5 Infiltration and Runoff

Infiltration and runoff are two important processes in the hydrologic cycle (Figure 1.1). Infiltration begins when precipitation reaches the land surface. Runoff begins when the precipitation rate exceeds the infiltration rate, and retention and surface storage are filled. The relationship between rainfall, infiltration, and runoff is illustrated in Figure 5.1.

Infiltration is the main source of water for vegetative growth and crop production, provides input to groundwater recharge, and transports water-soluble compounds, such as fertilizers, manures, herbicides, and other materials, from the land surface into the soil. Some infiltrated water eventually recharges the groundwater. A large fraction of infiltrated water returns to the atmosphere by evapotranspiration. A small fraction of infiltrated water may reappear as surface water and either runoff or infiltrate again.

Surface runoff discharges into channels, streams, rivers, lakes, or other surface water reservoirs. Aquatic life and a large portion of the human population depend on surface water. The quality and quantity of surface water largely depend on runoff quality

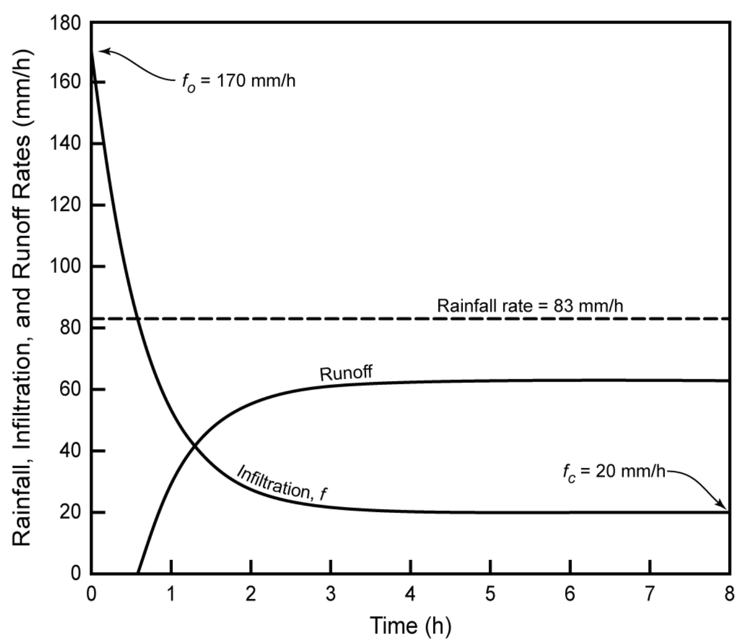

Figure 5.1–Illustration of infiltration and runoff curves for a constant rainfall rate.

and quantity. High runoff rates and volumes can cause soil erosion and flooding, damage or destroy structures, and destroy human and animal lives.

Infiltration

The term infiltration is the process of water entry into the soil. The rate at which water infiltrates into the soil is the infiltration rate, which has the dimensions of volume per unit of time per unit of area, which reduces to depth per unit of time. Infiltration rate should not be confused with hydraulic conductivity, which is the ratio of soil-water flow rate (flux) to the hydraulic gradient. After water enters the soil, it moves within the soil by a process known as percolation.

Infiltration may be limited by restrictions that often occur at the soil surface or at lower layers of the profile. The major factors influencing the rate of infiltration are the physical characteristics of the soil and the cover on the soil surface, but other factors such as soil water content, temperature, and rainfall intensity are also important.

5.1 Saturated Hydraulic Conductivity—Darcy Equation

The saturated hydraulic conductivity of the soil may limit the infiltration rate of a soil. The one-dimensional flow of water through a saturated soil can be computed from the Darcy equation,

$$q = A\ K\ \frac{dh}{ds} = A\ K\ \frac{\Delta h}{L} \tag{5.1}$$

where q = flux or flow rate (L^3/T),
A = cross-sectional area of the soil through which water is flowing (L^2),
K = saturated hydraulic conductivity of the flow medium (L/T),
dh/ds = hydraulic gradient (L/L) in the direction of flow s,
Δh = total change in head or potential causing flow for a distance L (L).

Hydraulic conductivity is determined from field or laboratory measurements.

The flow path may be downward (as during infiltration), horizontal, or upward. The equation is valid so long as the velocity of flow and the size of soil particles are such that the Reynolds number is less than one. Hydraulic conductivity K is a function of the effective diameter of the soil pores and the density and dynamic viscosity of the fluid. It is the average velocity of bulk flow in response to a unit gradient.

Application of the Darcy equation is more difficult for two- and three-dimensional flow systems that have complex boundary conditions. Where water movement is through two soil layers, such as a topsoil layer and a subsoil layer, the composite vertical hydraulic conductivity K can be computed from

$$K = \frac{L}{\frac{L_1}{K_1} + \frac{L_2}{K_2}} \tag{5.2}$$

where L = the total length of flow through all layers (L), and subscripts 1 and 2 represent the soil layers 1 and 2, respectively. Another L/K term should be added for each additional layer. Darcy's law is analogous to Ohm's law for electrical current and to Fourier's law for heat conduction.

Example 5.1

Calculate the composite hydraulic conductivity of a 0.9 m soil profile having three soil layers. The top soil layer is 0.3 m thick and has a vertical saturated hydraulic conductivity of 15 mm/h, and that of the middle layer (0.2 m thick) and bottom layer (0.4 m thick) are 2.0 and 8.0 mm/h, respectively.

Solution. For three soil layers with the thicknesses in mm, use Equation 5.2 in the form:

$$K = \frac{L}{\frac{L_1}{K_1} + \frac{L_2}{K_2} + \frac{L_3}{K_3}} \quad = \; K = \frac{900}{\frac{300}{15} + \frac{200}{2.0} + \frac{400}{8.0}} \quad = 5.3 \text{ mm/h}$$

Note that the soil layer with the lowest conductivity has a major influence on the composite conductivity.

5.2 Soil Factors

The soil is a pervious medium that contains a large number of micropores. The ease with which water moves through the soil largely depends on the size and permanence of the micropores. The size of the micropores and the infiltration into the soil depend on (1) soil texture, (2) the degree of aggregation between the individual particles, and (3) the arrangement of the particles and aggregates. In general, larger pore sizes and greater continuity of the pores result in higher infiltration rates.

Large openings called soil macropores may be present in the soil. The macropores are mostly inter-aggregate cavities; however, plant roots, wormholes, soil shrinking during drying, other natural phenomena, and soil tillage can also create these cavities. When macropores are present, soils tend to have higher infiltration rates, but water moving in macropores may pass through portions of the plant root zone and add little to water storage there.

The maintenance of a porous soil structure, particularly at the soil surface, is critical for infiltration. The infiltration rate is greatly reduced when the uppermost thin layer of the soil surface is relatively sealed or compacted (crusted). This can result from severe breakdown of soil structure caused by heavy equipment traffic, puddling, and other field operations. Surface sealing can also result from the beating action of raindrops and the sorting action of water flowing over the surface. The fine particles can fill the spaces between the large ones to form a relatively impervious seal.

The surface-sealing effect can be largely eliminated when the soil surface is protected by mulch, crop residue, or by some other permeable protection. The effective-

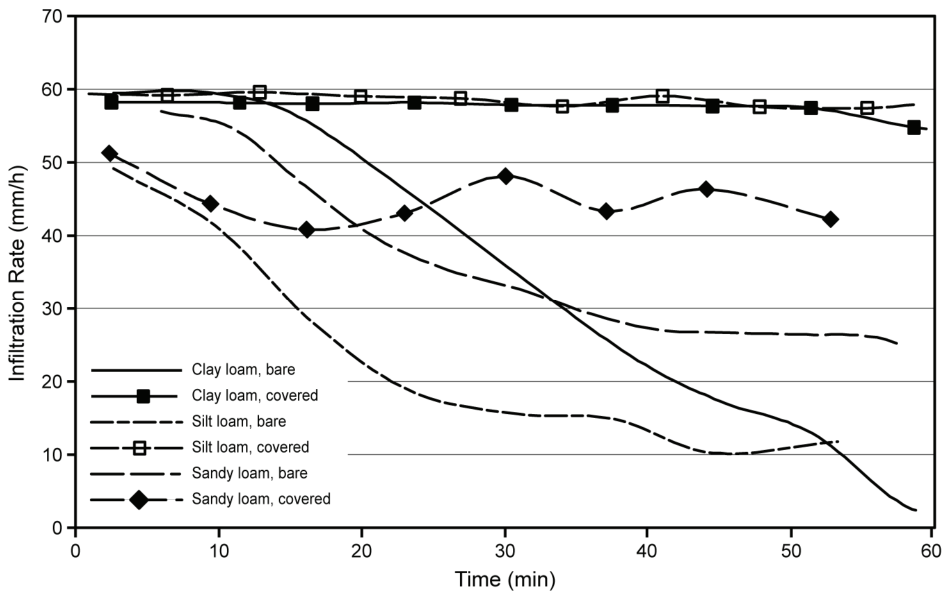

Figure 5.2–Effect of protective cover on infiltration rates. (Data provided by W. J. Elliot.)

ness of such protection is illustrated in Figure 5.2, which shows the measured infiltration rates for covered and uncovered clay loam, silt loam, and sandy loam soils. The soils protected by a cover maintained higher infiltration rates than the unprotected soils.

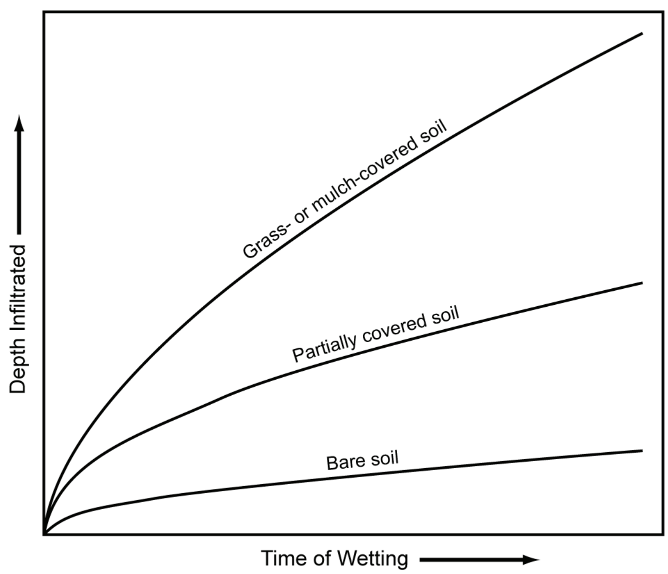

Figure 5.3–Illustration of infiltration depth curves for a soil with various surface covers.

5.3 Vegetation

Vegetation can greatly reduce surface sealing. In general, vegetative and surface conditions have more influence on infiltration rates than soil texture and structure. The protective cover may be grass, other close-growing vegetation, plant residue, and mulch. If the protection of a vegetative cover is lost, surface sealing may occur with drops in infiltration rates similar to those in Figure 5.2. Figure 5.3 illustrates typical infiltration depth curves for a given soil with different surface and vegetative conditions. Infiltration is higher for grass or mulched areas where the soil surface is protected than that for bare soil conditions. Other soils may have higher or lower depths of infiltration.

5.4 Soil Additives

Chemical additives (including fertilizers and manures) can change the physical characteristics of the soil, including the infiltration capacity. For example, polyacrylamide can ionically bond soils together to increase the aggregate size, which increases infiltration. In general, chemical additives are one of two types. The first type consists of materials that enhance the stability of the soil aggregates and improve the soil structure. Improved structure can considerably increase infiltration rates. The second type of additive is a wetting agent. It does not change the soil structure, but increases the wettability of the soil particles, which promotes faster water entry into the soil pores. It may be necessary to reapply wetting agents periodically as they leach out with continued water application.

Additives can also decrease infiltration rates. One group of chemical additives causes soil colloidal particles to swell. Swelling clays are sometimes added to soils. These swell and seal soil pores to reduce infiltration rates. Adding sodium salts will disperse the soil aggregates and reduce infiltration. Partial or complete sealants, such as petroleum or plastic films, are applied to soil surfaces to decrease or prevent infiltration. Decreased infiltration is desired to reduce seepage losses from reservoirs, waste storage structures, irrigation canals, or to increase runoff for surface water supplies.

5.5 Other Factors

Other factors affecting infiltration include land slope, antecedent soil water, air entrapment, surface roughness, and temperature (a special case is frozen soil). The effect of slope on infiltration rate is small, but more important on slopes less than 2% with high roughness. High soil water contents generally reduce or limit the infiltration rate. The soil matric potential decreases at high soil water content, reducing infiltration rates. The reduction is also because water causes some of the colloids in the soil to swell, reducing both the pore size and the rate of water movement. Entrapped air may remain in some of the soil pores and reduce both the infiltration rate and hydraulic conductivity. Field infiltration tests are customarily conducted to observe infiltration rates, once when the soil is dry and once when it is wet. The rate of change in infiltration values and the minimum infiltration values obtained from these tests may be used in a design process. In a completely saturated soil underlain by an impervious layer, infiltration is negligible.

The effect of water temperature on infiltration is not significant, perhaps because the soil changes the temperature of the entering water and the size of the pore spaces may change with temperature changes. Although freezing of the soil surface greatly reduces its infiltration capacity, freezing does not necessarily render the soil impervious. Near freezing the water viscosity increases and infiltration may be reduced.

5.6 Infiltration Curves

Infiltration data are commonly expressed as infiltration rate (Figure 5.1) or infiltration depth (Figure 5.3). In Figure 5.1 the initial infiltration rate exceeds the rate of water application; however, as the soil pores are filled with water, and as surface sealing occurs, the rate of water intake gradually decreases, asymptotically approaching a constant value that is known as final infiltration rate or steady-state infiltrability (Hillel, 1998). Figure 5.1 shows that when the initial infiltration rate exceeds the application rate, the actual infiltration rate is equal to the application rate. When the infiltration rate fell below the rainfall rate, water began to accumulate on the surface, creating the potential for runoff. Several empirical and theoretically-based equations have been developed and used for predicting the curves for infiltration depth or rate as a function of time (Hillel, 1998).

5.7 Kostiakov Equation

Kostiakov (1932) proposed the infiltration equation

$$F = k\,t^{a} \tag{5.3}$$

where F is the cumulative depth infiltrated at time t, and k and a are constants. The constants are obtained from measured infiltration data and have no particular physical meaning. This equation is only valid for short infiltration times because it does not account for the final infiltration rate. Additional terms have been added to alleviate this shortcoming (Chapter 16).

5.8 Horton Equation

The Horton (1940) equation is a common model for infiltration rate, which is expressed as

$$f = f_c + \left(f_0 - f_c\right) e^{-kt} \tag{5.4}$$

where f = infiltration rate (L/T) at time t (T),
f_c = steady-state infiltration rate at large times (L/T),
f_o = initial infiltration rate at time t = 0 (L/T),
k = constant for a given soil and initial condition (1/T).

The values of f_o, f_c, and k can be determined experimentally for any soil. The steady-state infiltration rate f_c can be approximated by the saturated hydraulic conductivity for a given soil. An example of Horton's equation is included in Figure 5.1 with f_c = 20 mm/h, f_o = 170 mm/h and k = 1.5/h.

Example 5.2

Use the Horton equation to estimate the infiltration rate of a soil after 20 min of infiltration. The initial infiltration rate of the soil is 75 mm/h, the steady-state infiltration rate is 10 mm/h and the constant k is 12.5 per hour.

Solution. In the Horton equation, time t = 20 min = 0.33 h, f_o = 75 mm/h, f_c = 10 mm/h, and $k = 12.5\ h^{-1}$. Substituting in Equation 5.1,

$$f = 10 + (75-10)\,e^{-(12.5)0.33} = 11.0 \text{ mm/h}$$

5.9 Philip Equation

Philip (1957) derived the following equation for infiltration into a uniform soil from a theoretical analysis of vertical, one-dimensional flow

$$F = St^{1/2} + At \tag{5.5}$$

where F is the depth (L) of infiltration in time t (T), S is a sorptivity term (L/T$^{1/2}$), and A is a conductivity term, (L/T).

5.10 Green-Ampt Equation

A widely used empirical equation for predicting infiltration was developed by Green and Ampt (1911) . The equation was developed by applying the Darcy equation (Equation 5.1) to the wetted soil zone and assuming vertical flow, uniform initial water content, and uniform soil hydraulic conductivity (near saturation). A working form of the Green-Ampt equation for cumulative infiltration depth is

$$F = K_e t + S_{avg} M \ln\left[1 + \frac{F}{S_{avg} M}\right] \tag{5.6}$$

where F = cumulative infiltration depth at time t (L),
K_e = effective hydraulic conductivity (L/T),
t = time (T),
S_{avg} = average matric suction at the wetting front (L),
M = fillable porosity (L^3/L^3).

Water is assumed to be freely available at the soil surface, but of negligible depth. The wetting front is modeled as an abrupt interface between saturated and unsaturated zones. The difference between initial and final water contents is the fillable porosity

Table 5.1 Hydrologic Soil Group Descriptions

Soil Group	Description	Basic Infiltration Rate, f_c (mm/h)
A	*Lowest Runoff Potential.* Includes deep sands with very little silt and clay, also deep rapidly permeable loess.	8 to 12
B	*Moderately Low Runoff Potential*. Mostly sandy soils less deep than A, and loess less deep or less aggregated than A, but the group as a whole has above average infiltration after thorough wetting.	4 to 8
C	*Moderately High Runoff Potential.* Comprises shallow soils and soils containing considerable clay and colloids, though less than those of group D. The group has below-average infiltration after presaturation.	1 to 4
D	*Highest Runoff Potential.* Includes mostly clays of high swelling percent, but the group also includes some shallow soils with nearly impermeable subhorizons near the surface.	0 to 1
Source: SCS (1972).		

$$M = \theta_f - \theta_i \tag{5.7}$$

where θ_f is the final soil water content (L^3/L^3) and θ_i the initial soil water content (L^3/L^3). The final water content is usually slightly less than saturation.

The parameters in the Green-Ampt model can be estimated from readily measured soil properties, using equations developed by Rawls and co-workers (Rawls and Brakensiek, 1989; Rawls et al., 1982). The DRAINMOD drainage model and the WEPP erosion prediction model both incorporate the Green-Ampt infiltration equation.

Because of the difficulty in evaluating infiltration, the SCS (1972) divided all soils into four hydrologic groups—A, B, C, and D—on the basis of infiltration rates. Table 5.1 gives hydrologic soil groups, descriptions, and their corresponding infiltration rates. The procedure for applying infiltration data to obtain runoff is discussed in the next section.

Runoff

Much of the water in ponds, lakes, and reservoirs comes from runoff. Quantity and quality of surface water, soil erosion, and stability of channels, stream banks, and other structures are affected by runoff volume and rate. Runoff constitutes the hydraulic "loading" that conservation structures or channels must withstand. The design of soil conservation structures, reservoirs, spillways, and channels must be based on runoff rate and/or volume. Therefore, understanding the runoff processes and their estimation techniques is important.

5.11 Definition

Runoff is that portion of the precipitation that flows overland toward stream channels, lakes, or oceans after the demands of interception, evapotranspiration, infiltration, surface storage, and surface detention are satisfied. In areas with significant land slopes, runoff may include near-surface flow that moves laterally beneath the soil surface and emerges at some point downhill to become surface runoff. This is often called *interflow* and is particularly important in forested watersheds where old root channels provide conduits. Appropriate knowledge of peak runoff rates, runoff volumes, and their spatial and temporal distributions is required for design and analysis.

5.12 The Runoff Process

Runoff can occur only when the rate of precipitation exceeds the soil infiltration rate after the demands for interception and surface storages are fulfilled. Interception by dense covers of forest or shrubs may be as much as 25% of the annual precipitation. A good stand of mature corn may have a net interception storage capacity of 0.5 mm per storm. Trees such as willows may intercept nearly 13 mm from a long, gentle storm. Interception also has a detention storage effect, delaying the progress of precipitation that reaches the soil surface only after running down the plant or dropping from the leaves. Runoff may not start even if the precipitation amount exceeds interception.

After the interception amount is met and the infiltration rate is exceeded, water begins to fill the depressions on the soil surface. This is called *depression storage*. After the depressions are filled, a thin static layer of *surface detention* water builds up on the soil surface, beyond which the water layer starts moving overland. After surface detention storage is satisfied, overland flow or runoff begins. The depth of the water layer continues to build up on the surface until the runoff rate is in equilibrium with the rate of precipitation less infiltration and interception. After precipitation ceases, the water in surface storage eventually infiltrates or evaporates.

Factors Affecting Runoff

Most of the factors that affect infiltration rate also affect runoff. The factors affecting runoff may be broadly categorized into precipitation characteristics or watershed characteristics. In addition, climatic variables (temperature, relative humidity, and wind speed and direction) have some effect on runoff.

5.13 Precipitation Characteristics

Rainfall amount, duration, intensity, and distribution pattern (Chapter 3) influence the rate and volume of runoff. Total runoff for a storm event is clearly related to the rainfall duration and intensity. Infiltration is high in the initial stages of a storm and decreases with time. Thus, a storm of short duration may produce no runoff, whereas a storm of the same intensity, but of longer duration, can produce runoff.

Rainfall intensity influences both the rate and the volume of runoff. An intense storm exceeds the infiltration rate by a greater margin than does a gentle rain; thus, the volume of runoff is greater for the intense storm even though the precipitation totals for the two rainfall events are the same. The intense storm may actually decrease the infiltration rate because of its destructive action on the soil structure at the surface.

Rate and volume of runoff from a watershed are influenced by the spatial distribution of rainfall amount and intensity over the watershed. Generally the maximum runoff rate will occur when the entire watershed contributes to runoff from a uniform rainfall; however, an intense storm on a portion of the watershed could result in a greater runoff rate than a moderate storm over the entire watershed.

Frozen soil conditions significantly impact surface runoff. Winter flooding occurs frequently on frozen lands east of the Cascade Mountains in the Pacific Northwest and in the northern parts of the Intermountain West. Rainfall intensities in these areas during winter are relatively low and runoff events usually occur from November through March (McCool et al., 2000) while the soil is frozen. During this period, rainfall on snow accompanied by high wind speeds and warm, moist Pacific air masses provide high dew point temperatures and accelerate the snowmelt rate. Since the infiltration rate of frozen soil is almost zero, almost all of the rain and snowmelt contributes to surface runoff.

5.14 Watershed Characteristics

Watershed factors affecting runoff are (a) size, shape, and orientation; (b) topography; (c) soil type; and (d) land use and land management conditions. Both runoff volumes and rates increase as watershed size increases; however, both rate and volume per unit of watershed area decrease as the runoff area increases. Watershed size may determine the season at which high runoff may be expected to occur. On watersheds in the Ohio River basin, 99% of the floods from drainage areas of 260 ha occur in May through September; 95% of the floods on drainage areas of 26 million ha occur in October through April.

Long, narrow watersheds are likely to have lower runoff rates than more compact watersheds of the same area. The runoff from the former does not concentrate as quickly as it does from the compact areas, and long watersheds are less likely to be covered uniformly by intense storms. If the long axis of a watershed is parallel to the storm path, a storm moving upstream will cause a lower peak runoff rate than a storm moving downstream. For a storm moving upstream, runoff from the lower end of the watershed may be diminished before the runoff contribution from the upstream area arrives at the outlet. However, a storm moving downstream causes a higher runoff rate because runoff from the lower portion flows concurrently with runoff arriving from the upstream area.

Topographic features, such as slopes of upland areas and channels, channel morphology, and the extent and number of depressed areas, affect rates and volumes of runoff. Watersheds having extensive flat areas or depressed areas without surface outlets have lower runoff than areas with well-defined surface drainage. The geological formation and soil types determine, to a large degree, the infiltration rate, and thus affect runoff.

Land cover and land use practices influence infiltration. Vegetation retards overland flow and increases surface detention and infiltration rates and volumes. Various tillage and land management conditions that affect infiltration directly affect runoff rates and volumes. Structures such as dams, levees, bridges, and culverts all influence runoff rates and volumes.

Estimating Runoff Volume

Total volume of runoff is important in the design of wetlands, retention ponds, reservoirs, and flood control dams. It is necessary to predict the total volume of runoff that may come from a watershed during a design flood so that the structures can be built to control the runoff. The total volume is also needed for estimating the total maximum daily load (TMDL) of pollutants to surface water sources and to develop criteria for protecting water quality.

5.15 Soil Conservation Service (SCS) Method for Runoff Volume Estimation

The Soil Conservation Service (SCS) method (also known as the *Curve Number* method) for predicting runoff volume was primarily developed from many years of storm flow records for agricultural watersheds in many parts of the United States. With proper modifications and assumptions, the method has also been used to estimate runoff from urban areas (SCS, 1986). The runoff volume is usually expressed in the units of depth similar to the precipitation units. An average runoff depth from the entire watershed area is usually considered. If the watershed area is known, the runoff in the units of volume can be obtained by multiplying the watershed area with the runoff depth. The basic runoff equation is

$$Q = \frac{(I - 0.2S)^2}{I + 0.8S} \tag{5.8}$$

where Q = direct surface runoff depth (mm),
I = storm rainfall depth (mm) (Chapter 3),
S = maximum potential difference between rainfall and runoff (mm).

The variable S includes both surface storage and infiltration potential of a watershed. Runoff decreases as S or infiltration increases. The *initial abstraction,* I_a, consists of interception losses, surface storage, and infiltration prior to runoff; it is assumed to be $0.2S$. The SCS (1972) developed a relationship between S and a variable called the curve number, CN, as

$$S = \frac{25400}{CN} - 254 \tag{5.9}$$

The curve number varies from 0 to 100. A higher value means a smaller initial abstraction (for example wet surface condition) and higher runoff. Conversely, a smaller value would give a higher initial abstraction (for example dry surface condition) and smaller runoff. Thus, if $CN = 100$, then $S = 0$, and $Q = I$.

Table 5.2 Runoff Curve Numbers for Urban Areas with Average Runoff Conditions and $I_a = 0.2S$

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	Average % Impervious Area[a]	*A*	*B*	*C*	*D*
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.):[b]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[c]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.05 ha or less (town houses)	65	77	85	90	92
0.1 ha	38	61	75	83	87
0.13ha	30	57	72	81	86
0.2 ha	25	54	70	80	85
0.4 ha	20	51	68	79	84
0.8 ha	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[d]		77	86	91	94

[a] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

[b] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

[c] Composite CNs for natural desert landscaping should be computed using procedures from SCS (1986) based on the impervious area percentage (CN = 98) and the pervious area CNs. The pervious area CNs are assumed equivalent to desert shrub in poor hydrologic condition.

[d] Composite CN to use for the design of temporary measures during grading and construction should be computed using procedures from SCS (1986) based on the degree of development (impervious area percentage) and the CNs for the newly graded pervious areas.

Adapted from SCS (1986).

Curve numbers depend on soil type, land use, ground cover, and soil water conditions. Typical CN values are presented in Tables 5.2, 5.3, and 5.4 for average soil water conditions. These values apply to antecedent rainfall condition II, which is an average value that is used primarily for design applications. Correction factors for other antecedent rainfall conditions are listed in Table 5.5. Antecedent rainfall condition I is for low runoff potential with soil having a low antecedent water content, generally

Table 5.3 Runoff Curve Numbers (CN) for Agricultural Lands with Average Runoff Conditions and I_a = 0.2S

Land Use or Crop	Treatment or Practice[a]	Hydrologic Condition[b]	Curve Number for Hydrologic Soil Group			
			A	*B*	*C*	*D*
Fallow	Bare soil		77	86	91	94
	Crop residue (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	75	82	86
		Good	64	74	81	85
	C + Terraced (T)	Poor	66	74	80	82
		Good	62	71	78	81
	C + T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	C + T	Poor	61	72	79	82
		Good	59	70	78	81
	C + T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded legumes or rotation meadow	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	C + T	Poor	63	73	80	83
		Good	51	67	76	80

[a] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
[b] Hydrologic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good $\geq$ 20 %), and (e) degree of surface roughness.
Source: SCS (1972, 1986).

Table 5.4 Runoff Curve Numbers (CN) for Other Agricultural, Arid, and Semiarid Lands with Average Runoff Conditions and $I_a = 0.2S$.

Cover Description		Curve Numbers for Hydrologic Soil Groups			
Cover Type	Hydrologic Condition	*A*	*B*	*C*	*D*
Other agricultural lands					
Pasture or range: continuous forage for grazing[a]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow (permanent) protected from grazing	Good	30	58	71	78
Brush: brush-weed-grass mixture with brush the major element[a]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30	48	65	73
Woods-grass combination (orchard or tree farm)[b]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods or forest land[c]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads		59	74	82	86
Arid and semiarid rangelands[d]					
Herbaceous: mixture of grass, weeds, and low-growing brush (brush the minor element)	Poor	[e]	80	87	93
	Fair	[e]	71	81	89
	Good	[e]	62	74	85
Oak-aspen-mountain brush	Poor	[e]	66	74	79
	Fair	[e]	48	57	63
	Good	[e]	30	41	48
Pinyon, juniper or both; grass understory	Poor	[e]	75	85	89
	Fair	[e]	58	73	80
	Good	[e]	41	61	71
Sagebrush with grass understory	Poor	[e]	67	80	85
	Fair	[e]	51	63	70
	Good	[e]	35	47	55
Desert shrub	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

[a] Poor: < 50% ground cover
Fair: 50 to 75% ground cover
Good: > 75% ground cover
[b] Computed for 50% woods and 50% grass; use CN for woods and pasture for other percentages.
[c] Poor: forest litter, small trees, and brush are destroyed by heavy grazing or regular burning
Fair: woods are grazed but not burned, and some forest litter covers the soil
Good: woods are protected from grazing, and litter and brush adequately cover the soil
[d] Poor: < 30% ground cover (litter, grass, and brush overstory
Fair: 30 to 70% ground cover
Good > 70% ground cover
[e] Curve numbers not developed.
Source: SCS (1986).

Table 5.5 Antecedent Rainfall Conditions and Curve Numbers (for $I_a = 0.2S$)

Curve Number for Condition II	Factor to Convert Curve Number from Condition II to	
	Condition I	Condition III
10	0.40	2.22
20	0.45	1.85
30	0.50	1.67
40	0.55	1.50
50	0.62	1.40
60	0.67	1.30
70	0.73	1.21
80	0.79	1.14
90	0.87	1.07
100	1.00	1.00

Condition	General Description	5-Day Antecedent Rainfall (mm)	
		Dormant Season	Growing Season
I	Optimum soil condition from about lower plastic limit to wilting point	<13	<36
II	Average value for annual floods	13 to 28	36 to 53
III	Heavy rainfall or light rainfall and low temperatures within 5 days prior to the given storm	>28	>53

Source: SCS (1972).

suitable for cultivation. Antecedent rainfall condition III is for wet conditions prior to a storm event. As indicated in Table 5.5, no upper limit for antecedent rainfall is intended. The limits for the dormant season apply when the soils are not frozen and when no snow is on the ground.

Since the duration of a storm affects the amount of rainfall, runoff volume must be evaluated for each design application. The time of concentration (see Section 5.16) is not a good criterion for the determination of storm volume since a short duration, high intensity storm may produce the largest peak flow for a given watershed, but not necessarily the maximum runoff volume. The SCS has established 6 hours as the minimum storm duration for runoff control structures, but this time is modified for conditions where a greater runoff may result.

Example 5.3

Estimate the volume of runoff during the growing season for a 50-yr return period that may be expected from a 40-ha watershed by O'Hare airport at Chicago, Illinois. Assume that antecedent rainfall during the last 4 of the 5 days prior to the storm was 40 mm and the critical duration of the storm is 6 h. The watershed has the following characteristics:

Sub-area (ha)	Topography (% slope)	Hydrologic Soil Group	Land Use, Treatment, and Hydrologic Condition
24	0 to 5	C	Row crop, contoured, good
16	5 to 10	B	Woodland, good

Solution. From Chapter 3, the 6-h rainfall for a 50-yr return period at O'Hare airport is 122.3 mm. Since the percentage reduction for converting point rainfall to areal rainfall is less than 1% (see Chapter 3) in this case, no correction need be made. Because the 5-day rainfall prior to the event was 40 mm during the growing season, antecedent rainfall condition II applies (Table 5.5). From Tables 5.3 and 5.4 for antecedent rainfall condition II, read the appropriate curve numbers and calculate the weighted value as follows:

Sub-area (ha)	Hydrologic Soil Group	Land Use, Treatment, and Condition	CN	$CN \times$ Area
24	C	Row crop, contoured, good	82	1968
16	B	Woodland, good	55	880
Total 40 ha				Total 2848

Weighted CN = 2848/40 = 71.2 or use 71. Substituting in Equation 5.9,

$$S = \frac{25400}{71} - 254 = 103.7 \text{ mm}$$

$$Q = \frac{(122.3 - 0.2 \times 103.7)^2}{122.3 + (0.8 \times 103.7)} = 50.3 \text{ mm or } \frac{50.3 \times 40 \times 10^4}{1000} = 20120 \text{ m}^3$$

Therefore, the estimated runoff from the 40-ha watershed is 50.3 mm (in depth) or 20 120 m^3 (in volume). The 50.3 mm runoff can also be expressed as 0.0503 m × 40 ha = 2.01 ha·m (in volume).

Example 5.4

Assume the information from Example 5.3 except that the watershed has the following characteristics and curve numbers (Table 5.2).

Sub-area (ha)	Hydrologic Soil Group	Land Use, Treatment, and Condition	CN	$CN \times$ Area
24	C	Industrial area	91	2184
16	B	Town houses	85	1360
Total 40 ha				Total 3544

Solution. Weighted CN = 3544/40 = 88.6 or use 89. Substituting into Equations 5.9 and 5.8 yields

$$S = \frac{25400}{89} - 254 = 31.4 \text{ mm}$$

$$Q = \frac{(122.3 - 0.2 \times 31.4)^2}{122.3 + (0.8 \times 31.4)} = 91.3 \text{ mm or } \frac{91.3 \times 40 \times 10^4}{1000} = 36\,520 \text{ m}^3$$

The runoff is about 80% higher than for the row crop and wooded watershed in Example 5.3.

Example 5.5

If 60 mm of rainfall occurs the day after the 50-yr storm in Example 5.3, what is the expected runoff?

Solution. From Table 5.5, antecedent rainfall condition III applies because the 5-day prior rainfall was 40 + 122.3 = 162.3 mm, which exceeds the value of 53 mm. From Table 5.5, interpolate a correction factor of 1.20 for CN = 71. The new curve number is 71 × 1.20 = 85.4 or use 85. From Equation 5.9 for CN = 85, calculate S = 43.4 mm.

$$S = \frac{25400}{85} - 254 = 44.8$$

Substituting into Equation 5.8, for Q,

$$Q = \frac{(60 - 0.2 \times 44.8)^2}{60 + 0.8 \times 44.8} = 27.2 \text{ mm}$$

Although the rainfall of 60 mm was about half the 120.6 mm in Example 5.3, the runoff was about 56% (27.2/49.0) of the previous amount, illustrating the importance of antecedent rainfall.

Design Runoff Rates

The design runoff rate is the maximum runoff rate that will occur from a storm of a specific duration and recurrence period. Structures and channels are designed for a specific return period and must withstand the runoff rates produced by events of that magnitude. Vegetated and temporary structures such as vegetative waterways, earthen channels, and filter strips are usually designed for the maximum runoff rate that may be expected once in 10 years. Expensive, permanent structures such as dams and reservoirs are designed for runoff expected to occur once in 50 or 100 years. Selection of the design return period, also called recurrence interval, depends on the economic balance between the cost of periodic repair or replacement of the facility and the cost of providing additional capacity to reduce the frequency of repair or replacement. In most instances, the potential damage from failure of the structure dictates the selection of the return period of the design storm.

A relationship between the return period (T) and the acceptable probability (P_r) of the design capacity being exceeded during the design life of a structure can be used to calculate the required design period (Haan et al., 1994), as

$$P_r = 1 - (1 - 1/T)^n \tag{5.10}$$

where n is the design life of the structure in years. According to Equation 5.10, for a structure designed for a 10-yr return period, the probability that the design capacity will be exceeded at least once during the 10-yr life of the structure is

$$P_\mathrm{r} = 1 - (1 - 1/10)^{10} = 0.65 \text{ or } 65\%$$

If this risk is reduced to P_r = 10%, the 10-yr structure should be designed on the basis of a T = 95-yr return period, which may be very expensive. The criteria on which the risk and return period are selected should be based on the consequences of the design capacity being exceeded.

There are several methods for estimating a design runoff rate. These methods make simplifying assumptions regarding the influence of some factors and necessarily neglect other factors. Methods presented here are applicable to watersheds of less than a few hundred hectares.

5.16 SCS-TR55 Method for Estimating Peak Runoff Rate

The SCS-TR55 method has been widely used to estimate peak runoff rates from small rural and urban watersheds (SCS, 1986). This method of estimating peak runoff rate is applicable to 24-hour rainfall events on watersheds that are smaller than 900 ha and with average slopes greater than 0.5% with one main channel or two tributaries with approximately equal times of concentration. The peak runoff rate equation was developed from the analysis of hydrographs by the SCS-TR20 computer program (SCS, 1983), and is given by

$$q = q_u A\, Q\, F_p \tag{5.11}$$

where q = peak runoff rate (m^3/s),
q_u = unit peak runoff rate (m^3/s per ha per mm of runoff),
A = watershed area (ha),
Q = runoff depth from a 24-h storm of the desired return period (Equation 5.8) (mm),
F_p = pond and swamp adjustment factor from Table 5.6.

Before Equation 5.11 is utilized for estimating peak runoff rate, the time of concentration of a watershed must be calculated. This is the time required for water to flow from the most hydraulically remote (in time of flow) point of the watershed to the outlet once the soil has become saturated and minor depressions are filled. It is assumed that, when the duration of a storm equals the time of concentration, all parts of the

Table 5.6 Adjustment Factor, F_p, for Pond and Swamp Areas that are Spread Throughout the Watershed

Percentage of Pond and Swamp Areas	F_p
0.0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72
Source: SCS (1986).	

watershed are contributing simultaneously to the discharge at the outlet. The time of concentration may be obtained from the equation (SCS, 1990)

$$T_c = L^{0.8}\left[\frac{\left(\frac{1000}{CN}-9\right)^{0.7}}{4407\left(S_g\right)^{0.5}}\right] \tag{5.12}$$

where T_c = time of concentration (h),
L = longest flow length (from the most remote point to the outlet) (m),
CN = runoff curve number (Section 5.15),
S_g = average watershed gradient (m/m).

After calculating the time of concentration from Equation 5.12, unit peak runoff rate q_u is obtained from Figure 5.4 using T_c and the ratio of initial abstraction (I_a) to

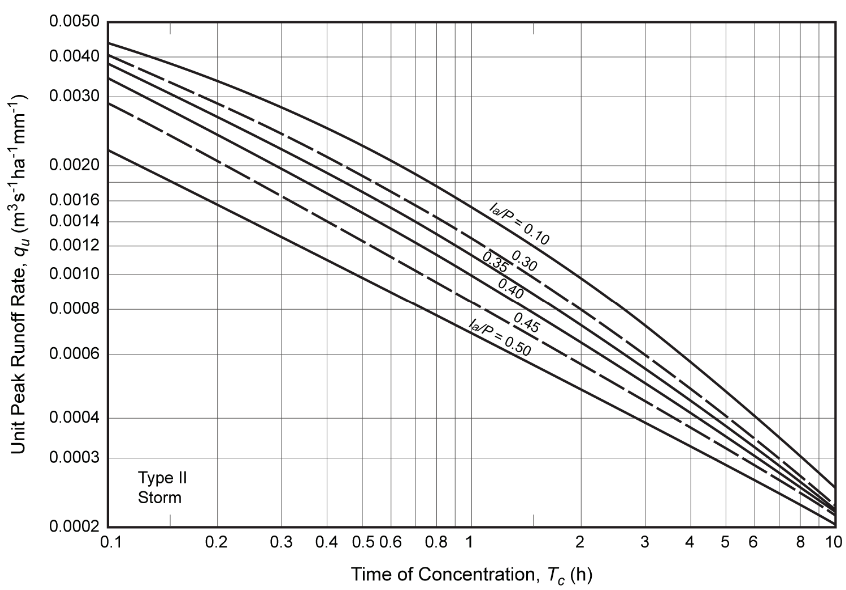

Figure 5.4–Unit peak runoff rates for SCS Type II rainfall distribution. (Revised from SCS, 1990.)

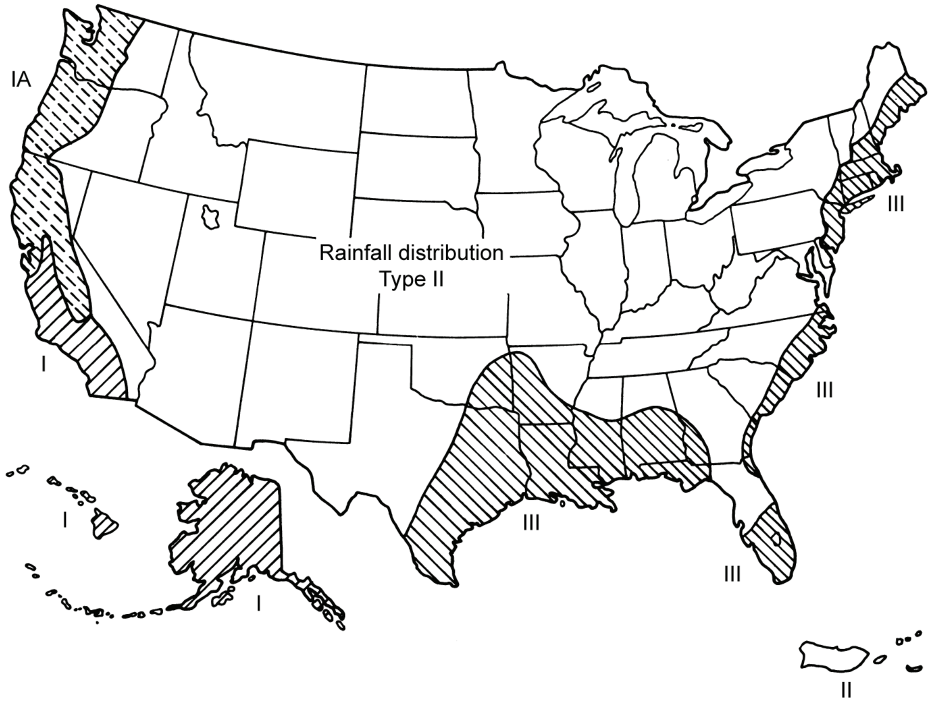

Figure 5.5–Approximate geographic boundaries for SCS rainfall distributions. (Revised from SCS, 1990.)

24-h rainfall (P) with a return period equal to the return period of the peak flow. This ratio (I_a /P) represents the fraction of rainfall that occurs before runoff begins. The initial abstraction is usually taken as $I_a = 0.2S$, and S is calculated using Equation 5.9. The curves in Figure 5.4 apply only for the Type II rainfall distribution, which is applicable for the unshaded areas of the United States shown in Figure 5.5. Curves for other types of rainfall are given by SCS (1986, 1990) and McCuen (1989). When using Figure 5.4, if $I_a /P < 0.10$, use $I_a /P = 0.10$. If $I_a /P > 0.50$, use $I_a /P = 0.50$. Application of the method is shown in Example 5.5.

Example 5.6

Determine the peak runoff rate from a 100-ha watershed from a 120-mm, 24-h storm that produced 10-mm depth of runoff. Assume a flow length of 1500 m, antecedent rainfall condition II, weighted average curve number of 75, an average watershed gradient of 0.02 m/m, and 0.2% pond and swamp areas.

Solution. Substitute in Equation 5.9 as

$$S = (25400/75) - 254 = 85$$

From Equation 5.12,

$$T_c = 1500^{0.8}\left[\frac{\left(\frac{1000}{75} - 9\right)^{0.7}}{4407\ (0.02)^{0.5}}\right] = 1.56 \text{ h}$$

Then $I_a = 0.2S = 0.2 \times 85 = 17$ mm

$$I_a/P = 17/120 = 0.14$$

Read from Figure 5.4, $q_u = 0.0012\ \text{m}^3\text{s}^{-1}\text{ha}^{-1}\text{mm}^{-1}$

Find $F_p = 0.97$ for 0.2% pond and swamp areas in the watershed from Table 5.6. Substitute these values into Equation 5.11:

$$q = 0.0012 \times 100 \times 10 \times 0.97 = 1.16\ \text{m}^3/\text{s}$$

A computer-based expansion of TR-55, WinTR-55, is available from NRCS. It can analyze watersheds up to 6500 ha with up to 10 sub-areas. It includes the ability to model channel reaches and pipe or weir structures within the watershed.

5.17 Rational Method for Estimating Peak Runoff Rate

The Rational Method is widely used for estimating peak runoff rates from small watersheds. It is particularly useful for conveyance designs that are based on relatively frequent (2-10 year) events. The Rational Method was first developed by Kuichling (1889), but many modifications, adjustments, and amplifications have been proposed to improve and adapt it for use with modern urban watersheds. Upper limits for watershed area, typically 800 to 1200 ha, can be found in local design guides.

The Rational Method assumes that runoff is produced from a constant-intensity rainfall that is uniform over the entire watershed for a duration that is equal to the time of concentration of the watershed. The Rational Method is expressed as

$$q = CiA \tag{5.13}$$

where q = peak discharge (L^3T^{-1}),
C = runoff coefficient;
i = average rainfall intensity (LT^{-1});
A = watershed area (L^2).

The runoff coefficient C is simply the ratio of outflow rate to inflow rate, where inflow rate is the product of rainfall intensity and watershed area.

For i in mm/h and A in ha, a conversion constant of 1/360 yields q in m^3/s. For i in inches per hour and A in acres, a conversion constant of 1.008 (usually rounded to 1) yields q in cubic feet per second.

The average rainfall intensity i for a design event is estimated from the intensity-duration-frequency data for the location. The design storm frequency is the same as that selected for the design flood. The duration of the design storm is normally set equal to the time of concentration t_c. A storm of that duration will be the most intense for which the entire watershed contributes to the discharge and will therefore produce the highest discharge. For a longer duration, the intensity would be less. For a shorter duration, the contributing area would be smaller. It is assumed that the effect of the reduction in area is greater than the effect of increased intensity.

Many methods have been proposed for calculation of time of concentration. Some methods separate sheet or overland flow from concentrated or channelized flow and add the travel times to obtain t_c. Others simplify t_c into a single calculation. The Kirpich (1940) formula is widely used for simple watersheds. It is

$$t_c = kL^{0.77}\, s^{-0.385} \tag{5.14}$$

where t_c = time of concentration (minutes);
k = conversion constant: 0.0195 for L in meters, 0.0078 for L in feet;
L = maximum length of flow (L);
s = average watershed slope (LL^{-1}).

The runoff coefficient C combines the effects of land use, land cover, soils, slopes, and management practices into a single number. Consult local agencies and design guides for recommended/required values and methods. Runoff coefficients for urban and suburban settings are given in Table 5.7. Table 5.8 provides values for agricultural watersheds that include adjustments for three average rainfall intensities. The values in Table 5.8 are for soils of Hydrologic Soil Group B. To adjust for Soil Groups A, C, or D, multiply C by the ratio of the runoff curve number for the desired soil group to the runoff curve number for Group B (Table 5.3).

For average recurrence intervals of 2-10 years, the C values from the tables are used directly. For more extreme events, the runoff coefficient can be adjusted with the values in Table 5.9 to account for the reduced importance of infiltration, detention storage, and other minor factors affecting runoff. The adjusted runoff coefficient cannot exceed 1, i.e., there can never be more runoff than rainfall.

Table 5.7 Runoff Coefficients Recommended by ASCE and WPCF

	Runoff Coefficients		Runoff Coefficients
Description of Area:		Character of Surface:	
Business		Pavement	
Downtown	0.70-0.95	Asphaltic and concrete	0.70-0.95
Neighborhood	0.50-0.70	Brick	0.70-0.85
Residential		Roofs	0.75-0.95
Single-family	0.30-0.50	Lawns, sandy soil	
Multiunits, detached	0.40-0.60	Flat, 2 percent	0.05-0.10
Multiunits, attached	0.60-0.75	Average, 2-7 percent	0.10-0.15
Residential (suburban)	0.25-0.40	Steep, 7 percent	0.15-0.20
Apartment	0.50-0.70	Lawns, heavy soil	
Industrial		Flat, 2 percent	0.13-0.17
Light	0.50-0.80	Average, 2 to 7 percent	0.18-0.22
Heavy	0.60-0.90	Steep, 7 percent	0.25-0.35
Parks, cemeteries	0.10-0.25		
Playgrounds	0.20-0.35		
Railroad yard	0.20 - 0.35		
Unimproved	0.10-0.30		
Source: ASCE and WPCF (1969).			

Table 5.8 Runoff Coefficient C for Agricultural Watersheds (Hydrologic Soil Group B)

Crop and Hydrologic Condition	Coefficient *C* for Rainfall Rates of		
	25 mm/h	100 mm/h	200 mm/h
Row crop, poor practice	0.63	0.65	0.66
Row crop, good practice	0.47	0.56	0.62
Small grain, poor practice	0.38	0.38	0.38
Small grain, good practice	0.18	0.21	0.22
Meadow, rotation, good	0.29	0.36	0.39
Pasture, permanent, good	0.02	0.17	0.23
Woodland, mature, good	0.02	0.10	0.15

Source: Horn and Schwab (1963).

Table 5.9 Runoff Coefficient Frequency Factors

Average Recurrence Interval (years)	C_f
2-10	1.0
25	1.1.
50	1.2
100	1.25

The simplest application of the Rational Method treats the watershed as a homogeneous unit having a single *C* value. Nonhomogeneous watersheds in which land uses/covers are mixed can be handled by using a weighted-average *C* in which the weights are the fractions of the total area represented by each use/cover. For watersheds that have larger distinct land use areas, better results can be obtained by treating the subareas as separate units and summing their discharges.

Example 5.7

Estimate the peak runoff rate for a 10-yr storm from 45 ha of farmland (row-crop, good practice) near St. Louis, Missouri. The average gradient is 0.5%, the maximum flow path is 975 m, and the soils are clays.

Solution. Use the Kirpich formula to calculate t_c.

$$t_c = 0.0195L^{0.77}\, s^{-0.385} = 0.0195(975^{0.77})(0.005^{-0.385}) = 30 \text{ min}$$

From Table 3.1, the rainfall intensity for a 10-yr 30-min event is 42 mm/0.5 h = 84 mm/h. This is close to 100 mm/h, so select $C = 0.56$ from Table 5.8. Assuming that the clay soils are Group D, the *C*-value must be adjusted by the curve number ratio from Table 5.3. Assuming contouring and good hydrologic condition, the curve number for Group D is 86 and the curve number for Group B is 75. Applying the adjustment factor of 86/75 to the Group B *C*-factor of 0.56 gives $C = 0.64$.

The Rational Method then gives

$$q = CiA = 0.64\left(84 \text{ mm/h}\right)\left(45 \text{ ha}\right) = 2420\frac{\text{ha}\cdot\text{mm}}{\text{h}} = 6.7 \text{ m}^3/\text{s}$$

Example 5.8

Estimate the peak runoff rate for a 25-yr storm on a 400-ha development in Greensboro, North Carolina. The land uses are 80% single family residences and 20% asphalt and concrete pavement. The average slope of the watershed is 2% and the longest flow path is 2800 m.

Solution. It would be better to use a method for time of concentration that computes overland and channel flows separately, but for this example, use the Kirpich formula.

$$t_c = 0.0195 L^{0.77} s^{-0.385} = 0.0078\left(2800^{0.77}\right)\left(0.02^{-0.385}\right) = 40 \text{ min}$$

A weighted-average runoff coefficient (using the middles of the ranges in Table 5.7) is calculated as

$$C_{avg} = 0.80(0.40) + 0.20(0.825) = 0.485$$

This value needs to be adjusted because the average recurrence interval (ARI) is greater than 10 years. The 25-yr ARI adjustment factor from Table 5.9 gives C = 1.1(0.485) = 0.5335.

Interpolation of the PFDS outputs for Greensboro gives rainfall intensity for a 25-yr 40-min storm as 84 mm/h. The Rational Method estimate of peak runoff rate is then

$$q = CiA = 0.5335(84)(400)\frac{1}{360} = 50 \text{ m}^3/\text{s}$$

5.18 Flood Frequency Analysis Method

Another method of runoff rate estimation, called flood frequency analysis, depends on the historical records from the drainage area under study. These records constitute a statistical array that defines the probable frequency of recurrence of floods of given magnitudes. Extrapolation of the frequency curves can be used to predict flood peaks for a range of return periods. The procedure for this method is the same as that for rainfall described in Chapter 3.

5.19 Computer Model Prediction of Runoff

Numerous computer models have been developed to predict storm runoff, many for special applications. The hydrologic and erosion models CREAMS (Knisel, 1980), WEPP (Nearing et al., 1989), and AnnAGNPS (Bingner et al., 2001), drainage model DRAINMOD (Skaggs, 1982), soil and water assessment SWAT (Arnold et al., 1993), storm water management SWMM (Metcalf and Eddy, Inc., et al., 1971) and water quality model RZWQM (Ahuja et al., 1999) include runoff predictions within the models. Many other models have been developed with runoff prediction capabilities for such applications as forests, water quality, frozen and thawing soils, wetlands, surface mines, wind erosion prediction, and plant growth applications (Goodrich and Woolhiser, 1991). Several hydrologic models have been integrated with GIS to take account of spatial variability of soils, cover, land use, and land management conditions.

When selecting a runoff model, the user should generally select the model that best suits the purpose. Generally, more sophisticated models require larger input files, and obtaining the necessary input data can be time consuming and difficult. Models allow the user the opportunity to compare the effects of different land use and land management practices on runoff and other watershed responses, and to select best management practices (BMPs) for a given situation.

Runoff Hydrographs

A hydrograph represents flow rate with respect to time. A stream flow hydrograph may have four components: (1) base flow, (2) direct runoff, (3) interflow, and (4) channel precipitation. Base flow is the local groundwater flow component that discharges into the stream. Direct runoff is the surface runoff component that reaches the stream and is often the most significant component of the hydrograph after a precipitation event. Interflow moves laterally in the soil at shallow depths and discharges into the streams. In flat watersheds with tile drainage systems, interflow may move through the tile and discharge into the stream. Channel precipitation is the flow contribution from precipitation directly on the stream. In hydrologic analysis, channel precipitation and interflow are most often considered together with the direct runoff. Since the base flow component is almost constant, runoff dictates the shape of a hydrograph. From stream flow records, the base flow component can be subtracted to obtain ordinates of the surface runoff hydrograph. Hydrograph analysis is a convenient method for determining peak flow rates.

Figure 5.6 shows the measured precipitation rate and runoff hydrograph from a 123-ha agricultural watershed having woods, pasture, cropland, and hay near Coshocton, Ohio. The element of the hydrograph representing the increasing flow rate is called the rising limb; the element representing flow rate from the peak to the end of the flow period is called the falling or recession limb. In Figure 5.6, both the precipitation rate and the runoff rate show two peaks with runoff slightly delayed. The area under the hydrograph represents the volume of runoff. For the storm, 35.6 mm of rainfall occurred while the runoff was 19.1 mm. Water samples were collected and analyzed. The results for one sample taken during a high flow period are given in Table 5.10.

Small agricultural watersheds rarely have adequate stream flow records for hydrograph development. However, methods are available for developing hydrographs when the actual flow record is not available. These methods are based on some common characteristics. One of the characteristics of hydrographs for a given watershed is that the duration of flow is nearly constant for individual storms regardless of the peak flow.

Several theoretical hydrographs have been proposed based on different statistical frequency distributions. Dodge (1959) developed a unit hydrograph from the Poisson probability function, Gray (1973) employed a two-parameter gamma distribution, and Reich (1962) investigated a three-parameter Pearson type III function. Linsley et al.

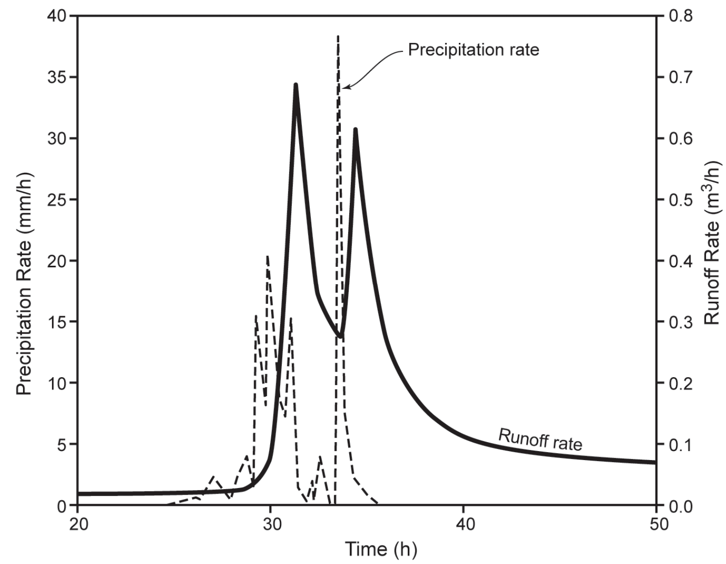

Figure 5.6–Example of a measured runoff hydrograph and precipitation from Watershed 196 of the North Appalachian Experimental Watershed, Coshocton, Ohio. (Adapted from data supplied by Bonta, 2004.)

Table 5.10 Results of Analysis of Runoff from Watershed 196 of the North Appalachian Experimental Watershed, Coshocton, Ohio

Cl (mg/L)	Br (mg/L)	NO_3-N (mg/L)	PO_4-P (mg/L)	SO_4 (mg/L)	Na (mg/L)	NH_4-N (mg/L)
8.2	0.0	1.0	0.0	22.2	8.2	0.0
K (mg/L)	Mg (mg/L)	Ca (mg/L)	TOC (mg/L)	pH	Sediment (mg/L)	
3.4	8.2	16.6	10.4	7.8	336	
Data provided by Bonta (2004).						

(1982) described several methods of developing unit hydrographs and a hydrograph for overland flow.

5.20 Triangular Hydrograph

The simplest form of runoff hydrograph is the triangular hydrograph, as shown by the dashed lines in Figure 5.7. Except for the tail end, it is a good approximation of an actual hydrograph. As shown in Figure 5.7, the time-to-peak T_p and the peak flow rate for the triangular hydrograph are the same as those of the dimensionless hydrograph. When constructing a triangular hydrograph, the altitude of the triangle should be the peak flow rate and the base of the triangle taken as 2.67 T_p. These are applied to either subareas of a watershed or time increments of a rainstorm. In either case, the flow rates at a given time are added from several triangular hydrographs, thus producing a curvilinear hydrograph.

5.21 Dimensionless Hydrograph

Commons (1942) and later others developed dimensionless hydrographs, as shown by the smooth curve in Figure 5.7. The shape approximates the flow from an intense storm from a small watershed. Sometimes called synthetic hydrographs, they have an idealized shape and can be used to develop approximate design hydrographs for any small watershed for which flow records are not available. The dimensionless hydrograph in Figure 5.7 divides the peak flow rate into 100 flow units and divides the duration of flow into 100 units of time with a total area of 2670 volume units under the hydrograph. To develop a design hydrograph for a watershed from the dimensionless hydrograph, the peak flow rate (q) and the total runoff volume (Q) for the desired return period storm must be known. The values of q and Q can be determined for any desired storm for a watershed by methods described in Sections 5.15 and 5.16.

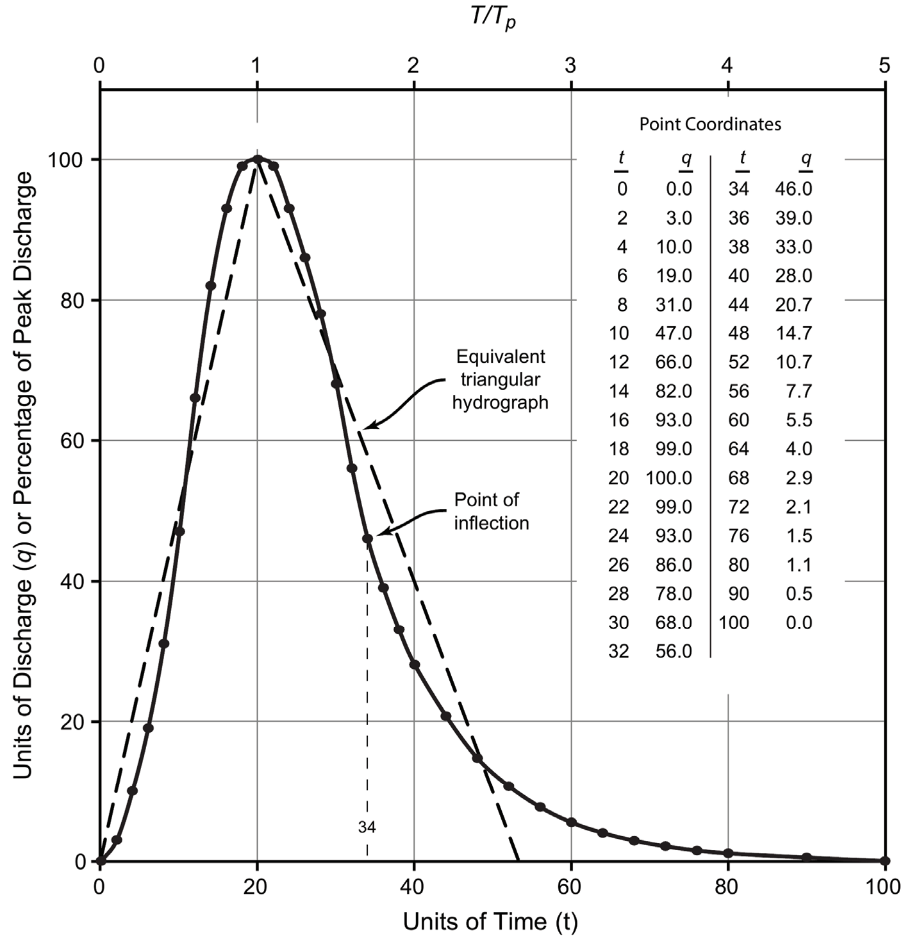

Figure 5.7–Dimensionless and triangular flood hydrographs. (Adapted from NRCS National Engineering Handbook, Part 630, Chapter 16: Hydrographs. 210-VI-NEH. March 2007.)

The design hydrograph is developed from the dimensionless hydrograph by using appropriate conversion factors. The factor u is the ratio of the total runoff volume from the desired storm to the area under the dimensionless hydrograph. Since the area under the dimensionless hydrograph is 2670 volume units and the design storm has a total runoff volume Q, each volume unit under the dimensionless hydrograph has a value of

$$u = Q/2670 \tag{5.15}$$

in the design hydrograph. The factor w is the ratio of peak runoff rate (q) for the design storm to the peak flow of 100 on the dimensionless hydrograph. Thus, each unit of flow on the dimensionless hydrograph has a value of

$$w = q/100 \tag{5.16}$$

in the hydrograph of the design storm. The factor k is the value that each unit of time on the dimensionless hydrograph represents in the design hydrograph. On the design hydrograph, 1/100 of the peak flow times 1/100 of the duration of runoff must equal 1/2670 of the total runoff volume. Since w is equal to 1/100 of the design peak flow, k must be equal to 1/100 of the design hydrograph duration, and u is 1/2670 of the design runoff volume. Therefore,

$$w\,k = u \quad or \quad k = u/w \tag{5.17}$$

When runoff rate q is measured in m^3/s, runoff volume Q is measured in ha·m, and time is measured in minutes, u will be in ha·m per unit, w will be in m^3/s per unit, and

$$k = \frac{u(\text{ha}\cdot\text{m})\times 10\,000(\text{m}^2/\text{ha})}{w(\text{m}^3/s)\times 60(\text{s}/\text{min})} = 167\frac{u}{w}\ \text{minutes per unit} \tag{5.18}$$

The coordinates of the design hydrograph are obtained by multiplying the ordinates and abscissas of the dimensionless hydrograph by w and k, respectively. Example 5.9 illustrates the development of a design hydrograph from the dimensionless hydrograph.

Example 5.9

Using the dimensionless hydrograph, develop a design runoff hydrograph for a return period of 50 years for the watershed in Example 5.3. Assume a peak runoff rate of 7.0 m^3/s.

Solution. From Example 5.3, the runoff volume is 2.01 ha·m. From Equations 5.15, 5.16, and 5.18,

$$u = 2.01/2670 = 0.000753 \text{ ha·m/unit}$$

$$w = 7.0/100 = 0.07 \text{ m}^3\text{/s per unit}$$

$$k = 167 \times 0.000753/0.07 = 1.80 \text{ min/unit}$$

Ordinates and abscissas of the design runoff hydrograph are obtained by multiplying the values of q and t from Figure 5.7 by w and k, respectively. The calculated coordinates are as follows:

$k \times t$ (min)	$w \times q$ (m^3/s)	$k \times t$ (min)	$w \times q$ (m^3/s)	$k \times t$ (min)	$w \times q$ (m^3/s)
0.0	0.00	39.6	6.93	86.3	1.03
3.6	0.21	43.1	6.51	93.5	0.75
7.2	0.70	46.7	6.02	100.7	0.54
10.8	1.33	50.3	5.46	107.9	0.39
14.4	2.17	53.9	4.76	115.1	0.28
18.0	3.29	57.5	3.92	122.2	0.20
21.6	4.62	61.1	3.22	129.4	0.15
25.2	5.74	64.7	2.73	136.6	0.11
28.8	6.51	68.3	2.31	143.8	0.08
32.4	6.93	71.9	1.96	161.8	0.04
36.0	7.00	79.1	1.45	179.8	0.00

If these points are plotted, the hydrograph will have the shape shown in Figure 5.7 with point *i* at T_p and a peak flow of 7.0 m^3/s.

5.22 Unit Hydrograph

The unit hydrograph developed by Sherman (1932), as described by Haan et al. (1994), is "… a hydrograph of runoff resulting from a unit of excess rainfall occurring at a uniform rate, uniformly distributed over a watershed in a specified duration of time." In addition to the restrictions in the definition, the unit hydrograph is assumed to reflect all watershed characteristics so that the runoff rate is proportional to the runoff volume for a rainfall excess of a specific duration.

Based on the above definition, every watershed has a unit hydrograph for a specific duration of rainfall excess. Durations of 20 minutes, 1 h, 2 h, 6 h, or 24 h are typical examples. Theoretically a unit hydrograph could be constructed for an infinite number of durations for every watershed. For practical reasons, the unit hydrograph is applied to rainfall excesses of durations up to 25% different than the duration of the unit hydrograph (Haan et al., 1994).

Like the dimensionless hydrograph, a unit hydrograph can also be used to construct a design runoff hydrograph for an ungauged watershed if the rainfall excess depth for any specific storm duration is known. The difference between these two methods is that the shape of the dimensionless hydrograph is fixed regardless of watershed location, whereas the shape of a unit hydrograph of any duration can be specific to a particular watershed. Therefore, the design hydrograph constructed from a unit hydrograph of a watershed can be more accurate for that watershed than that constructed from the idealized dimensionless hydrograph. In this case, however, the unit hydrograph for a specific rainfall excess duration for the specific watershed must be available to construct a design hydrograph for that watershed.

Plots of unit hydrographs show flow rate versus time. The flow rate is typically shown as volume per time. Because a unit hydrograph has a depth associated with it, the flow rate is actually volume per unit time per unit depth of excess rainfall. To obtain the ordinates of the design runoff hydrograph from a unit hydrograph, the corresponding ordinate of the unit hydrograph should be multiplied by the rainfall excess depth. The following example illustrates a simple procedure for constructing a unit hydrograph and the application of the unit hydrograph for developing a design runoff hydrograph.

Example 5.10

Actual runoff rates resulting from a rainfall excess of 2-hour duration for a particular watershed are given as follows:

Time, h	0	1	2	3	4	5	6	7
Flow rate, m^3/s	0	2	5	3	2	1	0.5	0

The total runoff volume for this event was 35 mm. Construct a 2-h unit hydrograph. Apply the unit hydrograph to develop a design runoff hydrograph for this watershed that will result from two successive 2-h rainfall runoff events (rainfall excess duration is 4 h) of 15 and 25 mm.

Solution. First, the ordinates of the unit hydrographs are calculated by dividing the actual runoff rates by the total runoff depth of 35 mm to obtain a 2-h, 1-mm unit hydrograph. The values are shown in column [2]. For example, dividing the flow rate of 2 m^3/s at time = 1 h by 35 mm, the unit hydrograph ordinate = 2/35 = 0.057 $m^3s^{-1}mm^{-1}$. Details of the calculations are tabulated below.

Column [1] contains the time steps and column [2] has the corresponding ordinates for a 2-h unit hydrograph. These 2-h unit hydrograph ordinates were utilized to construct a design runoff hydrograph for excess rainfall duration of 4 hours. The first excess rainfall of 15 mm lasted for 2 h and the hydrograph ordinates for this event were obtained by multiplying the unit hydrograph ordinates from column [2] by

Time (h) [1]	Unit Hydrograph Ordinates ($m^3s^{-1}mm^{-1}$) [2]	Rain Event [3]	Runoff (mm) [4]	Hydrograph Ordinates for Rain 1 (m^3/s) [5]	Hydrograph Ordinates for Rain 2 (m^3/s) [6]	Ordinate for the Design Hydrograph (m^3/s) [7]
0	0	1	15	0	--	0
1	0.057			0.855	--	0.855
2	0.143	2	25	2.145	0	2.145
3	0.086			1.290	1.425	2.715
4	0.057			0.855	3.575	4.430
5	0.029			0.435	2.150	2.585
6	0.014			0.210	1.425	1.635
7	0			0	0.725	0.725
8				--	0.350	0.350
9				--	0	0

15 mm. These values are shown in column [5]. A second rainfall excess of 25 mm occurred for the next 2 hours, and the hydrograph ordinates for this rain were obtained by multiplying the unit hydrograph ordinates in column [2] by 25 mm. These values are shown in column [6] start at the third hour and end at the ninth hour after the start of runoff. The values of the flow in columns [5] and [6] were added to obtain the direct runoff ordinates in column [7]. Column [7] values can be plotted against time (column [1]) to obtain the design runoff hydrograph. This plot will show that the peak runoff rate is about 4.5 m^3/s and occurs early in the fourth hour.

Events of different durations cannot be added to obtain unit hydrographs. The unit hydrographs of multiple events of the same duration can be added to calculate new unit hydrograph. For example, three 1-h unit hydrographs can be added, but a 1-h and a 2-h cannot be added for a 3-h unit hydrograph. After the three 1-h unit hydrographs are added, the sum must be divided by three to obtain the 3-h unit hydrograph.

Internet Resources

NRCS source for TR-55 and TR-20:
http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/home/?cid=stelprdb1042480

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Problems

5.1 Assuming that the Horton infiltration Equation 5.4 is valid, determine the constant infiltration rate if f_0 = 50 mm/h, f at 10 min is 13 mm/h, and k = 12.9 h^{-1}. What is the infiltration rate at 20 min? Determine the infiltrated depth after 2 h of wetting.

5.2 Estimate the runoff volume, in mm of depth, from 80 mm of rainfall on a 90-ha watershed during the growing season. Assume antecedent rainfall condition II, 50 ha of row crop contoured with terraces, poor condition, soil group C, and 40 ha of alfalfa, straight, good condition, soil group B.

5.3 Estimate the runoff volume, in mm, m^3, and ha·m, for the watershed and conditions in Problem 5.2, except that a 60-mm storm occurred 3 days before the 80-mm rainfall.

5.4 Estimate the runoff volume, in mm, for the watershed and conditions in Problem 5.2, except that no rainfall occurred for 6 days before the 80-mm rainfall.

5.5 Estimate the runoff volume, in mm, for a 24-h, 50-yr return period storm at your location. Assume the watershed conditions in Problem 5.2.

5.6 Estimate the runoff volume, in mm of depth, from 80 mm of rainfall on a 90-ha watershed during the growing season. Assume antecedent rainfall condition II, 50 ha of houses on 0.1-ha lots, soil group B, and 40 ha of businesses, soil group B.

5.7 Estimate the peak runoff rate from a 150-ha watershed from a 100-mm, 24-h storm that produced 8 mm depth of runoff. Assume antecedent rainfall condition II, $CN = 70$, flow length = 2000 m, average gradient = 0.01 m/m, and 1% pond and swamp.

5.8 Estimate the design peak runoff rate for a 24-h, 25-yr storm. The watershed consists of 60 ha, one-third in good permanent meadow with no grazing and the remainder in row crops on the contour in good condition. The hydrologic soil group for the area under meadow is B and that for the row crop is C. The watershed is near St. Louis, Missouri. The maximum length of flow of water is 1000 m and the fall along this path is 5.0 m.

5.09 If the watershed in Problem 5.8 is in western Tennessee at 36ºN 89ºW, estimate peak runoff rate. If the return period was decreased to 2 years, what would be the rate of runoff?

5.10 Estimate the runoff volume and peak runoff rate from a 24-h, 25-y storm during the dormant season on a 200-ha watershed near Benson, Arizona. The watershed has desert shrub in fair condition, and the soil is hydrologic group C. Assume an average slope of 0.02 m/m, that a 10-mm rainfall occurred 3 days prior to this event, and the maximum length of flow is 3000 m.

5.11 Assuming the dimensionless hydrograph is applicable, determine the duration of flow if the peak runoff is 4.0 m^3/s and the volume of runoff is 50 mm for a 50-year storm from an 85-ha watershed. What are the coordinates for point n on the design hydrograph?

5.12 By frequency analysis methods discussed in Chapter 3, determine the estimated maximum annual discharge for return periods of 5 and 100 years from the following 18 years of maximum annual floods (1979-1996) from a gauged watershed: 50, 92, 108, 1, 0.8, 1.5, 20, 36, 0.5, 0.3, 56, 38, 2, 8, 14, 5, 0.4, and 3 mm. Is the length of record adequate for these estimates to be reliable 90% of the time?

5.13 The flow rates from a 1-h rainfall excess of 40 mm in an agricultural watershed were 0, 3, 8, 15, 7, 5, 3, 1, and 0 m^3/s at 0, 1, 2, 3, 4, 5, 6, 7, and 8 h, respectively. By the unit hydrograph method, construct the design runoff hydrograph for the watershed for a total excess rainfall of 90 mm that fell in the following order: 30 mm in the first hour, 50 mm in the second hour, and 10 mm in the third hour.

5.14 Construct a 2-h unit hydrograph from the data in Problem 5.13.