squares method.
(b) Determine the corresponding coefficient of determination and standard error of estimate for part (a).
(c) Physically, when a storm with constant rainfall intensity I continues indefinitely, the term $I \times A$ represents the steady-state peak discharge. Under the steady-state condition, let $b = 1$ in the above equation. Determine the least-squares estimate for the constant a and the corresponding coefficient of determination. How do you interpret the meaning of constant a? Does your estimated value of a make sense from the physical viewpoint? Please explain.

8.1.4 For a particular watershed, the observed runoff volume and the corresponding peak discharges, Q_p, are shown below.

Q_p (m^3/s)	0.42	0.12	0.58	0.28	0.66
Runoff (cm)	1.28	0.40	2.29	1.42	3.19

(a) Determine the coefficients (a and b) in the equation $QP = a \times RO^b$ using the least-squares method where Q_p = peak discharge, RO = runoff volume, and a, b = coefficients.
(b) Estimate the peak discharge for the runoff volume of 4 cm.

8.2.1 Consider a 10-km^2 catchment receiving rainfall with intensity that varies from 0 to 30 mm/hr in a linear fashion for the first 6 hr. Then, rainfall intensity stays constant at 30 mm/hr over the next 6 hr before it stops. During the rainfall period of 12 hr, the rate of surface runoff from the catchment increases linearly from 0 at $t = 0$ hr to 30 mm/hr at $t = 12$ hr.

(a) Assume that hydrologic losses, such as infiltration, evaporation, etc. are negligible in the catchment. Determine the time when the surface runoff ends, if surface runoff decreases linearly to zero from 30 mm/hr at $t = 12$hr.
(b) At what time does the total storage in the catchment reach its maximum and what is the corresponding storage (in km^3)?
(c) Identify the times at which the rate of increase and decrease in storage are the largest.

8.2.2 The following table contains the records of rainfall and surface runoff data from a catchment with the drainage area of 2.25 km^2. Column (2) is the rainfall hyetograph for the averaged intensity whereas column (3) is the runoff hydrograph for instantaneous flow rate.

Time	Avg. intensity (mm/h)	Instantaneous discharge (m^3/s)
1:00		0
1:10	60	12
1:20	150	24
1:30	30	48
1:40	18	30
1:50	60	18
2:00	72	24
2:10	24	30
2:20	12	18
2:30		12
2:40		6
2:50		0

(a) Determine the rolling-time and clock-time maximum rainfall depth (in mm) for durations of 30 min and 50 min.
(b) Determine the time and the corresponding volume (in m^3) when the maximum storage occurs. Assume that the initial storage volume is zero.
(c) Determine the percentage of rainfall volume that becomes the surface runoff.

8.2.3 Based on the table given below showing the hydrograph ordinates at 24-hour intervals, use graph paper to (a) identify the time instant at which the direct runoff ends and (b) determine the recession constant, k, in the following equation:

$$Q(t_2) = Q(t_1) \times e^{-k(t_2 - t_1)}$$

in which $t_2 > t_1$ are two time points on the baseflow recession of a hydrograph.

Time (day)	Flow (m^3/s)	Time (day)	Flow (m^3/s)
1	6	8	91
2	970	9	79
3	707	10	68
4	400	11	58
5	254	12	50
6	162	13	43
7	121	14	37

8.2.4 The following table contains the records of rainfall and surface runoff data of a particular storm event for a watershed with the drainage area of 8.4 km^2.

(a) Determine the maximum rainfall intensity for durations of 30 min, 60 min, and 120 min.
(b) Determine the percentage of total rainfall that is lost from appearing as the surface runoff.
(c) Determine the time period that the storage volume of the water is increasing in the watershed.

Time of day	Cumulative rainfall (mm)	Instantaneous runoff (m^3/s)
4:00	0	0.0
4:30	30	0.3
5:00	80	4.2
5:30	90	11.3
6:00	95	13.0
6:30	127	34.0
7:00	130	45.3
7:30	132	21.2
8:00		9.3
8:30		4.8
9:00		2.5
9:30		1.4
10:00		0.8
10:30		0.0

8.2.5 The following table contains the records of rainfall and surface runoff data of a storm event for a watershed having the drainage area of 8.0 km^2.

(a) Determine the maximum rainfall intensity for durations of 15 min, 30 min, and 60 min.
(b) Determine the percentage of total rainfall that is lost from appearing as the surface runoff.
(c) Determine the time period that the storage volume of the water is increasing in the watershed and the maximum storage volume (in cm).
(d) Determine the Φ index and the corresponding rainfall excess hyetograph.

Time	Cumul. rainfall (mm)	Instant. flow rate (m^3/s)
4:00	0	0
4:15	15	1
4:30	40	4
4:45	45	11
5:00	48	13
5:15	64	35
5:30	65	45
5:45	66	21
6:00	66	10
6:15	66	5
6:30		3
6:45		1
7:00		0

8.2.6 Consider a watershed with drainage area of 1 km^2, For a given storm event, the recorded average rainfall intensity and total instantaneous discharge at the outlet of the watershed are listed in the following table. Assume that the baseflow is 1 m^3/s and rainfall amount in the first 30 min is the initial loss.

(a) Determine the total amount of rainfall.
(b) Determine the volume of direct runoff.
(c) Determine the average infiltration *rate*, i.e., ϕ index
(d) Determine the rainfall excess *intensity* hyetograph corresponding to the ϕ index obtained in (c). Also, what is the duration of rainfall excess hyetograph?

Time (min)	Intensity (mm/h)	Discharge (m^3/s)
0	10	1
30	20	1
60	38	4
90	26	8
120	10	6
150	20	4
180	24	3
210		1

8.2.7 On a particular day in 1990, there was a rainstorm event that produced the following rainfall and runoff on a 36-km^2 drainage basin.

Time (min)	0	15	30	45	60	75	90	105	120
Cumulative rainfall (mm)	0	10	50	75	90	100			
Instantaneous runoff (m^3/s)	10	30	160	360	405	305	125	35	10

For the above rainstorm event, assume that the baseflow is 10 m^3/s.

(a) Find the volume of direct runoff (in mm).
(b) Assuming the initial loss is 10 mm, determine the value of ϕ index (in mm/hr) and the corresponding effective rainfall intensity hyetograph (in mm/hr).

8.3.1 Determine the 4-hr unit hydrograph using the following data for a watershed having a drainage area of 200 km^2, assuming a constant rainfall abstraction rate and a constant baseflow of 20 m^3/s.

Four-hour period	1	2	3	4	5	6	7	8	9	10	11
Rainfall (cm)	1.0	2.5	4.0	2.0							
Storm flow (m^3/s)	20	30	60	95	130	170	195	175	70	25	20

8.3.2 Using the unit hydrograph developed in problem 8.3.1, determine the direct runoff from the 200 km^2 watershed using the following rainfall excess pattern.

Four-hour period	1	2	3	4
Rainfall excess (cm)	2.0	3.0	0.0	1.5

8.3.3 The ordinates at 1-hr intervals of a 1-hr unit hydrograph are 100, 300, 500, 700, 400, 200, and 100 cfs. Determine the direct runoff hydrograph from a 3-hr storm in which 1 in of excess rainfall occurred in the first hour, 2 in the second hour, and 0.5 in the third hour. What is the area of the watershed in mi^2?

8.3.4 You are responsible for developing the runoff hydrograph for a watershed that fortunately has gauged information at a very nearby location on the stream. You have obtained information on an actual rainfall-runoff event for this watershed. This information includes the actual rainfall hyetograph and the resulting runoff hydrograph. You have been asked to develop the runoff hydrograph for a design rainfall event. The peak discharge from this new developed storm hydrograph will be used to design a hydraulic structure. You will assume a constant baseflow and a constant rainfall abstraction. Explain the hydrologic analysis procedure that you will use to solve this problem.

8.3.5 Consider a drainage basin with an area of 226.8 km^2. From a storm event, the observed cumulative rainfall depth and the corresponding runoff hydrograph are given in the following table. Assume that the baseflow is 10 m^3/s.

Time (hr)	0	3	6	9	12	15	18	21	24	27	30	33	36
Cumulative rainfall (cm)	0.0	0.5	1.5	2.5	4.5	6.5	8.0	9.5					
Discharge (m^3/s)	10	10	30	50	130	175	260	240	220	145	70	40	10

(a) Determine the value of the ϕ index and the corresponding effective rainfall hyetograph.
(b) Set up the system of equations, $Pu = Q$, and solve for the unit hydrograph.

8.3.6 Suppose the 4-hr unit hydrograph for a watershed is

Time (hr)	0	2	4	6	8	10	12
UH (m^3/s/cm)	0	20	50	75	90	40	0

(a) Determine the 2-hr unit hydrograph.
(b) Suppose that a 10-year design storm has a total effective rainfall of 50 mm and the corresponding hyetograph has a rainfall of 30 mm in the first 2 hr period, no rain in the second period, and 20 mm in the third period. Assuming the baseflow is 10 m^3/s, calculate the total runoff hydrograph from this 10-year design storm.

8.3.7 Consider a drainage basin with an area of 151.2 km^2. From a storm event, the observed cumulative rainfall depth and the corresponding runoff hydrograph are given in the following table. Assume that the baseflow is 10 m^3/s.

Time (hr)	0	2	4	6	10	12	14	16	18	20	22	24	26
Cumulative rainfall (cm)	0.0	0.5	1.5	2.5	4.5	6.5	8.0	9.5					
Instantaneous discharge (m^3/s)	10	10	30	50	130	175	260	240	220	145	70	40	10

(a) Determine the value of the ϕ index and the corresponding effective rainfall hyetograph.
(b) Set up the system of equations, $Pu = Q$, for deriving the 6-hr unit hydrograph and define the elements in P, u, and Q. Solve for the unit hydrograph.

8.3.8 Suppose the 30-min unit hydrograph for a watershed is

Time (hr)	0	3	6	9	12	15	18	21
UH (m^3/s/cm)	0	15	45	65	50	25	10	0

(a) Determine the 1-hr unit hydrograph.
(b) A storm has a total depth of 4 cm with an effective rainfall hyetograph of 2 cm in the first 2 hr, 1 cm in the second 2 hr. Assuming the baseflow is 10 m^3/s, calculate the total runoff hydrograph from the storm.

8.3.9 The following table lists a 15-min unit hydrograph.

Time (min)	0	5	10	15	20	25	30	35	40	45
15 min UH (m^3/s/cm)	0.0	1.5	3.5	6.0	5.5	5.0	3.0	2.0	0.5	0.0

(a) Determine the corresponding area of the drainage basin.
(b) Determine the 10-min unit hydrograph.
(c) Determine the total runoff hydrograph resulting from the following effective rainfall, assuming the base flow is 2 m^3/s.

Time (min)	0	10	20	30
Cumul. rainfall excess (mm)	0	10	30	35

8.3.10 Consider there are two rain gauges (A and B) in a watershed with an area of 21.6 km^2. The cumulative rainfall depths over time for a particular storm event at both rain gauge stations are given in the table below. The storm also produces a runoff hydrograph at the outlet of the watershed, shown in the last row of the table. Assume that the baseflow is 10 m^3/s. It is known, by the Thiessen polygon, that the contributing areas for the rain gauges are identical.

Time (hr)	0	1	2	3	4	5	6	7	8	9	10
Cumul. Rainfall at Station A (mm)	0	25	80	105	100	140	180	230	230		
Cumul. rainfall at Station B (mm)	0	15	40	85	130	180	210	230	230		
Instantaneous discharge (m^3/s)	10	10	30	60	30	40	80	40	30	10	10

(a) Determine the basin-wide representative rainfall hyetograph (in mm/hr) for the storm event. Furthermore, determine the value of ϕ index (in mm/hr) and the corresponding effective rainfall hyetograph (in mm/hr).
(b) Set up the system of equations $Pu = Q$.
(c) Solve part (b) *by the least-squares method* for the unit hydrograph. What is the duration of the unit hydrograph obtained?

8.3.11 Consider a drainage basin with an area of 167.95 km^2. From a storm event, the observed cumulative rainfall depth and the corresponding runoff hydrograph are given in the following table. Assume that the baseflow is 10 m^3/s.

Time (hr)	0	2	4	6	8	10	12	14	16	18	20	22	24
Cumulative rainfall (cm)	0.0	0.5	1.5	2.5	4.5	6.5	8.0	9.5					
Instantaneous discharge (m^3/s)	10	10	30	50	130	175	260	240	220	145	70	40	10

(a) Determine the value of the ϕ index and the corresponding effective rainfall hyetograph.
(b) Set up the system of equations, $Pu = Q$, for deriving the 4-hr unit hydrograph and define the elements in P, u, and Q.
(c) Solve for the unit hydrograph ordinates.

8.3.12 Consider a drainage basin with an area of 216 km^2. From a storm event, the observed cumulative rainfall depth and the corresponding runoff hydrograph are given in the following table. Assume that the baseflow is 20 m^3/s.

Time (hr)	0	6	12	18	24	30
Cumulative rainfall (cm)	0.0	1.5	4.5	9.5		
Discharge (m^3/s)	20	50	160	340	180	20

(a) Determine the value of the ϕ index and the corresponding effective rainfall hyetograph.
(b) Determine the 6-hr unit hydrograph by the least-squares method.

8.3.13 On a particular date, the measured rainfall and runoff from a drainage basin of 54 ha (54×10^4 m^2) are listed in the table below.
(a) Assuming that the base flow is 2 m^3/s, determine the effective rainfall hyetograph by the ϕ index method.
(b) Based on the effective rainfall hyetograph and direct runoff hydrograph obtained in Part (a), use the least-squares method to determine the 15-min unit hydrograph.

Time (min)	0	15	30	45	60	75
Cumul. rain (mm)	0	5	20	40	50	
Instant. flow (m^3/s)	2	6	10	7	3	2

8.3.14 You have been given information on an actual rainfall-runoff event and asked to develop the runoff hydrograph for another storm hydrograph for the watershed. The following is known:

Time (hr)	Rainfall (in)	Discharge (cfs)
0		10
1	1.1	20
2	2.1	130
3	0	410
4	1.1	570
5		510
6		460
7		260
8		110
9		60
10		10

A constant base flow of 10 cfs and a uniform rainfall loss of 0.1 in/hr are applicable. Determine the size of the drainage basin. Then determine the direct runoff hydrograph for a 2.0-in excess precipitation for the first hour, no rainfall for the second hour, followed by a 2.0-in excess precipitation for the third hour.

8.4.1 A watershed has a drainage area of 14 km^2; the length of the main stream is 7.16 km, and the main channel length from the watershed outlet to the point opposite the center of gravity of the watershed is 3.22 km. Use $C_t = 2.0$ and $C_p = 0.625$ to determine the standard synthetic unit hydrograph for the watershed. What is the standard duration? Use Snyder's method to determine the 30-min unit hydrograph for the watershed.

8.4.2 Watershed A has a 2-hr unit hydrograph with $Q_{pR} = 276$ m^3/s, $t_{pR} = 6$ hr, $W_{50} = 4.0$ hr, and $W_{75} = 2$ hr. The watershed area = 259 km^2, $L_c = 16.1$ km, and $L = 38.6$ km. Watershed B is assumed to be hydrologically similar with an area of 181 km^2, $L = 25.1$ km, and $L_c = 15.1$ km. Determine the 1-hr synthetic unit hydrograph for watershed B. Determine the direct runoff hydrograph for a 2-hr storm that has 1.5 cm of excess rainfall the first hour and 2.5 cm of excess rainfall the second hour.

8.4.3 A watershed has an area of 39.3 mi^2 and a main channel length of 18.1 mi, and the main channel length from the watershed outlet to the point opposite the centroid of the watershed is 6.0 mi. The regional parameters are $C_t = 2.0$ and $C_p = 0.6$. Compute the T_P, Q_P, W_{50}, W_{75}, and T_B for Snyder's standard synthetic unit hydrograph and the same information for a 3-hr Snyder's synthetic unit hydrograph. Also, what is the duration of the standard synthetic unit hydrograph?

8.4.4 Compute the 3-hr Snyder's synthetic unit hydrograph for the watershed in problem 8.4.3.

8.4.5 You are performing a hydrologic study (rainfall-runoff analysis) for a watershed Z. Unfortunately, there is no gauged data or other hydrologic studies so you do not have a unit hydrograph. However, you do have data for another nearby watershed, known as watershed X, which has gauged information (including the discharge hydrograph for a known rainfall event). What procedure would you use to develop a design runoff hydrograph for a design storm for watershed Z? What are you assuming about the watersheds in this procedure?

8.4.6 You have determined the following from the basin map of a given watershed: $L = 100$ km, $L_c = 50$ km, and drainage area $=$ 2000 km^2. From the unit hydrograph developed for the watershed, the following were determined: $t_R = 6$ hr, $t_{pR} = 15$ hr, and the peak discharge $= 80$ km^3/s/cm. Determine the regional parameters used in the Snyder's synthetic unit hydrograph procedure.

8.4.7 Derive a 3-hr unit hydrograph by the Snyder method for a watershed of 54 km^2 area. It has a main stream that is 10-km long. The distance measured from the watershed outlet to a point on the stream nearest to the centroid of the watershed is 3.75 km. Take $C_t = 2.0$ and $C_p = 0.65$. Sketch the 3-hr unit hydrograph for the watershed.

8.4.8 Derive a 2-hr unit hydrograph by the Snyder method for a watershed of 50 km^2 area. It has a main stream that is 8 km long. The distance measured from the watershed outlet to a point on the stream nearest to the centroid of the watershed is 4 km. Take $C_t = 2.0$ and $C_p = 0.65$. Graph the 2-hr unit hydrograph for the watershed.

8.4.9 Use the Clark unit hydrograph procedure to compute the 1-hr unit hydrograph for a watershed that has an area of 5.0 km^2 and a time of concentration of 5.5 hr. The Clark storage coefficient is estimated to be 2.5 hr. Use a 1-hr time interval for the computations. Use the HEC U.S. Army Corps of Engineer synthetic time-area relationship.

8.4.10 Compute the Clark unit hydrograph parameters (T_c and R) for a 2.17-mi^2 (1389 acre) urban watershed in Phoenix, Arizona that has a flow path of 1.85 mi, a slope of 30.5 ft/mi, and an imperviousness of 21 percent. A rainfall intensity of 2.56 in/hr is to be used. The time of concentration for the watershed is computed using $T_c = 11.4L^{0.50} K_b^{0.52} S^{-0.31} i^{-0.38}$, where T_c is the time of concentration in hours, L is the length of the longest flow path in miles, K_b is a watershed resistance coefficient ($K_b = -0.00625\log A + 0.04$) for commercial and residential areas, A is the watershed area in acres, S is the slope of the flow path in ft/mi, and i is the rainfall intensity in in/hr. The storage coefficient is $R = 0.37T_c^{1.11} A^{-0.57} L^{0.80}$, where A is the area in mi^2.

8.4.11 Compute the 15-min Clark unit hydrograph for the Phoenix watershed in problem 8.4.10.

8.4.12 For the situation in problem 8.4.10, compute the time of concentration. However, now the rainfall intensity is not given, instead use the rainfall intensity duration frequency relation in Figure 7.2.15, with a 25-year return period.

8.4.13 Develop the 15-min Clark unit hydrograph for a 2.17-mi^2 (1389 acre) rural watershed that has a flow path of 1.85 mi, a slope of 30.5 ft/mi, and an imperviousness of 21 percent. A rainfall intensity of 2.56 in/hr is to be used. The time of concentration for the watershed is computed using $T_c = 11.4L^{0.50} K_b^{0.52} S^{-0.31} i^{-0.38}$, where T_c is the time of concentration in hours, L is the length of the longest flow path in miles, K_b is a watershed resistance coefficient ($K_b = -0.01375\log A + 0.08$), A is the watershed area in acres, and S is the slope of the flow path in ft/mi, and i is the rainfall intensity in in/hr.

8.5.1 Using the 4-hr unit hydrograph developed in problem 8.3.1, use the S-curve method to develop the 8-hr unit hydrograph for this 200 km^2 watershed.

8.5.2 Using the one-hour unit hydrograph given in problem 8.3.3, develop the 3-hr unit hydrograph using the S-curve method.

8.5.3 Suppose the 4-hr unit hydrograph for a watershed is

Time (hr)	0	2	4	6	8	10	12	14
UH (m^3/s/10 mm)	0	15	45	65	50	25	10	0

(a) Determine the 2-hr unit hydrograph.
(b) From the frequency analysis, the 10-year 4-hr storm has a total depth of 3 cm with an effective rainfall intensity of 1 cm/hr in the first 2 hr and 0.5 cm/hr in the second 2 hr. Assuming the baseflow is 10 m^3/s, calculate the total runoff hydrograph from the 10-year 4-hr storm.

8.5.4 Suppose the 4-hr unit hydrograph for the watershed is

Time (hr)	0	2	4	6	8	10	12	14	16
UH (m^3/s/cm)	0	19	38	32	22	13	6	2	0

(a) What is the area of the watershed?
(b) Determine the 2-hr unit hydrograph.
(c) Suppose that a 25-year design storm has a total effective rainfall of 7 cm and the corresponding hyetograph has 5 cm in the first 2 hr and 2 cm in the second 2 hr. Assuming the baseflow is 10 m^3/s, calculate the total runoff hydrograph from this 25-year design storm.

8.5.5 Suppose the 6-hr unit hydrograph for the watershed is

Time (hr)	0	3	6	9	12	15	18	21
UH (m^3/s/cm)	0	15	45	65	50	25	10	0

(a) What is the area of the watershed?
(b) Determine the 3-hr unit hydrograph.
(c) Suppose that a 10-year design storm has a total effective rainfall of 6 cm and the corresponding hyetograph has 4 cm in the first 3 hr and 2 cm in the second 3 hr. Assuming the baseflow is 10 m^3/s, calculate the total runoff hydrograph from this 10-year design storm.

8.5.6 Suppose the 6-hr unit hydrograph for a watershed is

Time (hr)	0	3	6	9	12	15	18
UH ($m^3/s/cm$)	0	60	90	50	30	10	0

(a) Determine the 3-hr unit hydrograph.
(b) Suppose that a 50-year design storm has a total effective rainfall of 9 cm and the corresponding hyetograph has 2 cm in the first 3 hr, 5 cm in the second 3 hr, and 2 cm in the third 3 hr. Assuming the baseflow is 20 m^3/s, determine the total runoff hydrograph from this 50-year design storm.

8.5.7 Suppose the 6-hr unit hydrograph for a watershed is

Time (hr)	0	3	6	9	12	15	18	21
UH ($m^3/s/cm$)	0	15	45	65	50	25	10	0

(a) Determine the 2-hr unit hydrograph.
(b) Assuming the baseflow is 10 m^3/s, calculate the total runoff hydrograph from an effective rainfall hyetograph of 2 cm in the first 2 hr and 1 cm in the second 2 hr.

8.7.1 Determine the weighted curve numbers for a watershed with 60 percent residential (1/4-acre lots), 20 percent open space, good condition, and 20 percent commercial and business (85 percent impervious) with corresponding soil groups of C, D, and C.

8.7.2 Rework example 8.7.1 with corresponding soil groups of B, C, D, and B.

8.7.3 The watershed in problem 8.7.1 experienced a rainfall of 5 in; what is the runoff volume per unit area?

8.7.4 Rework example 8.7.2 with a 7-in rainfall.

8.7.5 Calculate the cumulative abstractions and the excess rainfall hyetograph for the situation in problems 8.7.1 and 8.7.3. The rainfall pattern is 1.5 in during the first hour, 2.5 in during the second hour, and 1.0 in during the third hour.

8.7.6 Calculate the cumulative abstraction and the excess rainfall hyetograph for the situation in problems 8.7.2 and 8.7.4. The rainfall pattern is 2.0 in during the first hour, 3.0 in during the second hour, and 2.0 in during the third hour.

8.7.7 Consider an urban drainage basin having 60 percent soil group B and 40 percent soil group C. The land use pattern is 1/2 commercial area and 1/2 industrial district. Determine the rainfall excess *intensity* hyetograph under the dry antecedent moisture condition (AMC I) from the recorded storm given in problem 8.2.6.

8.7.8 Consider a drainage basin having 60 percent soil group B and 40 percent soil group C. Five years ago, the watershed land use pattern was 1/2 wooded area with good cover and 1/2 pasture with good condition. Now, the land use has been changed to 1/3 wooded area, 1/3 pasture land, and 1/3 residential area (1/4-acre lot). Estimate the volume of increased runoff due to the land use change over the past 5-year period for a storm with 6 in of rainfall under the dry antecedent moisture condition (AMC I).

8.7.9 Refer to problem 8.7.8 for the present watershed land use pattern. Determine the effective rainfall hyetograph for the following storm event using the SCS method under the dry antecedent moisture condition (AMC I). Next determine the value of the ϕ index corresponding to the effective rainfall hyetograph.

Time (h)	0–0.5	0.5–1.0	1.0–1.5	1.5–2.0
Rainfall intensity (in/h)	6.0	3.0	2.0	1.0

8.7.10 Consider a drainage basin having 60 percent soil group A and 40 percent soil group B. Five years ago, the land use pattern in the basin was 1/2 wooded area with poor cover and 1/2 cultivated land with good conservation treatment. Now, the land use has been changed to 1/3 wooded area, 1/3 cultivated land, and 1/3 commercial and business area. Estimate the increased runoff volume during the dormant season due to the land use change over the past 5-year period for a storm of 350 mm in total depth. This storm depth corresponds to a duration of 6-hr and 100-year return period. The total 5-day antecedent rainfall amount is 30 mm. Note: 1 inch = 25.4 mm.

8.7.11 Consider a drainage basin of 500-ha having hydrologic soil group D. In 1970, the watershed land use pattern was 50% wooded area with good cover, 25% range land with good condition, and 25% residential area (1/4-acre lot). Ten years later, the land use has been changed to 30% wooded area, 20% pasture land, 40% residential area (1/4-acre lot), and 10% commercial and business area.

(a) Compute the weighted curve numbers in 1970 and 1980.
(b) Using the SCS method, estimate the percentage of change (increase or decrease) in runoff volume with respect to year 1970 due to the land use change over the 10-year period for a 2-hr, 50-mm rainfall event under the wet antecedent moisture condition.
(c) Suppose that the 2-hr, 50-mm rainfall event has the following hyetograph. Determine the rainfall excess intensity hyetograph (in mm/hr) and the corresponding incremental infiltration (in mm) over the storm duration under 1980 conditions.

Time (hr)	0–0.5	0.5–1.0	1.0–1.5	1.5–2.0
Rainfall intensity (mm/hr)	4.0	50.0	30.0	16.0

8.7.12 Consider a drainage basin of 36 km^2 having hydrologic soil group B. In 1970, the watershed land use pattern was 60 percent wooded area with good cover, 25 percent range land with good condition, and 15 percent residential area (1/4-acre lot). Twenty years later (i.e., in 1990), the land use has been changed to 30 percent wooded area with good cover, 20 percent pasture land with good condition, 40 percent residential area (1/4-acre lot), and 10 percent commercial and business area.

On a particular day in 1990, there was a rainstorm event that produced rainfall and runoff recorded in the following table.

Time (min)	0	15	30	45	60	75	90	105	120
Cumulative rainfall (mm)	0	10	50	75	90	100			
Instantaneous runoff (m^3/s)	10	30	160	360	405	305	125	35	10

(a) Using the SCS method, estimate the percentage of change (increase or decrease) in runoff volume in 1990 with respect to that of 1970 for the above rainstorm event under the normal antecedent moisture condition.
(b) Under the wet antecedent moisture condition, determine the rainfall excess intensity hyetograph (in mm/hr) and the corresponding incremental infiltration (in mm) over the storm duration for the above rainstorm event under the 1990 conditions.
(c) For a baseflow of 10 m^3/s, determine the volume of direct runoff.
(d) Assuming an initial loss of 10 mm, determine the Φ index (in mm/hr) and the corresponding excess rainfall hyetograph.

8.7.13 Consider a drainage basin with an area of 200 km^2. From a storm event, the observed cumulative rainfall depth and the corresponding runoff hydrograph are given in the following table. Assume that the baseflow is 20 m^3/s.

Time (hr)	0	4	8	12	16	20
Cum. rainfall (cm)	0.0	1.6	5.5	7.5	7.5	7.5
Discharge (m^3/s)	20	40	130	300	155	20

(a) Determine the effective rainfall hyetograph by the SCS method with a curve number $CN = 85$.
(b) Determine the 4-hr unit hydrograph by the least-squares method.

8.7.14 During a rain storm event, rain gauge at location X broke down and rainfall record was not available. Fortunately, three rain gauges nearby did not have a technical problem and their rainfall readings, along with other information, are provided in the table below.

Rain gauge	A	B	C	X
Event depth (mm)	40	60	50	Missing
Distance to gauge X (km)	10	8	12	0
Elevation (m)	20	50	40	60
Mean annual rainfall (mm)	1500	2000	1800	2200
Polygon area (km^2)	10	30	20	40

The four rain gauges are located either within or in the neighborhood of a watershed having a drainage area of 100 km^2. According to the Thiessen polygon method, the contributing area of each rain gauge is shown in the above table.

For this particular storm event, the duration of the storm is 3 hr and the percentage of rainfall depth in the first hour is 50 percent, in the second hour 30 percent, and in the third hour 20 percent. The watershed largely consists of woods and meadow of good hydrologic condition with soil group B. Among the two land cover types, woods occupy 60 percent of the total area while the meadow makes up the remaining 40 percent.

For this watershed, the 1-hr unit hydrograph resulting from 1 cm of effective rainfall has been derived and is given below:

Time (hr)	0	1	2	3	4	5
Flow rate (m^3/s/cm)	0	10	30	20	10	0

(a) Select a method to estimate missing rainfall depth at station X by an appropriate method. Explain the reasons why the method is selected.
(b) Determine the basin-wide equivalent uniform rainfall depth and the corresponding hyetograph for this particular storm.
(c) According to part (b), determine the total effective rainfall depth and the corresponding rainfall excess hyetograph for the storm. It is known that a storm occurred 2 days before this particular storm and, therefore, the ground is quite wet.
(d) What is the magnitude of peak runoff discharge produced by this particular storm? It is reasonable to assume that the baseflow is 5 m^3/s.

8.7.15 Consider a drainage basin having 60 percent soil group A and 40 percent soil group B. Five years ago, the land use pattern in the basin was 1/2 wooded area with poor cover and 1/2 cultivated land with good conservation treatment. Now the land use has been changed to 1/3 wooded area, 1/3 cultivated land, and 1/3 commercial and business area.

(a) Estimate the increased runoff volume during the dormant season due to the land use change over the past 5-year period for a storm of 35 cm total depth under the dry antecedent moisture condition (AMC I). This storm depth corresponds to a duration of 6-hr and 100-year return period. The total 5-day antecedent rainfall amount is 30 mm. (Note: 1 in = 25.4 mm.)
(b) Under the present watershed land use pattern, find the effective rainfall hyetograph (in cm/hr) for the following storm event using SCS method under the dry antecedent moisture condition (AMC I).

Time (hr)	0–0.5	0.5–1.0	1.0–1.5	1.5–2.0
Avg. rainfall intensity (cm/hr)	16.0	9.0	5.0	3.0

8.7.16 Consider a drainage basin having 60 percent soil group B and 40 percent soil group C. Five years ago, the watershed land use pattern was 1/2 wooded area with good cover and 1/2 pasture with good condition. Today, the wooded area and pasture each have been reduced down to 1/3 and the remaining 1/3 of the drainage basin has become a residential area (1/4-acre lot). Consider the rainstorm event having the observed rainfall mass data given below.

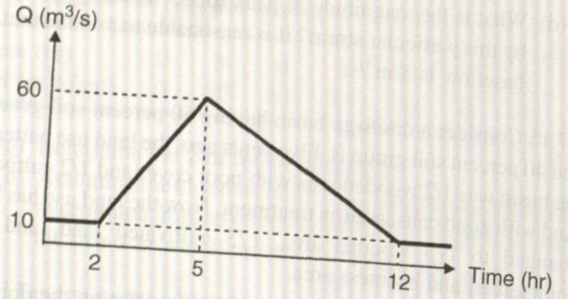
Time (min)	0	30	60	90	120
Cumulative Rainfall (mm)	0	152	228	278	304

(a) Estimate the increased total runoff volume due to the land use change over the past 5-year period for the above rainstorm event under the dry antecedent moisture condition (AMC I).

(b) Referring to the present watershed land use condition, find the effective rainfall hyetograph (in mm/hr) for the rainstorm event given the above using the SCS method under the dry antecedent moisture condition (AMC I).

8.7.17 Consider a drainage basin of 1,200 hectares having 60 percent soil group B and 40 percent soil group C. Five years ago, the watershed land use pattern was 1/2 wooded area with good cover and 1/2 pasture with good condition. Now the land use has been changed to 1/3 wooded area, 1/3 pasture land, and 1/3 residential area (1/4-acre lot). For a storm event, the cumulative rainfall and the corresponding runoff hydrograph are shown in the following table and figure, respectively. (Note: 1 hectare = 10,000 m^2; 1 in = 25.4 mm).

Time (hr)	0	1	2	3	4
Cumulative rainfall (mm)	0	60	120	160	180



(a) Estimate the increased total runoff volume by the SCS method due to the land use change over the past 5-year period for this storm under the wet antecedent moisture condition.

(b) Find the effective rainfall intensity hyetograph for the storm event using the SCS method under the wet antecedent moisture condition and current land use condition.

(c) Find the value of the ϕ index corresponding to the above direct runoff hydrograph assuming the baseflow is 10 m^3/s.

8.7.18 Consider a drainage basin having soil group D. The land use pattern in the basin is 1/3 wooded area with good cover, 1/3 cultivated land with conservation treatment, and 1/3 commercial and business area. For the rainstorm event given below, use the SCS method under the dry antecedent moisture condition (AMC I).

(a) Estimate the total volume of effective rainfall (in cm).

(b) Find the effective rainfall hyetograph (in cm/hr) for the following storm event.

Time (hr)	0–0.5	0.5–1.0	1.0–1.5	1.5–2.0
Avg. rainfall intensity (cm/hr)	16.0	9.0	5.0	3.0

8.8.1 Develop the SCS triangular unit hydrograph for a 400-acre watershed that has been commercially developed. The flow length is 1,500 ft, the slope is 3 percent, and the soil group is group B.

8.8.2 Prior to development of the 400-acre watershed in problem 8.8.1, the land use was contoured pasture land with fair condition. Compute the SCS triangular unit hydrograph and compare with the one for commercially developed conditions.

8.8.3 Using the watershed defined in problems 8.8.1 and 8.8.2, determine the SCS triangular unit hydrograph assuming residential lot size. Compare with the results in problem 8.8.2.

8.8.4 A 20.7-km^2 watershed has a time of concentration of 1.0 hr. Calculate the 10-min unit hydrograph for the watershed using the SCS triangular unit hydrograph method. Determine the direct runoff hydrograph for a 30-min storm having 1.5 cm of excess rainfall in the first 10 min, 0.5 cm in the second 10 min, and 1.0 cm in the third 10 min.

8.9.1 Develop a flowchart of the kinematic overland flow runoff model described in Section 8.9.

8.9.2 Develop the appropriate equations to solve equation (8.9.9) by Newton's method.

8.9.3 Derive equation (8.9.12).

REFERENCES

Chow, V. T. (editor), *Handbook of Applied Hydrology*, McGraw-Hill, New York, 1964.

Chow, V. T., D. R. Maidment, and L. W. Mays, *Applied Hydrology*, McGraw-Hill, New York, 1988.

Clark, C. O., "Storage and the Unit Hydrograph," *Trans. American Society of Civil Engineers*, Vol. 110, pp. 1419–1488, 1945.

Engman, E. T., "Roughness Coefficients for Routing Surface Runoff", *Journal of Irrigation and Drainage Engineering;* American Society of Civil Engineers, 112(1), pp. 39–53, 1986.

Flood Control District of Maricopa County, *Drainage Design Manual for Maricopa County, Arizona*, Phoenix, AZ, 1995.

Ford, D., E. C. Morris, and A. D. Feldman, "Corps of Engineers' Experience with Automatic Calibration Precipitation-Runoff Model," in *Water and Related Land Resource Systems* edited by Y. Haimes and J. Kindler, p. 467–476, Pergamon Press, New York, 1980.

Hewlett, J. D., and W. L. Nutter, *An Outline of Forest Hydrology*, University of Georgia Press, Athens, GA, 1969.

Horton, R. E., "Erosional Development of Streams and Their Drainage Basins; Hydrological Approach to Quantitative Morphology," *Bull. Geol. Soc. Am.*, vol. 56, pp. 275–370, 1945.

Masch, F. D., *Hydrology*, Hydraulic Engineering Circular No. 19, FHWA-10-84-15, Federal Highway Administration, U.S. Department of the Interior, McLean, VA, 1984.

Morris, E. M., and D. A. Woolhiser, "Unsteady One-Dimensional Flow over a Plane: Partial Equilibrium and Recession Hydrographs," *Water Resources Research*, vol. 16, no. 2, pp. 355–360, 1980.

Mosley, M. P., and A. I. McKerchar, "Streamflow," in *Handbook of Hydrology* (edited by D. R. Maidment), McGraw-Hill, New York, 1993.

Palmer, V. J., "Retardance Coefficients for Low Flow in Channels Lined with Vegetation," *Transactions of the American Geophysical Union*, 27(11), pp. 187–197, 1946.

Papadakis, C. N., and M. N. Kazan, "Time of Concentration in Small, Rural Watersheds," *Proceedings of the Engineering Hydrology Symposium*, ASCE, Williamsburg, Virginia, pp. 633–638, 1987.

Ponce, V. M., *Engineering Hydrology; Principles and Practices*, Prentice-Hall, Englewood Cliffs, NJ, 1989.

Sanders, T. G. (editor), *Hydrology for Transportation Engineers*, U.S. Dept. of Transportation, Federal Highway Administration, 1980.

Strahler, A. N., "Quantitative Geomorphology of Drainage Basins and Channel Networks," section 4-II in *Handbook of Applied Hydrology* (edited by V. T. Chow), McGraw-Hill, New York, 1964.

Straub, T. D., C. S. Melching, and K. E. Kocher, *Equations for Estimating Clark Unit-Hydrograph Parameters for Small Rural Watersheds in Illinois*, U.S. Geological
Report 00-4184, Urbana, IL, 20(

U.S. Army Corps of Engineer:
Flood Hydrograph Package, Us

U.S. Department of Agricul
Engineering Handbook, Sectior
ernment Printing Office, Washi

U.S. Department of Agricu
Hydrology for Small Watersh
DC, June, 1986.

Woolhiser, D. A., and J. A. Liggett, "Unsteady, One-Dimensional Flow over a Plane—the Rising Hydrograph," *Water Resources Research*, vol. 3(3), pp. 753–771, 1967.

Woolhiser, D. A., R. E. Smith, and D. C. Goodrich, *KINEROS, A Kinematic Runoff and Erosion Model: Documentation and User Manual*, U. S. Department of Agricultural Research Service, ARS-77, Tucson, AZ, 1990.