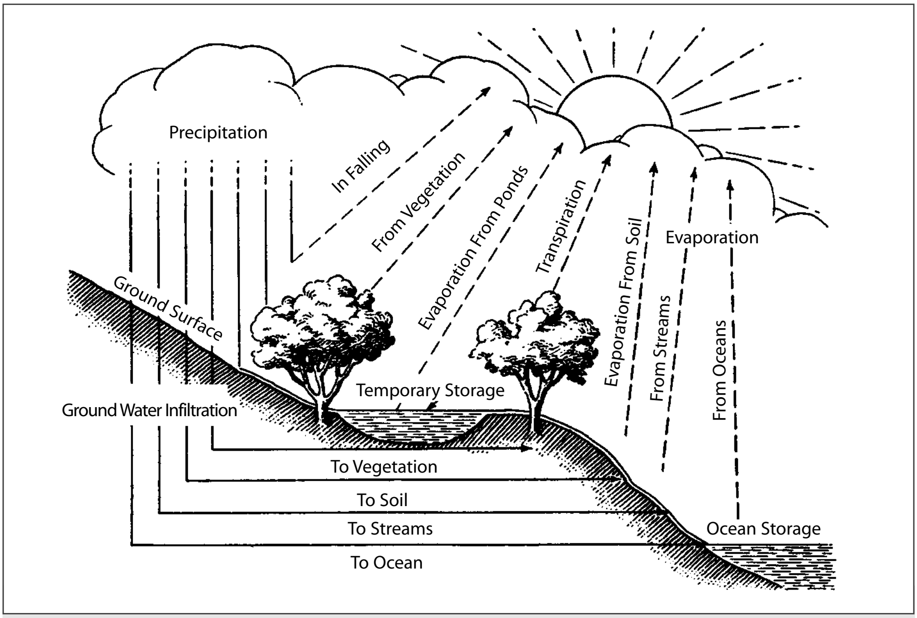
CHAPTER three

INTRODUCTION

The hydrological cycle is a continuous process whereby water precipitates from the atmosphere and is transported from ocean and land surfaces back to the atmosphere from which it again precipitates. There are many inter-related phenomena involved in this process as conceptualized in Figure 3.1.

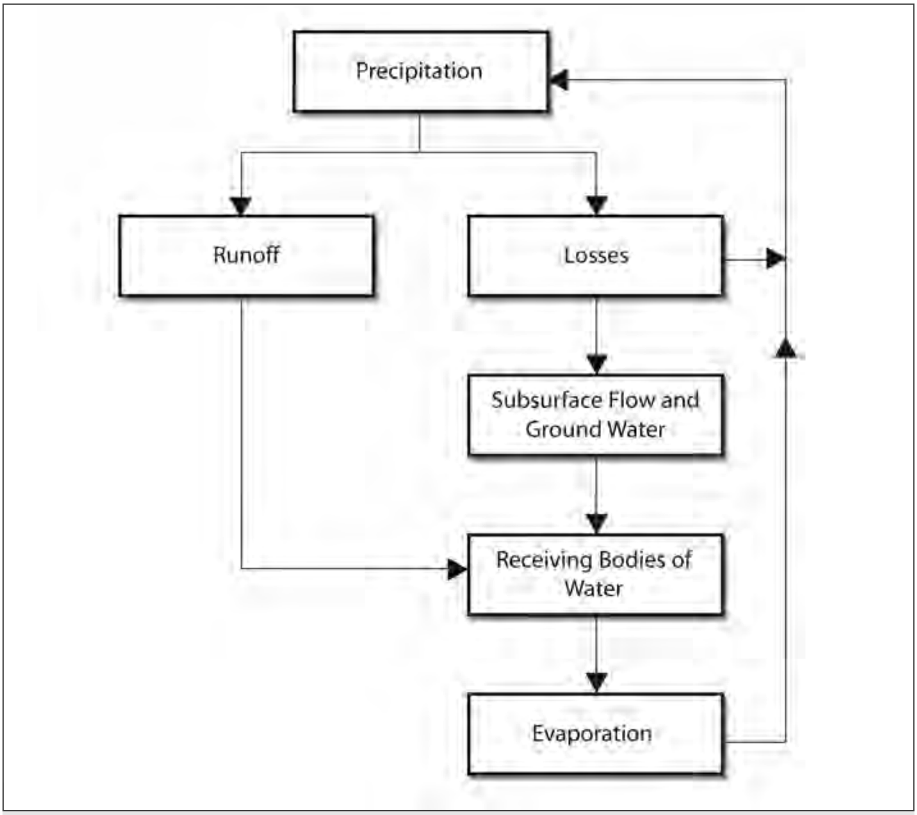


■ **Figure 3.1** Hydrologic Cycle - where water comes from and where it goes. From M.G. Spangler's "Soil Engineering".

Different specialist interests, such as meteorologists, oceanographers or agronomists, focus on different components of the cycle. From the point of view of the drainage engineer, the relevant part of the cycle can be represented in idealistic fashion by the block diagram of Figure 3.2.

Urbanization complicates that part of the hydrologic cycle which is affected by the modifications of natural drainage paths, impounding of water, diversion of storm water and the implementation of storm water management techniques.

The objective of this chapter is to introduce the drainage engineer to the methods of estimating precipitation and runoff, those components of the hydrologic cycle that affect design decisions. Emphasis is placed on the description of alternative methods for analyzing or simulating the rainfall-runoff process, particularly where these apply to comput-



■ **Figure 3.2** Block Diagram of Hydrologic Cycle.

er models. This should help the user of these models in determining appropriate data and in interpreting the results, thereby lessening the “black box” impression that users often have.

It is often necessary to describe many of these processes in mathematical terms. Every effort has been made to keep the presentation simple but some fundamental knowledge of hydrology has been assumed.

ESTIMATION OF RAINFALL

The initial data required for drainage design is a description of the rainfall. In most cases this will be a single event storm, i.e., a period of significant and continuous rainfall preceded and followed by a reasonable length of time during which no rainfall occurs. Continuous rainfall records extending many days or weeks may sometimes be used for the simulation of a series of storms, particularly where the quantity rather than the quality of runoff water is of concern.

The rainfall event may be either historic, taken from recorded events, or idealized. The main parameters of interest are the total amount (or depth) of precipitation (P_{tot}), the duration of the storm (t_d), and the distribution of the rainfall intensity (i) throughout the storm event. The frequency of occurrence (N) of a storm is usually expressed in years. It is an estimate based on statistical records of the long-term average time interval expected to elapse between successive occurrences of two storms of a particular severity. For example, a storm of depth P_{tot} with a duration of t_d is expected to occur, on average, every N years. The word "expected" is emphasized because there is absolutely no certainty that after a 25-year storm has occurred, a storm of equal or greater severity will not occur for another 25 years. This fact, while statistically true, is often difficult to convey to concerned or affected citizens.

Rainfall Intensity-Duration-Frequency Curves

Rainfall intensity-duration-frequency (IDF) curves are derived from the statistical analysis of rainfall records compiled over a number of years. Each curve represents the intensity-time relationship for a certain return frequency, from a series of storms. These curves are then said to represent storms of a specific return frequency.

The intensity, or the rate of rainfall, is usually expressed in depth per unit time. The frequency of occurrence (N), in years, is a function of the storm intensity. Larger storm intensities occur less frequently. The highest intensity for a specific duration of N years of records is called the N year storm, with a frequency of once in N years.

The curves may be in graphical form as shown in Figure 3.3, or may be represented by individual equations that express the time-intensity relationships for specific frequencies. Such equations are in the form

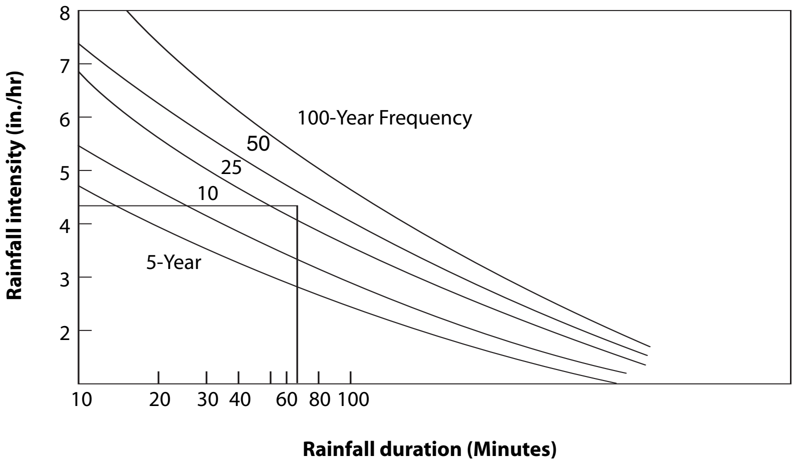
$$i = \frac{a}{(t + c)^b}$$

where: i = intensity (in./hr)
t = time (minutes)
a,b,c = constants developed for each IDF curve

The fitting of rainfall data to the equation may be obtained by either graphical or least square methods.

It should be noted that the IDF curves do not represent a rainfall pattern, but are the distribution of the highest intensities over time durations for a storm of N frequency.

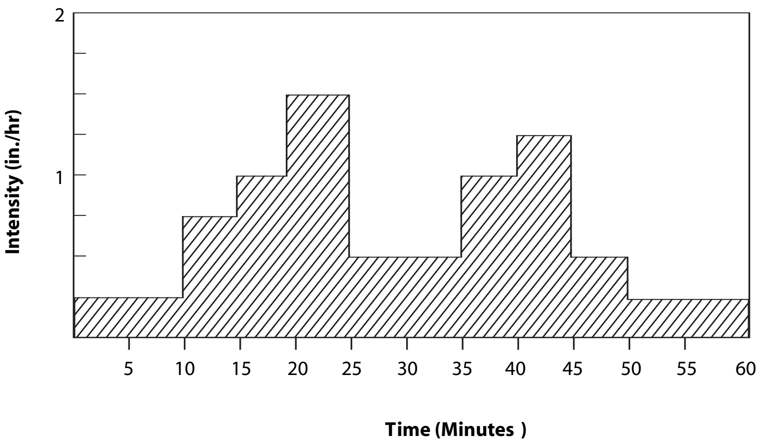
The rainfall intensity-duration-frequency curves are readily available from governmental agencies, local municipalities and towns, and are therefore widely used for designing drainage facilities and flood flow analysis.



■ **Figure 3.3** Rainfall intensties for various storm frequencies vs. rainfall duration.

Rainfall Hyetographs

The previous section discussed the dependence of the average rainfall intensity of a storm on various factors. It is also important to consider, from historical rainfall events, the way in which the precipitation is distributed in time over the duration of the storm. This can be described using a rainfall hyetograph which is a graphical representation of the variation of rainfall intensity with time. Rainfall hyetographs can be obtained (usually in tabular rather than graphical form) from weather stations that have suitable records of historical rainfall events. Figure 3.4 shows a typical example.



■ **Figure 3.4** Rainfall hyetograph.

Rainfall intensity is usually plotted in the form of a bar graph. It is therefore assumed that the rainfall intensity remains constant over the timestep (Figure 3.4) used to describe the hyetograph. This approximation becomes a truer representation of reality as the timestep gets smaller. However, very small timesteps may require very large amounts of data to represent a storm. At the other extreme, it is essential that the timestep not be too large, especially for short duration events or for very small catchments. Otherwise, peak values of both rainfall and runoff can be "smeared" with consequent loss of accuracy. This point should be kept in mind, when using a computer model, since it is standard practice to employ the same timestep for the description of the rainfall hyetograph and for the computation of the runoff hyetograph. Choice of a timestep is therefore influenced by:

a) accuracy of rainfall-runoff representation,
b) the number of available data points, and
c) size of the watershed.

Synthetic Rainfall Hyetographs

An artificial or idealized hyetograph may be required for a number of reasons, two of which are:

a) The historic rainfall data may not be available for the location or the return frequency desired.

b) It may be desirable to standardize the design storm to be used within a region so that comparisons of results from various studies may be made.

The time distribution of the selected design hyetograph will significantly affect the timing and magnitude of the peak runoff. Therefore, care should be taken in selecting a design storm to ensure that it is representative of the rainfall patterns in the area under study. In many cases, depending upon the size of the watershed and degree of urbanization, it may be necessary to use several different rainfall hyetographs to determine the sensitivity of the results to the different design storms. For example, when runoff from pervious areas is significant, it will be found that late peaking storms produce a higher peak runoff than early peaking storms of the same total depth. Early peaking storms are reduced in severity by the initially high infiltration capacity of the ground.

Selection of the storm duration will also influence the hyetograph characteristics. The handbook of the Natural Resource Conservation Service (formerly Soil Conservation Service) recommends that a six hour storm duration be used for watersheds with a time of concentration (which is discussed in detail later in this chapter) less than or equal to six hours. For watersheds where the time of concentration exceeds six hours, the storm duration should equal the time of concentration.

A number of different synthetic hyetographs are described in the following sections. These include:

a) uniform rainfall (as in the Rational Method),
b) the Chicago hyetograph,
c) Huff's rainfall distribution curves, and
d) SCS design storms.

Uniform Rainfall

The simplest possible design storm is to assume that the intensity is uniformly distributed throughout the storm duration. The intensity is then represented by

$$i = i_{ave} = \frac{P_{tot}}{t_d}$$

where: P_{tot} = total precipitation
t_d = storm duration

This simplified approximation is used in the Rational Method, assuming that the storm duration, td, is equal to the time of concentration, tc, of the catchment (see Figure 3.5). A rectangular rainfall distribution is only used for approximations or rough estimates. It can, however, have some use in explaining or visualizing rainfall-runoff processes, since any hyetograph may be considered as a series of such uniform, short duration pulses of rainfall.

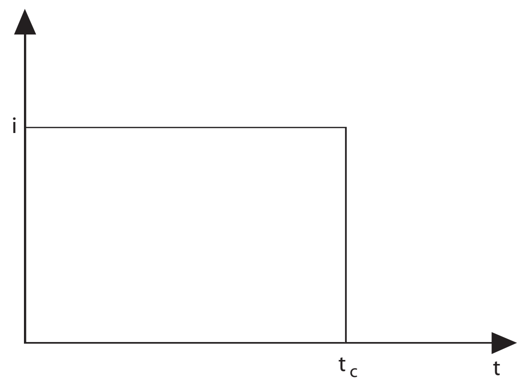


Figure 3.5 Uniform rainfall intensity.

Chicago Hyetograph

The Chicago hyetograph is assumed to have a time distribution such that if a series of ever increasing "time-slices" were analyzed around the peak rainfall, the average intensity for each "slice" would lie on a single IDF curve. Therefore, the Chicago design storm displays statistical properties which are consistent with the statistics of the IDF curve. That being the case, the synthesis of the Chicago hyetograph starts with the parameters of an IDF curve together with a parameter (r) which defines the fraction of the storm duration that occurs before the peak rainfall intensity. The value of r is derived from the analysis of actual rainfall events and is generally in the range of 0.3 to 0.5.

The continuous curves of the hyetograph in Figure 3.6 can be computed in terms of the times before (t_b) and after (t_a) the peak intensity by the two equations below.

After the peak:

$$i_a = \frac{a\left[(1-b)\dfrac{t_a}{1-r} + c\right]}{\left(\dfrac{t_a}{1-r} + c\right)^{1+b}}$$

Before the peak:

$$i_b = \frac{a\left[(1-b)\dfrac{t_b}{r} + c\right]}{\left(\dfrac{t_b}{r} + c\right)^{1+b}}$$

where: t_a = time after peak
t_b = time before peak
r = ratio of time before the peak occurs to the total duration time (the value is derived from analysis of actual rainfall events)

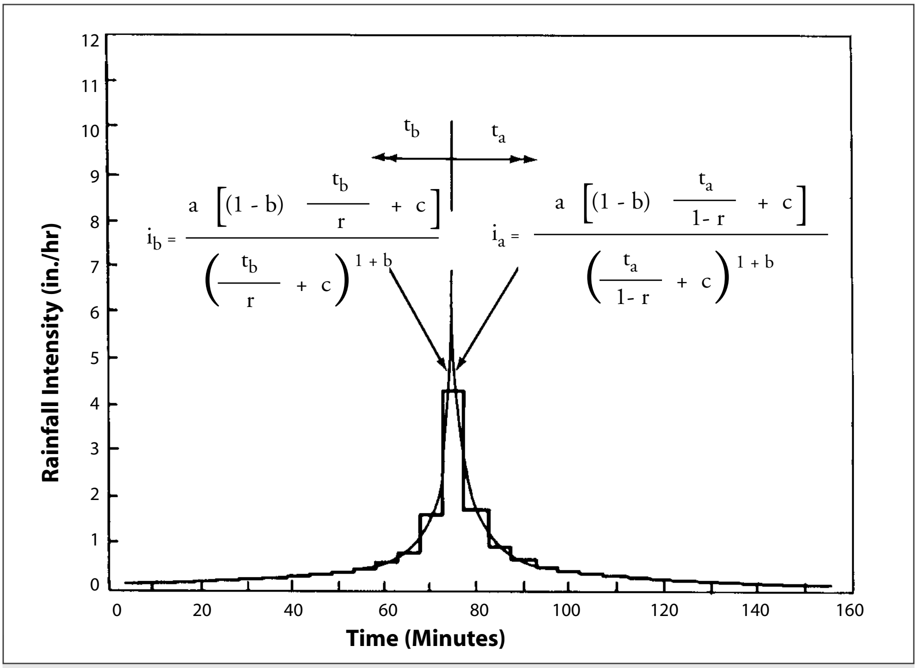


Figure 3.6 Chicago hyetograph.

The Chicago storm is commonly used for small to medium watersheds (0.1 to 10 square miles) for both rural and urban conditions. Typical storm durations are in the range of 1.0 to 4.0 hours. It has been found that peak runoff flows computed using a Chicago design storm are higher than those obtained using other synthetic or historic storms. This is due to the fact that the Chicago storm attempts to model the statistics of a large collection of real storms and thus tends to present an unrealistically extreme distribution. Also, the resultant peak runoff may exhibit some sensitivity to the timestep used; very small timesteps give rise to more peaked runoff hydrographs (which are discussed later in this chapter).

The Huff Rainfall Distribution Curves

Huff analyzed the significant storms in 11 years of rainfall data recorded by the State of Illinois. The data were represented in non-dimensional form by expressing the accumulated depth of precipitation, P_t, (that is, the accumulated depth at time t after the start of rainfall) as a fraction of the total storm depth, P_{tot}, and plotting this ratio as a function of a non-dimensional time, t/t_d, where t_d is time of duration.

The storms were grouped into four categories depending on whether the peak rainfall intensity fell in the 1st, 2nd, 3rd or 4th quartile of the storm duration. In each category, a family of curves was developed representing values exceeded in 90%, 80%, 70%, etc., of the storm events. Thus the average of all the storm events in a particular category is represented by the 50% curve. Table 3.1 shows the dimensionless coefficients for each quartile expressed at intervals of 5% of t_d.

Table 3.1

Dimensionless Huff storm coefficients

t / t_d	P_t / P_{tot} for Quartile			
	1	2	3	4
0.00	0.000	0.000	0.000	0.000
0.05	0.063	0.015	0.020	0.020
0.10	0.178	0.031	0.040	0.040
0.15	0.333	0.070	0.072	0.055
0.20	0.500	0.125	0.100	0.070
0.25	0.620	0.208	0.122	0.085
0.30	0.705	0.305	0.140	0.100
0.35	0.760	0.420	0.155	0.115
0.40	0.798	0.525	0.180	0.135
0.45	0.830	0.630	0.215	0.155
0.50	0.855	0.725	0.280	0.185
0.55	0.880	0.805	0.395	0.215
0.60	0.898	0.860	0.535	0.245
0.65	0.915	0.900	0.690	0.290
0.70	0.930	0.930	0.790	0.350
0.75	0.944	0.948	0.875	0.435
0.80	0.958	0.962	0.935	0.545
0.85	0.971	0.974	0.965	0.740
0.90	0.983	0.985	0.985	0.920
0.95	0.994	0.993	0.995	0.975
1.00	1.000	1.000	1.000	1.000

The first quartile curve is generally associated with relatively short duration storms in which 62% of the precipitation depth occurs in the first quarter of the storm duration. The fourth quartile curve is normally used for longer duration storms in which the rainfall is more evenly distributed over the duration t_d and is often dominated by a series of rain showers or steady rain or a combination of both. The third quartile has been found to be suitable for storms on the Pacific seaboard.

The study area and storm duration for which the distributions were developed vary considerably, with t_d varying from 3 to 48 hours and the drainage basin area ranging from 10 to 400 square miles. The distributions are most applicable to Midwestern regions of North America and regions of similar rainfall climatology and physiography.

To use the Huff distribution the user need only specify the total depth of rainfall (P_{tot}), the duration (t_d) and the desired quartile. The curve can then be scaled up to a dimensional mass curve and the intensities are obtained from the mass curve for the specified timestep (t).

SCS Design Storms

The U.S. Soil Conservation Service (SCS) design storm was developed for various storm types, storm durations and regions of the United States. The storm duration was initially selected to be 6 hours. Durations of 3 hours and up to 48 hours have, however, been

Table 3.2

SCS Type II rainfall distribution for 3h, 6h, 12h and 24h durations

3 Hour			6 Hour			12 Hour			24 Hour		
Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)
						0.5	1	1			
			0.5	2	2	1.0	1	2	2	2	2
						1.5	1	3			
0.5	4	4	1.0	2	4	2.0	1	4	4	2	4
						2.5	2	6			
			1.5	4	8	3.0	2	8	6	4	8
						3.5	2	10			
1.0	8	12	2.0	4	12	4.0	2	12	8	4	12
						4.5	3	15			
			2.5	7	19	5.0	4	19	10	7	19
						5.5	6	25			
1.5	58	70	3.0	51	70	6.0	45	70	12	51	70
						6.5	9	79			
			3.5	13	83	7.0	4	83	14	13	83
						7.5	3	86			
2.0	19	89	4.0	6	89	8.0	3	89	16	6	89
						8.5	2	91			
			4.5	4	93	9.0	2	93	18	4	93
						9.5	2	95			
2.5	7	96	5.0	3	96	10.0	1	96	20	3	96
						10.5	1	97			
			5.5	2	98	11.0	1	98	22	2	98
						11.5	1	99			
3.0	4	100	6.0	2	100	12.0	1	100	24	2	100

developed. The rainfall distribution varies depending on duration and location. The 3, 6, 12 and 24 hour distributions for the SCS Type II storm are given in Table 3.2. These distributions are used in all regions of the United States with the exception of the Pacific coast.

The design storms were initially developed for large (10 square mile) rural basins. However, the longer duration (6 to 48 hour) distributions and a shorter 1 hour duration thunderstorm distribution have been used in urban and smaller rural areas.

The longer duration storms tend to be used for sizing detention facilities while at the same time providing a reasonable peak flow for sizing the conveyance system.

ESTIMATION OF EFFECTIVE RAINFALL

Only a fraction of the precipitation that falls during a storm contributes to the overland flow or runoff from the catchment. The balance is diverted in various ways.

Evaporation In certain climates, some fraction of the rainfall evaporates before reaching the ground. Since rainfall is measured by gages on the earth's surface, this subtraction is automatically taken into account in recorded storms and may be ignored by the drainage engineer.

Interception This fraction is trapped in vegetation or roof depressions and never reaches the catchment surface. It eventually dissipates by evaporation.

Infiltration Rainfall that reaches a pervious area of the ground surface will initially be used to satisfy the capacity for infiltration into the upper layer of the soil. After even quite a short dry period the infiltration capacity can be quite large (for example, 4 in./hr) but this gradually diminishes after the start of rainfall as the storage capacity of the ground is saturated. The infiltrated water will:

a) evaporate directly by capillary rise,
b) rise through the root system and be transpired from vegetal cover where it then evaporates,
c) move laterally through the soil in the form of ground water flow toward a lake or a stream, and/or
d) penetrate to deeper levels to recharge the ground water.

Surface Depression Storage If the intensity of the rainfall reaching the ground exceeds the infiltration capacity of the ground, the excess will begin to fill the small depressions on the ground surface. Clearly this will begin to happen almost immediately on impervious surfaces. Only after these tiny reservoirs have been filled will overland flow commence and contribute to the runoff from the catchment. Since these surface depressions are not uniformly distributed, it is quite possible that runoff will commence from some fraction of the catchment area before the depression storage on another fraction is completely filled. Typical recommended values for surface depression storage are given in Table 3.3.

Table 3.3

Typical recommended values for depth of surface depression storage

Land Cover	Recommended Value (in.)
Large Paved Areas	0.1
Roofs, Flat	0.1
Fallow Land Field without Crops	0.2
Fields with Crops (grain, root crops)	0.3
Grass Areas in Parks, Lawns	0.3
Wooded Areas and Open Fields	0.4

The effective rainfall is thus that portion of the storm rainfall that contributes directly to the surface runoff hydrograph. This can be expressed as follows:

Runoff = Precipitation - Interception - Infiltration - Surface Depression Storage

All of the terms are expressed in units of depth.

A number of methods are available to estimate the effective rainfall and thus the amount of runoff for any particular storm event. These range from the runoff coefficient (C) of the Rational Method to relatively sophisticated computer implementations of semi-empirical methods representing the physical processes. The method selected should be based on the size of the drainage area, the data available and the degree of sophistication warranted for the design. Three methods for estimating effective rainfall are:

1) the Rational Method,
2) the Soil Conservation Service (SCS) Method, and
3) the Horton Method.

The Rational Method

If an impervious area (A) is subjected to continuous and long lasting rainfall of a specific intensity (i), then after a time (time of concentration, T_c) the runoff rate will be given by:

$$Q = k \cdot C \cdot i \cdot A$$

where:
Q = peak runoff rate (ft^3/s)
k = constant = 1.0 for U.S. Customary Units
C = runoff coefficient
i = rainfall intensity (in./hr)
A = drainage area (acres)

When using the Rational Method, the following assumptions are considered:

a) The rainfall intensity is uniform over the entire watershed during the entire storm duration.
b) The maximum runoff rate occurs when the rainfall lasts as long or longer than the time of concentration.
c) The time of concentration is the time required for the runoff from the most remote part of the watershed to reach the point under design.

The variable C is the component of the Rational Method that requires the most judgment, and the runoff is directly proportional to the value assigned to C. Care should be exercised in selecting the value as it incorporates all of the hydrologic abstractions, soil types and antecedent conditions. Table 3.4 lists typical values for C, as a function of land use, for storms that have (approximately) a 5 to 10 year return period. It is important to note that the appropriate value of C depends on the magnitude of the storm. Significantly higher values of C may be necessary for more extreme storm events. This is perhaps one of the most serious deficiencies associated with this method.

Table 3.4

Recommended runoff coefficients based on description of area

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

It often is desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area. Coefficients, with respect to surface type, are shown in Table 3.5.

Table 3.5

Recommended runoff coefficients based on character of surface

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The coefficients in these two tables are applicable for storms of 5 to 10 year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

The SCS Method

Referred to here as the SCS Method, the Natural Resource Conservation Service (formerly Soil Conservation Service) developed a relationship between rainfall (P), retention (S), and effective rainfall or runoff (Q). The retention, or potential storage in the soil, is established by selecting a curve number (CN). The curve number is a function of soil type, ground cover and Antecedent Moisture Condition (AMC).

The hydrological soil groups, as defined by SCS soil scientists, are:

A. (Low runoff potential) Soils having a high infiltration rate, even when thoroughly wetted, consisting chiefly of deep, well to excessively well drained sands or gravel.

B. Soils having a moderate infiltration rate when thoroughly wetted, consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse texture.

C. Soils having a slow infiltration rate when thoroughly wetted, consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture.

D. (High runoff potential) Soils having a very slow infiltration rate when thoroughly wetted, consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Knowing the hydrological soil group and the corresponding land use, the runoff potential or CN value of a site may be determined. Table 3.6 lists typical CN values. Three levels of Antecedent Moisture Condition are considered in the SCS Method. The Antecedent Moisture Condition (AMC) is defined as the amount of rainfall in a period of five to thirty days preceding the design storm. In general, the heavier the antecedent rainfall, the greater the runoff potential. AMC definitions are as follows:

AMC I - Soils are dry but not to the wilting point. This is the lowest runoff potential.

AMC II - This is the average case, where the soil moisture condition is considered to be average.

AMC III - Heavy or light rainfall and low temperatures having occurred during the previous five days. This is the highest runoff potential.

Table 3.6

Runoff curve numbers for selected agricultural, suburban and urban land use (Antecedend Moisture Condition II and I_a = 0.2 S)

LAND USE DESCRIPTION			HYDROLOGIC SOIL GROUP A	B	C	D
Cultivated land[1]:	without conservation treatment		72	81	88	91
	with conservation treatment		62	71	78	81
Pasture or range land:	poor condition		68	79	86	89
	good condition		39	61	74	80
Meadow:	good condition		30	58	71	78
Wood or forest land:	thin stand, poor cover, no mulch		45	66	77	83
	good cover[2]		25	55	70	77
Open spaces, lawns, 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
Commercial and business areas (85% impervious)			89	92	94	95
Industrial districts (72% impervious)			81	88	91	93
Residential[3]:	Average lot size	Average % Impervious[4]				
	1/8 acre	65	77	85	90	92
	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
Paved parking lots, roofs, driveways, etc.[5]			98	98	98	98
Streets and roads:	paved with curbs and storm sewers5		98	98	98	98
	gravel		76	85	89	91
	dirt		72	82	87	89

1. For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug 1972.
2. Good cover is protected from grazing and litter and brush cover soil.
3. Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
4. The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
5. In some warmer climates of the country, a curve number of 95 may be used.

The CN values in Table 3.6 are based on Antecedent Moisture Condition II. Thus, if moisture conditions I or III are chosen, then a corresponding CN value is determined as provided in Table 3.7.

Table 3.7

Curve number relationships for different Antecedent Moisture Conditions

CN for	CN for		CN for	CN for	
Condition II	Condition I	Condition III	Condition II	Condition I	Condition III
100	100	100	60	40	78
99	97	100	59	39	77
98	94	99	58	38	76
97	91	99	57	37	75
96	89	99	56	56	75
95	87	98	55	35	74
94	85	98	54	34	73
93	83	98	53	33	72
92	81	97	52	32	71
91	80	97	51	31	70
90	78	96	50	31	70
89	76	96	49	30	69
88	75	95	48	29	68
87	73	95	47	28	67
86	72	94	46	27	66
85	70	94	45	26	65
84	68	93	44	25	64
83	67	93	43	25	63
82	66	92	42	24	62
81	64	92	41	23	61
80	63	91	40	22	60
79	62	91	39	21	59
78	60	90	38	21	58
77	59	89	37	20	57
76	58	89	36	19	56
75	57	88	35	18	55
74	55	88	34	18	54
73	54	87	33	17	53
72	53	86	32	16	52
71	52	86	31	16	51
70	51	85	30	15	50
69	50	84			
68	48	84	25	12	43
67	47	83	20	9	37
66	46	82	15	6	30
65	45	82	10	4	22
64	44	81	5	2	13
63	43	80	0	0	0
62	42	79			
61	41	78			

The potential storage in the soils is based on an initial abstraction (I_a) which is the interception, infiltration and depression storage prior to runoff, and infiltration after runoff.

The effective rainfall is defined by the relationship:

$$Q = \frac{(P - I_a)^2}{P + S - I_a}$$

where $S = [(100/CN) - 10] \bullet 25.4$

The original SCS Method assumed the value of I_a to be equal to 0.2S. However, many engineers have found that this may be overly conservative, especially for moderate rainfall events and low CN values. Under these conditions, the I_a value may be reduced to be a lesser percentage of S or may be estimated and input directly into the above equation.

The Horton Method

The Horton infiltration equation, which defines the infiltration capacity of the soil, changes the initial rate (f_o) to a lower rate (f_c). The "maximum infiltration capacity" infiltration rate, calculated using the equation, only occurs when the available rainfall equals or exceeds that infiltration rate. Until that occurs, the infiltration rate is the rainfall intensity. Therefore, if the infiltration capacity is given by:

$$f_{cap} = f_c + (f_o - f_c)\, e^{-t \bullet k}$$

then the actual infiltration (f), will be defined by one of the following two equations:

$$f = f_{cap} \quad \text{for } i \geq f_{cap}$$

$$f = i \quad \text{for } i \leq f_{cap}$$

where:
- f = actual infiltration rate into the soil
- f_{cap} = maximum infiltration capacity of the soil
- f_o = initial infiltration capacity
- f_c = final infiltration capacity
- i = rainfall intensity
- k = exponential decay constant (1/hours)
- t = elapsed time from start of rainfall (hours)

Figure 3.7 shows a typical rainfall distribution and infiltration curve.

For the initial timesteps, the infiltration rate exceeds the rainfall rate. The reduction in infiltration capacity is dependent more on the reduction in storage capacity in the soil rather than the elapsed time from the start of rainfall. To account for this, the infiltration curve should, therefore, be shifted (dashed line for first timestep, Δt) by an elapsed time that would equate the infiltration volume to the volume of runoff.

A further modification is necessary if surface depression is to be accounted for. Since the storage depth must be satisfied before overland flow can occur, the initial finite values of the effective rainfall hyetograph must be reduced to zero until a depth equivalent to the surface depression storage has been accumulated. The final hyetograph is the true effective rainfall that will generate runoff from the catchment surface.

The selection of the parameters for the Horton equation depends on soil type, vegetal cover and Antecedent Moisture Conditions. Table 3.8 shows typical values for f_o and f_c

(in./hour) for a variety of soil types under different crop conditions. The value of the lag constant should typically be between 0.04 and 0.08.

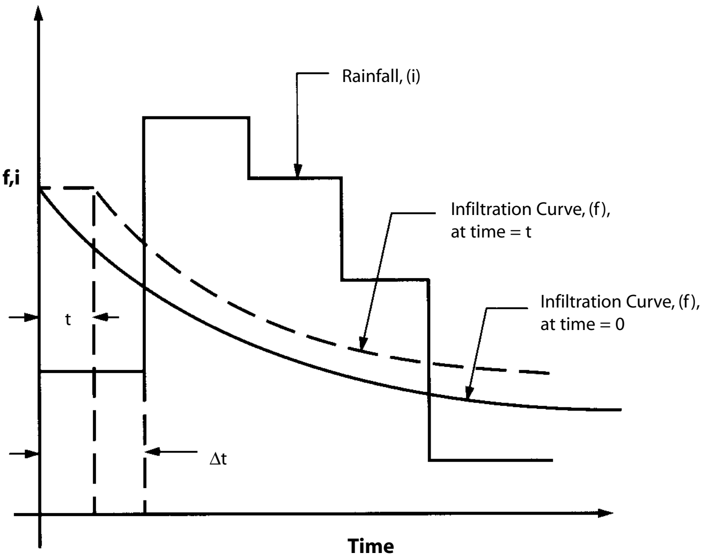


Figure 3.7 Representation of the Horton equation.

Table 3.8

Typical values for the Horton equation parameters (in./hr)

	Loam, Clay K = 0.08		Clayey Sand K = 0.06		Sand, Loess, Gravel K = 0.04	
Land Surface Types	f_o	f_c	f_o	f_c	f_o	f_c
Fallow land field without crops	15	8	33	10	43	15
Fields with crops (grain, root crops, vines)	36	3	43	8	64	10
Grassed verges, playground, ski slopes	20	3	20	3	20	3
Noncompacted grassy surface, grass areas in parks, lawns	43	8	64	10	89	18
Gardens, meadows, pastures	64	10	71	15	89	18
Coniferous woods	53*	53*	71*	71*	89*	89*
City parks, woodland, orchards	89	53	89	71	89*	89*

Notes: *K=0

Comparison of the SCS and Horton Methods

Figure 3.8 illustrates the various components of the rainfall-runoff process for the SCS and Horton Methods. The following example serves to demonstrate the difference between the SCS Method, in which the initial abstraction is used, and the moving curve Horton Method, in which surface depression storage is significant.

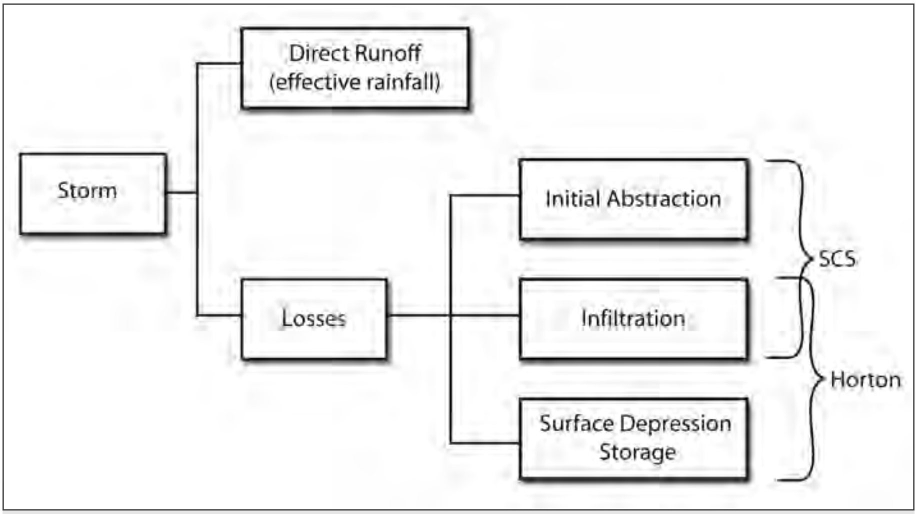


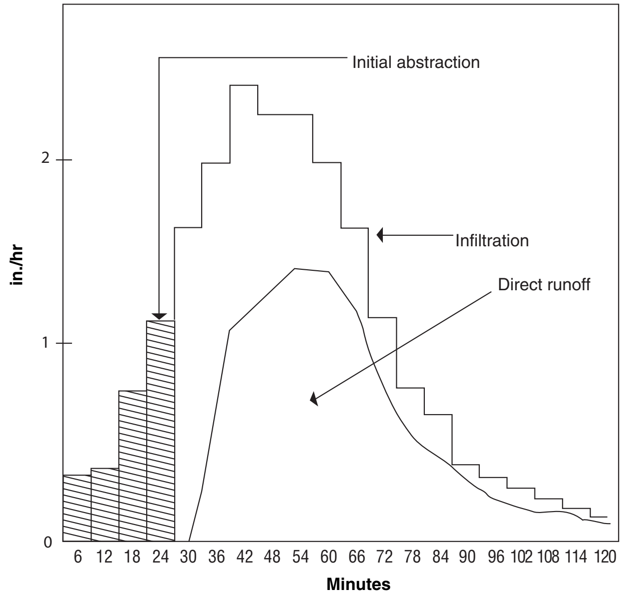
Figure 3.8 Conceptual components of rainfall.

The incident storm is assumed to be represented by a second quartile Huff curve with a total depth of 2 inches and a duration of 120 minutes.

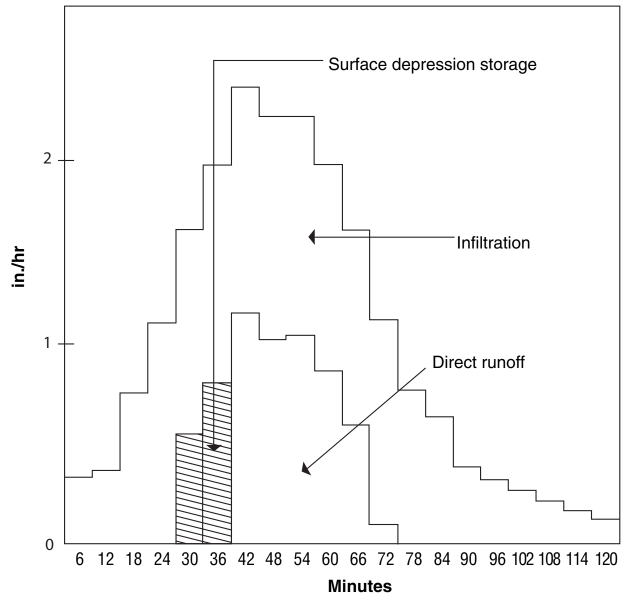
In one case the SCS Method is used with the initial abstraction set at an absolute value of I_a = 0.24 in. The curve number used is 87.6. Figure 3.9 shows that no runoff occurs until approximately 30 minutes have elapsed, at which time the rainfall has satisfied the initial abstraction. From that point, however, the runoff, although small, is finite and continues to be so right to the end of the storm.

The Horton case is tested using values of f_o = 1.2 in./hr, f_c = 0.4 in./hr, K = 0.25 hour, and a surface depression storage depth of 0.2 inches. These values have been found to give the same volumetric runoff coefficient as the SCS parameters. Figure 3.10 shows that infiltration commences immediately and absorbs all of the rainfall until approximately 30 minutes have elapsed. The initial excess surface water has to fill the surface depression storage which delays the commencement of runoff for a further 13 minutes. After 72 minutes the rainfall intensity is less than f_c and runoff is effectively stopped at that time.

It will be found that the effective rainfall hyetograph generated using the Horton equation has more leading and trailing “zero” elements, so that the effective hyetograph is shorter but more intense than that produced using the SCS Method.



■ **Figure 3.9** SCS Method with I_a = 0.24 in. and CN = 87.6.



■ **Figure 3.10** Horton equation with f_o = 1.18 in./hr, f_c = 0.39 in./hr, K = 0.25, and surface depression storage = 0.2 inches.

ESTABLISHING THE TIME OF CONCENTRATION

Apart from the area and the percentage of impervious surface, one of the most important characteristics of a catchment is the time that must elapse until the entire area is contributing to runoff at the outflow point. This is generally called the time of concentration (T_c). This time is comprised of two components:

1) The time for overland flow to occur from a point on the perimeter of the catchment to a natural or artificial drainage conduit or channel.
2) The travel time in the conduit or channel to the outflow point of the catchment.

In storm sewer design, the time of concentration may be defined as the inlet time plus travel time. Inlet times used in sewer design generally vary from 5 to 20 minutes, with the channel flow time being determined from pipe flow equations.

Factors Affecting Time of Concentration

The time taken for overland flow to reach a conduit or channel depends on a number of factors:

a) Overland flow length (L). This should be measured along the line of longest slope from the extremity of the catchment to a drainage conduit or channel. Long lengths result in long travel times.
b) Average surface slope (S). Since T_c is inversely proportional to S, care must be exercised in estimating an average value for the surface slope.
c) Surface roughness. In general, rough surfaces result in longer travel times and smooth surfaces result in shorter travel times. Therefore, if a Manning equation is used to estimate the velocity of overland flow, T_c will be proportional to the Manning roughness factor (n).
d) Depth of overland flow (y). Very shallow surface flows move more slowly than deeper flows. However, the depth of flow is not a characteristic of the catchment alone but depends on the intensity of the effective rainfall and surface moisture excess.

Several methods of estimating the time of concentration are described below. Since it is clear that this parameter has a strong influence on the shape of the runoff hydrograph, it is desirable to compare the value to that obtained from observation, if possible. In situations where sufficient historical data is not available, it may help to compare the results obtained by two or more methods. The impact on the resultant hydrograph, due to using different methods for establishing the time of concentration, should then be assessed.

The Kirpich Formula

This empirical formula relates T_c to the length and average slope of the basin by the equation:

$$T_c = 0.000128\, L^{0.77}\, S^{-0.385} \quad \text{(See Figure 3.11)}$$

where:
T_c = time of concentration (hours)
L = maximum length of water travel (ft)
S = surface slope, given by H/L (ft/ft)
H = difference in elevation between the most remote point on the basin and the outlet (ft)

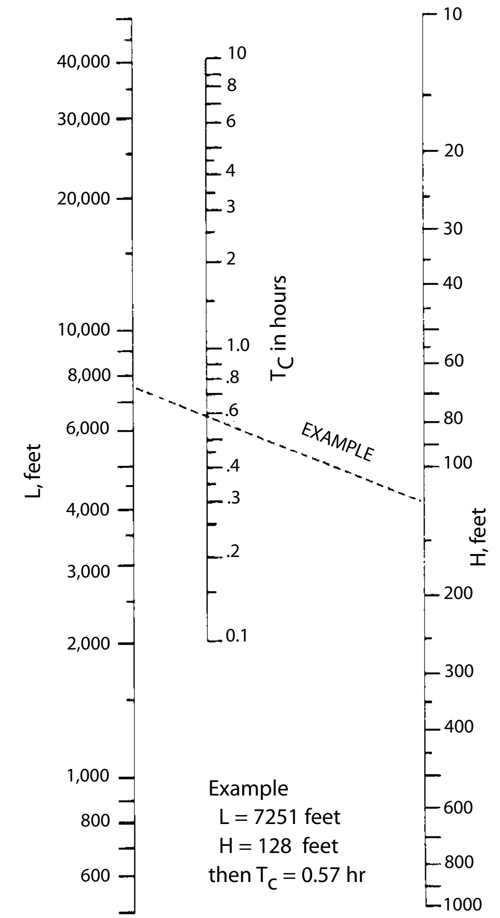


Figure 3.11 T_c nomograph using the Kirpich formula.

From the definition of L and S it is clear that the Kirpich formula combines both the overland flow, or entry time, and the travel time in the channel or conduit. It is, therefore, particularly important that in estimating the drop (H), the slope (S) and ultimately the time of concentration (T_c), an allowance, if applicable, be made for the inlet travel time.

The Kirpich formula is normally used for natural basins with well defined routes for overland flow along bare earth or mowed grass roadside channels. For overland flow on grassed surfaces, the value of T_c obtained should be doubled. For overland flow in concrete channels, a multiplier of 0.2 should be used.

For large watersheds, where the storage capacity of the basin is significant, the Kirpich formula tends to significantly underestimate T_c.

The Uplands Method

When calculating travel times for overland flow in watersheds with a variety of land covers, the Uplands Method may be used. This method relates the time of concentration to the basin slope, basin length and type of ground cover. Times are calculated for individual areas, with their summation giving the total travel time.

A velocity is derived using the $V/S^{0.5}$ values from Table 3.9 and a known slope. The time of concentration is obtained by dividing the length by the velocity.

Table 3.9

$V/S^{0.5}$ Relationship for Various Land Covers

Land Cover	$V/S^{0.5}$ (ft/s)
Forest with Heavy Ground Litter, Hay Meadow (overland flow)	2.0
Trash Fallow or Minimum Tillage Cultivation; Contour, Strip Cropped, Woodland (overland flow)	5.0
Short Grass Pasture (overland flow)	7.5
Cultivated, Straight Row (overland flow)	9.0
Nearly Bare and Untilled (overland flow) or Alluvial Fans in Western Mountain Regions	10.0
Grassed Waterway	15.0
Paved Areas (sheet flow); Small Upland Gullies	20.0

A graphical solution can be obtained from Figure 3.12. However, it should be noted that the graph is simply a log-log plot of the values of $V/S^{0.5}$ given in Table 3.9.

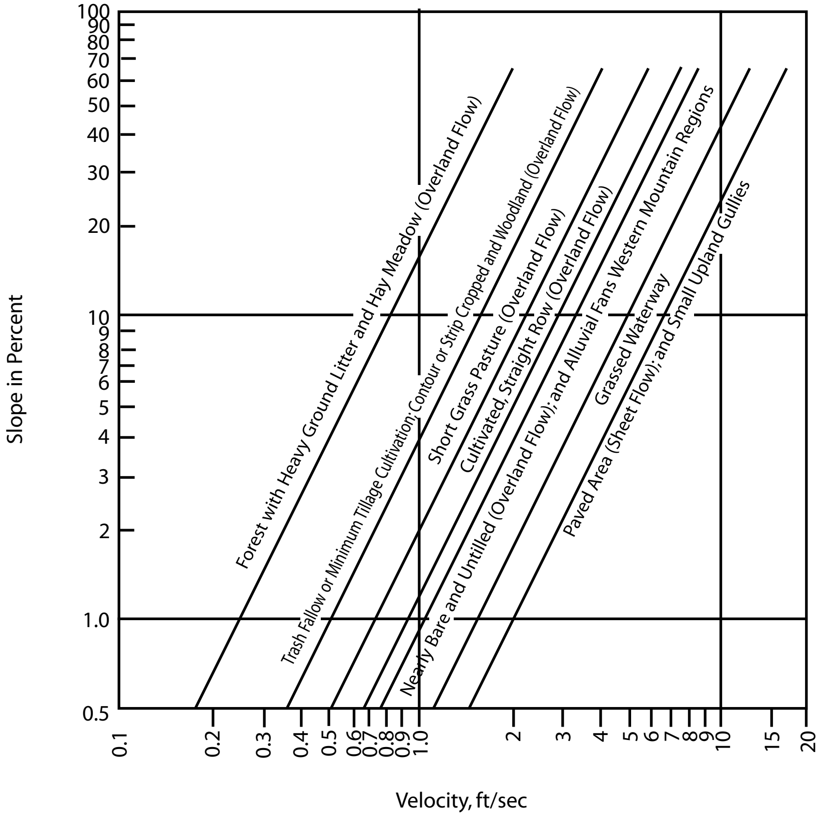


Figure 3.12 Velocities for Uplands Method for estimating travel time for overland flow.

The Kinematic Wave Method

The two methods described above have the advantage of being quite straightforward and may be used for either simple or more complex methods of determining the runoff. Apart from the empirical nature of the equations, the methods assume that the time of concentration is independent of the depth of overland flow, or more generally, the magnitude of the input. A method in common use, which is more physically based and which also reflects the dependence of T_c on the intensity of the effective rainfall, is the Kinematic Wave Method.

The method was proposed to analyze the kinematic wave resulting from rainfall of uniform intensity on an impermeable plane surface or rectangular area. The resulting equation is as follows:

$$T_c = 0.116\,(L \bullet n/S)^{0.6}\, i_{eff}^{\;-0.4}$$

where:
- T_c = time of concentration (hr)
- L = length of overland flow (ft)
- n = Manning's roughness coefficient
- S = average slope of overland flow (ft/ft)
- i_{eff} = effective rainfall intensity (in./hr)

Other Methods

Other methods have been developed to determine T_c for specific geographic regions or basin types. These methods are often incorporated into an overall procedure for determining the runoff hydrograph. Before using any method, the user should ensure that the basis on which the time of concentration is determined is appropriate for the area under consideration.

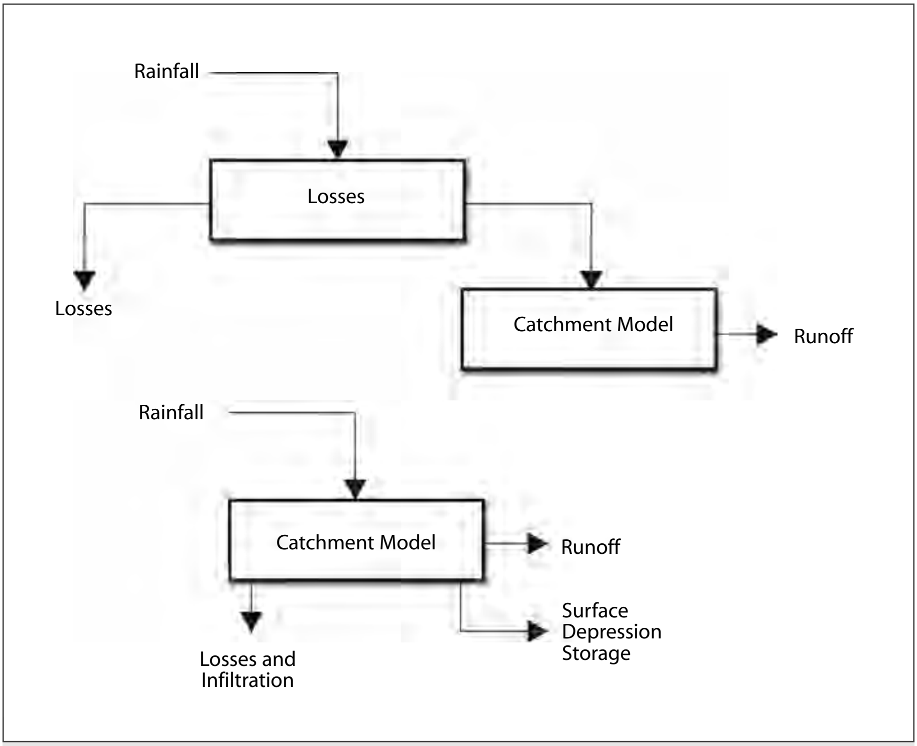
DETERMINATION OF THE RUNOFF HYDROGRAPH

The following sections outline alternative methods for generating the runoff hydrograph, which is the relationship of discharge over time. Emphasis will be given to establishing the hydrograph for single storm events. Methods for estimating flow for urban and rural conditions are given.

Irrespective of the method used, the results should be compared to historical values wherever possible. In many cases, a calibration/validation exercise will aid in the selection of the most appropriate method.

All of the methods described could be carried out using hand calculations. However, for all but the simplest cases the exercise would be very laborious. Furthermore, access to computers and computer models is very economical. For these reasons, emphasis will be placed on describing the basis of each method and the relevant parameters. A subsequent section will relate the methods to several computer models.

Rainfall-runoff models may be grouped into two general classifications, which are illustrated in Figure 3.13.



■ **Figure 3.13** Classification of rainfall-runoff models:
Effective Rainfall (top) and Surface Water Budget (bottom).

One approach uses the concept of effective rainfall, in which a loss model is assumed that divides the rainfall intensity into losses (initial infiltration and depression storage) and effective rainfall. The effective rainfall hyetograph is then used as input to a catchment model to produce a runoff hydrograph. It follows from this approach that infiltration must stop at the end of the storm.

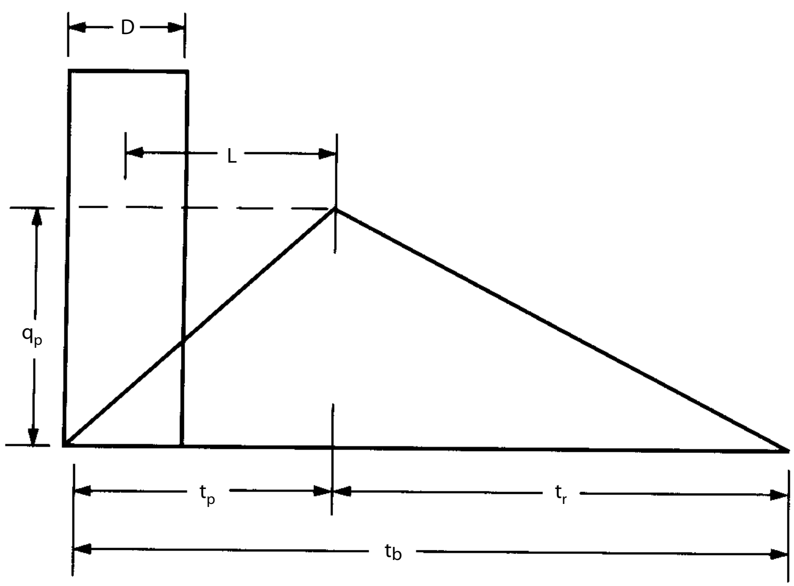
The alternative approach employs a surface water budget in which the infiltration or loss mechanism is incorporated into the catchment model. In this method, the storm rainfall is used as input and the estimation of infiltration and other losses is an integral part of the runoff calculation. This approach implies that infiltration will continue as long as there is excess water on the surface. Clearly, this may continue after the rainfall ends.

SCS Unit Hydrograph Method

A unit hydrograph represents the runoff distribution over time for one unit of rainfall excess over a drainage area for a specified period of time. This method assumes that the ordinates of flow are proportional to the volume of runoff from any storm of the same duration. Therefore, it is possible to derive unit hydrographs for various rainfall blocks by convoluting the unit hydrograph with the effective rainfall distribution. The unit hydrograph theory is based on the following assumptions:

1. For a given watershed, runoff-producing storms of equal duration will produce surface runoff hydrographs with approximately equivalent time bases, regardless of the intensity of the rain.
2. For a given watershed, the magnitude of the ordinates representing the instantaneous discharge from an area will be proportional to the volumes of surface runoff produced by storms of equal duration.
3. For a given watershed, the time distribution of runoff from a given storm period is independent of precipitation from antecedent or subsequent storm periods.

The U.S. Natural Resource Conservation Service (formerly Soil Conservation Service), based on the analysis of a large number of hydrographs, proposed a unit hydrograph that only requires an estimate of the time to peak (t_p). Two versions of this unit hydrograph were suggested; one being curvilinear in shape, while the other is a simple asymmetric triangle as shown in Figure 3.14. The SCS has indicated that the two hydrographs give very similar results as long as the time increment is not greater than 0.20 •T_c.



■ **Figure 3.14** SCS triangular unit hydrograph.

The following parameters must be determined to define the triangular unit hydrograph; the time to peak (t_p), the peak discharge corresponding to 1 inch of runoff (q_p), and the base time of the hydrograph (t_b).

Once these parameters are determined, the unit hydrograph can be applied to a runoff depth or to a series of runoff depths. When applied to a series of runoff depths, sub-hydrographs are developed for each and summed to provide an overall hydrograph. A series of runoff depths, for instance, may be a sequence of runoff depths such as those values obtained from a hyetograph where excess rainfall is that portion of the rainfall that is runoff, calculated as the rainfall adjusted to account for retention losses.

The lag time (L) is the delay between the center of the excess rainfall period (D) and the peak of the runoff (t_p). The SCS has suggested that the lag time, for an average watershed and fairly uniform runoff, can be approximated by:

$$L \approx 0.6\, T_c$$

The estimate of the time to peak (t_p) is therefore affected by the time of concentration (T_c) and the excess rainfall period (D). It is calculated using the relationship:

$$t_p = 0.5\, D + 0.6\, T_c$$

where T_c may be determined by any acceptable method such as those described in the previous section.

For a series of runoff depths, where the timestep used is Δt, the parameter D should be replaced by Δt in the above equation, so that it becomes:

$$t_p = 0.5\, \Delta t + 0.6\, T_c$$

The duration of the recession limb of the hydrograph is assumed to be $t_r = (5/3)\, t_p$ so that the time base is given by $t_b = (8/3)\, t_p$.

The ordinates of the unit hydrograph are expressed in units of discharge per unit depth of runoff. In terms of the notation used in Figure 3.14:

$$q_p = 484\, A/t_p$$

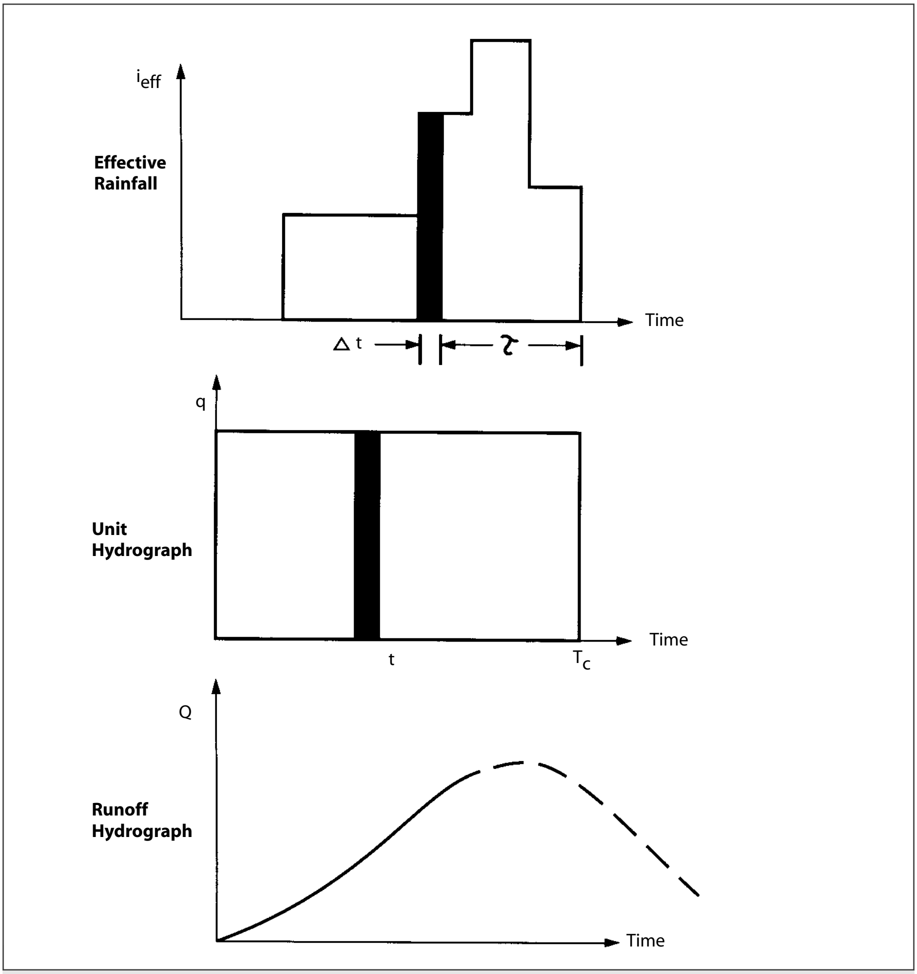
where: q_p = peak discharge, ft^3/s per inch of runoff
A = catchment area, sq. mi.
t_p = time to peak, hours

The numerical constant in the above equation is a measure of the watershed characteristics. This value varies between about 300 for flat marshy catchments and 600 for steep flashy catchments. A value of 484 is recommended by the SCS for average watersheds.

From the above equation it can be seen that the time to peak (t_p), and therefore the peak discharge of the unit hydrograph (q_p), is affected by the value of the excess rainfall period (D) and, in the case of a series of runoff depths, the timestep used (Δt). Values of D or Δt in excess of 0.25 t_p should not be used as this can lead to the underestimation of the peak runoff.

Rectangular Unit Hydrograph Method

An alternative option to the triangular distribution used in the SCS Method is the rectangular unit hydrograph. Figure 3.15 illustrates the concept of convoluting the effective rainfall with a rectangular unit hydrograph. The ordinate of the unit hydrograph is defined as the area of the unit hydrograph divided by the time of concentration (T_c).



■ **Figure 3.15** Convolution process using a rectangular unit hydrograph.

The Rational Method is often used for a rough estimate of the peak flow. This method, which assumes the peak flow occurs when the entire catchment surface is contributing to runoff, may be simulated using a rectangular unit hydrograph. The effective rainfall hydrograph is reduced to a simple rectangular function and $i_{eff} = k \bullet C \bullet i$. The effective rainfall, with duration t_d, is convoluted with a rectangular unit hydrograph that has a base equal to the time of concentration (T_c). If t_d is made equal to T_c, the resultant runoff hydrograph will be symmetrical and triangular in shape with a peak flow given by $Q = k \bullet C \bullet i \bullet A$ and a time base of $t_b = 2\ T_c$. If the rainfall duration (t_d) is not equal to T_c, then the resultant runoff hydrograph is trapezoidal in shape with a time base of $t_b = t_d + T_c$ and a peak flow given by the following

$$Q = k \bullet C \bullet i \bullet A\ (t_d / T_c) \qquad \text{for } td \le T_c$$

$$Q = k \bullet C \bullet i \bullet A \qquad \text{for } td > T_c$$

This approach makes no allowance for the storage effect due to the depth of overland flow and results in an "instantaneous" runoff hydrograph. This may be appropriate for impervious surfaces in which surface depression storage is negligible, but for pervious or more irregular surfaces it may be necessary to route the instantaneous hydrograph through a hypothetical reservoir in order to more closely represent the runoff hydrograph.

Linear Reservoir Method

Pederson suggested a more complex response function in which the shape of the unit hydrograph is assumed to be the same as the response of a single linear reservoir to an inflow having a rectangular shape and duration Δt. A linear reservoir is one in which the storage (S) is linearly related to the outflow (Q) by the formula:

$$S = K \bullet Q$$

where: K = the reservoir lag or storage coefficient (hours)

In Pederson's method, the value of K is taken to be $0.5\ T_c$ where T_c is computed from the kinematic wave equation in which the rainfall intensity used is the maximum for the storm being modeled. The use of i_{max} is justified since this intensity tends to dominate the subsequent runoff hydrograph. The resulting unit hydrograph is illustrated in Figure 3.16 and comprises a steeply rising limb, which reaches a maximum at time t = Δt, followed by an exponential recession limb. The two curves can be described by the following

$$q_p = \frac{(1 - e^{-\Delta t/K})}{\Delta t} \quad \text{at } t = \Delta t$$

$$q = q_p \bullet e^{-(t-\Delta t)/K} \quad \text{for } t > \Delta t$$

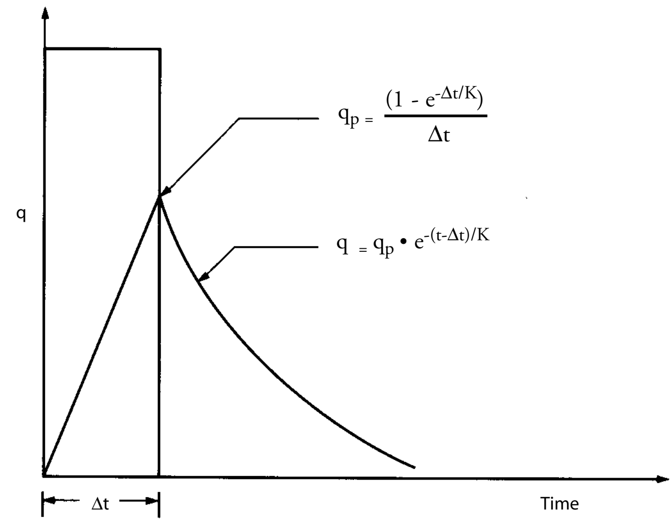


Figure 3.16 The single linear reservoir.

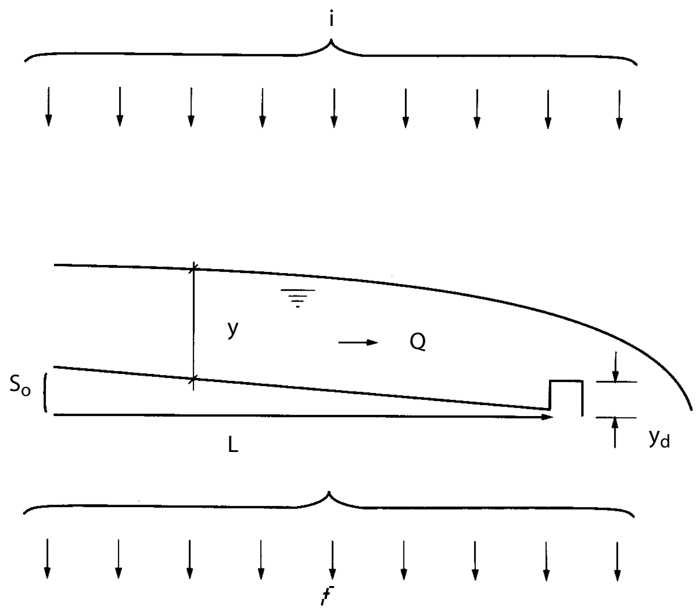
An important feature of the method is that the unit hydrograph always has a time to peak of Δt and is incapable of reflecting different response times as a function of catchment length, slope or roughness. It follows that the peak of the runoff hydrograph will usually be close to the time of peak rainfall intensity, irrespective of the catchment characteristics.

SWMM Runoff Algorithm

The Storm Water Management Model was originally developed for the U.S. Environmental Protection Agency in 1971. Since then it has been expanded and improved by the EPA and many other agencies and companies. In particular, the capability for continuous simulation has been included (in addition to the original ability to handle single event simulation), quality as well as quantity is simulated, and snow-melt routines are included in some versions.

The model is intended for use in urban or partly urban catchments. It comprises five main "blocks" of code in addition to an Executive Block or supervisory calling program. Following is a description of the basic algorithm of the Runoff Block, which is used to generate the runoff hydrograph in the drainage system based on a rainfall hyetograph, Antecedent Moisture Conditions, land use and topography.

The method differs from those described previously in that it does not use the concept of effective rainfall, but employs a surface water budget approach. In that approach, rainfall, infiltration, depression storage and runoff are all considered as processes occurring simultaneously at the land surface. The interaction of these inputs and outputs may be visualized with reference to Figure 3.17.



■ **Figure 3.17** Representation of the SWMM/Runoff algorithm.

Treating each sub-catchment as an idealized, rectangular plane surface having a breadth (B) and length (L), the continuity or mass balance equation at the land surface is given by:

Inflow = (Infiltration + Outflow) + Rate of Surface Ponding

This may be expressed as

$$i \bullet L \bullet B = (f \bullet L \bullet B + Q) + L \bullet B \bullet (\Delta y/\Delta t)$$

where: i = rainfall intensity
f = infiltration rate
Q = outflow
y = depth of flow over the entire surface

The depth of flow (y) is computed using the Manning equation, taking into account the depth of surface depression storage (y_d) which is also assumed to be uniform over the entire surface. The dynamic equation is given by:

$$Q = B\ (1/n)\ (y - y_d)^{5/3}\ S^{1/2}$$

where: n = Manning's roughness coefficient for overland flow
S = average slope of the overland flow surface

The infiltration rate (f) must be computed using a method such as the 'moving curve' Horton equation or the Green-Ampt model. Infiltration is assumed to occur as long as excess surface moisture is available from rainfall, depression storage or finite overland flow.

It is important to note that the value of Manning's "n" used for overland flow is somewhat artificial (for example, in the range of 0.1 to 0.4) and does not represent a value that might be used for channel flow calculation.

Various methods can be used for the simultaneous solution of the continuity and dynamic equations. One method is to combine the equations into one nondifferential equation in which the depth (y) is the unknown. Once the depth is determined (for instance, by an interactive scheme such as the Newton-Raphson Method) the outflow (Q) follows.

COMPUTER MODELS

Many computer models have been developed for the simulation of the rainfall/runoff process. Table 3.10 lists several of these models and their capabilities.

Table 3.10

Hydrologic computer models

Model Characteristics	HEC-1	HYMO	HSPF	ILLUDAS	MIDUSS	OTTHYMO	QUALHYMO	SCS TR-20	SCS TR-55	SSARR	STANFORD	STORM	SWMM	USDAHL-74
Model Type:														
Single Event	●	●		●	●	●		●	●				●	
Continuous			●				●			●	●	●	●	●
Model Components:														
Infiltration	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Evapotranspiration			●							●	●	●	●	●
Snowmelt	●		●							●	●	●	●	●
Surface Runoff	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Subsurface Flow	●		●				●			●	●			●
Reservoir Routing	●	●	●		●	●	●	●	●	●			●	
Channel Routing	●	●	●	●	●	●	●	●	●	●	●		●	
Water Quality			●				●					●	●	
Application:														
Urban Land Use			●	●	●	●	●		●			●	●	
Rural Land Use	●	●	●		●	●	●	●		●	●	●		●
Ease of Use:														
High	●	●		●	●	●	●	●	●			●		●
Low			●							●	●		●	

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Deep corrugated structural arch during high flow.

Round pipe multiple opening installation.