Chapter 4 - Urban Hydrologic Procedures

This chapter provides an overview of hydrologic methods and procedures commonly used in urban highway drainage design. The chapter condenses information from *Highway Hydrology*, Hydraulic Design Series 2 (HDS-2) (FHWA 2002). The chapter introduces the methods and procedures, their data demands, and their limitations. Most of these procedures can be applied using commonly available computer programs. HDS-2 contains additional information and detail on the methods described.

4.1 Rainfall (Precipitation)

Rainfall, along with watershed characteristics, determines the flows for storm drainage design. The following sections describe three representations of rainfall the designer can use to derive flood flows: uniform rainfall intensity, variable rainfall intensity, and synthetic design storm events.

4.1.1 Uniform Rainfall Intensity

Although rainfall intensity varies during precipitation events, many of the procedures that designers use to derive peak flow (see Section 4.2) are based on an assumed uniform rainfall intensity. Intensity is the rate of rainfall and is typically given in units of inches per hour (millimeters per hour).

Federal, State, and local agencies have developed Intensity-Duration-Frequency (IDF) curves throughout the United States through frequency analysis of rainfall events for thousands of rainfall gages. The IDF curve, as illustrated in Figure 4.1, provides a summary of a site's rainfall characteristics by relating storm duration and exceedance probability (frequency) to rainfall intensity (assumed constant over the duration). To interpret an IDF curve, find the rainfall duration along the horizontal axis, go vertically up the graph until reaching the proper return period, then go horizontally to the left and read the intensity off the vertical axis. Most highway agency drainage manuals contain regional IDF curves and NOAA Atlas 14 provides a national source of IDF curves.

4.1.2 Variable Rainfall Intensity (Hyetograph)

Rainfall intensity in any given storm varies over time. Figure 4.2 depicts a mass rainfall curve showing the accumulation of rainfall during an actual or design storm. The instantaneous intensity is the slope of the mass rainfall curve at a particular time. For hydrologic analysis, designers typically divide the storm into convenient time increments and determine the average intensity over each of the selected periods. The designer then plots these results as a rainfall hyetograph, like that presented in Figure 4.2.

Hyetographs provide greater flexibility in creating design storms than a uniform rainfall intensity by specifying the precipitation variability over time. Designers use them in conjunction with hydrographic (rather than peak flow) methods. In storm drain design, hyetographs are relevant for volumetric applications such as storage routing of hydrographs. Hyetographs allow for simulation of actual rainfall events and calibration of hydrologic models, which can provide valuable information on the relative flood risks of different events. The National Climatic Data Center (NCDC) at NOAA often makes available hyetographs of actual storms.

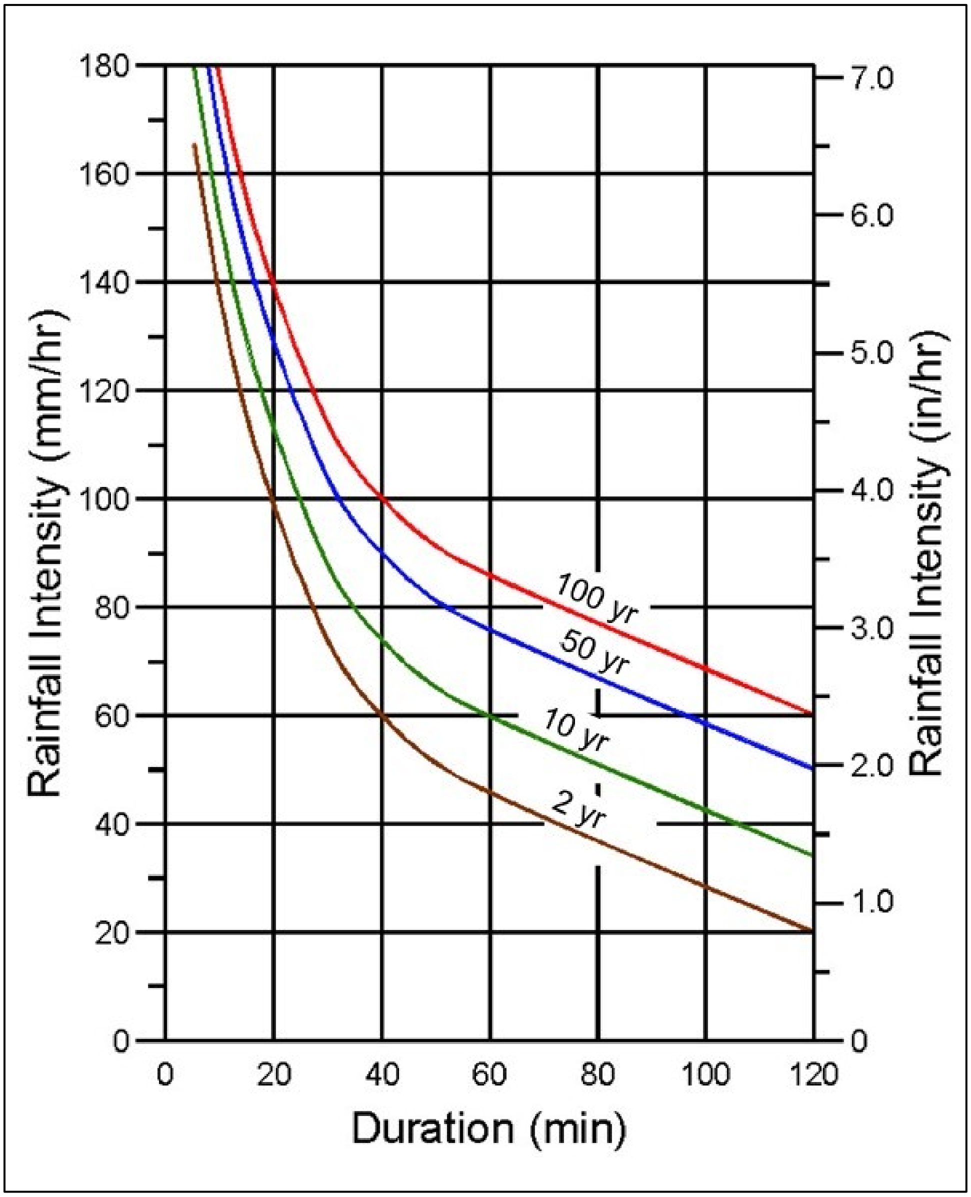

Figure 4.1. Example IDF curves.

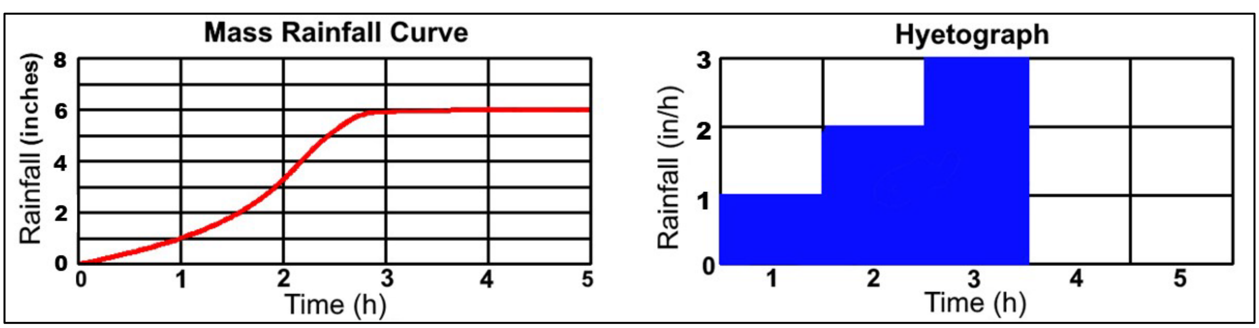

Figure 4.2. Example mass rainfall curve and corresponding hyetograph.

4.1.3 Synthetic Design Storm Events

Designers typically base drainage design on synthetic, rather than actual, rainfall events. The U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS), developed and documented 24-hour rainfall distributions, which is described in HDS-2. The SCS 24-hour rainfall distributions are widely used synthetic hyetographs and incorporate the intensity-duration relationship for the design AEP. This approach assumes that the maximum rainfall for any duration within the 24-hour duration has the same AEP. For example, a 0.1 AEP, 24-hour design storm contains the 0.1 AEP rainfall depths for all durations up to 24 hours as derived from IDF curves. Other sources of rainfall distributions exist including NOAA Atlas 14.

4.2 Peak Flow

Peak flows are generally adequate for design and analysis of conveyance systems such as storm drains or open channels. This section discusses methods used to derive peak flows for both gaged and ungaged sites. The NRCS (SCS) peak flow method is another approach that calculates peak flow as a function of drainage basin area, potential watershed storage, and the time of concentration. This rainfall-runoff methodology separates total rainfall into direct runoff, retention, and initial abstraction. HDS-2 provides more detailed discussion on this method.

4.2.1 Statistical Analysis

Designers use statistical analysis to evaluate peak flows where adequate gaged streamflow data exist. Frequency distributions, used in the analysis of hydrologic data, include the normal distribution, the log-normal distribution, the Gumbel extreme value distribution, and the log-Pearson type III distribution. The log-Pearson type III distribution is a three-parameter gamma distribution with a logarithmic transform of the independent variable. Designers use it widely for flood analyses because the data frequently fit the assumed population. This flexibility led the United States Geological Survey (USGS) to recommend its use as the standard distribution for flood frequency studies by all U.S. Government agencies, as documented in Bulletin 17C (England et al. 2019). Figure 4.3 presents an example of a log-Pearson type III distribution frequency curve (FHWA 2002). Designers do not commonly use statistical analysis methods in urban drainage design due to the lack of adequate streamflow data. Consult HDS-2 (FHWA 2002) for additional information on these methods.

4.2.2 Rational Method

One of the most used approaches for the calculation of peak flow from small areas is the Rational Method, given as:

$$Q = \frac{CIA}{K_u} \tag{4.1}$$

where:

Q	=	Flow, ft^3/s (m^3/s)
C	=	Dimensionless runoff coefficient
I	=	Rainfall intensity, in/h (mm/h)
A	=	Drainage area, ac (ha)
K_u	=	Unit conversion constant, 1.0 in CU (360 in SI)

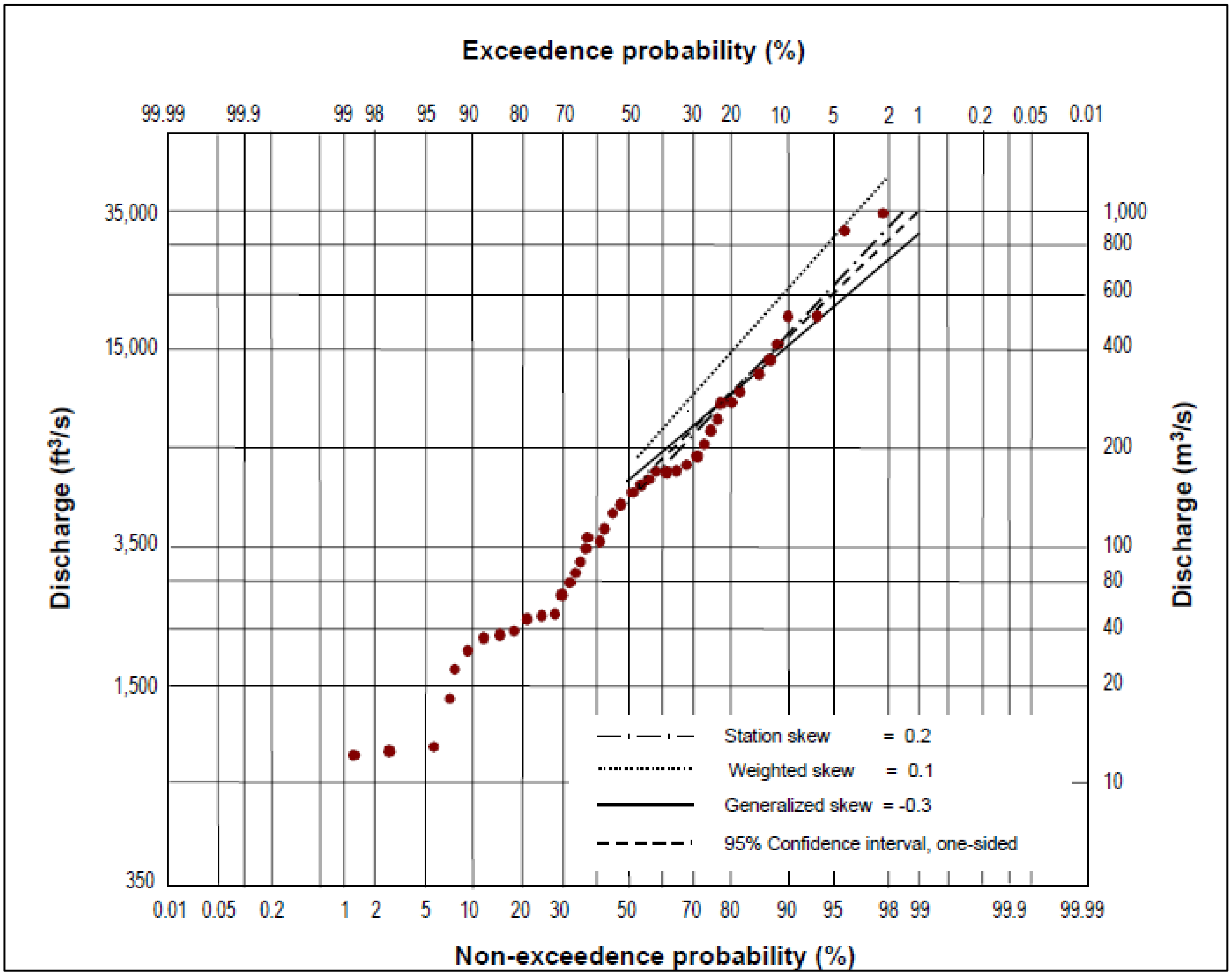

Figure 4.3. Log-Pearson type III distribution analysis, Medina River, Texas.

As described in HDS-2 (FHWA 2002), the Rational Method, assumes:

- Peak flow occurs when the entire watershed contributes to the flow.
- Rainfall intensity is the same over the entire drainage area.
- Rainfall intensity is uniform over a time duration equal to the time of concentration, t_c. (The time of concentration is the time water takes to travel from the hydraulically most remote point of the basin to the point of interest.)

Rational Method

Emil Kuichling, City Engineer of Rochester, New York, developed the Rational Method in 1889. The methodology is based rainfall-runoff observations of small urban watersheds ranging from 25 to 357 acres. The assumptions emerge from his findings.

- Frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 0.1 AEP rainfall intensity produces the 0.1 AEP peak flow.
- The coefficient of runoff is the same for all storms of all recurrence probabilities.

Related to the last assumption, designers sometimes use a frequency-of-event correction factor as a modifier to the Rational Method runoff coefficient. Some agencies recommend use of this coefficient, but FHWA does not endorse its use. The intent of the correction factor is to compensate for the reduced effect of infiltration and other hydrologic abstractions during less frequent, higher intensity storms. The frequency-of-event correction factor is multiplied times the runoff coefficient, C, to produce an adjusted runoff coefficient.

Because of the inherent assumptions, HDS-2 recommends application of the Rational Method to drainage areas smaller than 200 ac (80 ha) and provides additional information on the Rational Method (FHWA 2002).

4.2.2.1 Runoff Coefficient

The runoff coefficient, C, in equation 4.1 is a function of the ground cover and other characteristics that influence hydrologic abstraction. Table 4.1 summarizes typical values for C. If the basin contains varying amounts of different land cover or other abstractions, a weighted coefficient can be calculated through areal weighting as follows (FHWA 2002):

$$C = \sum \frac{C_x A_x}{A_{total}} \tag{4.2}$$

where:

x = Subscript designating values for incremental areas with consistent land cover

Example 4.1: Calculation of the runoff coefficient.

Objective: Estimate weighted runoff coefficient, C, for existing and proposed conditions.

Given: The following existing and proposed land uses:

Existing conditions (unimproved):

Land Use	Area (ac)	Runoff Coefficient
Unimproved Area	22.1	0.25
Grass	21.2	0.22
Total	43.3	-

Proposed conditions (improved):

Land Use	Area (ac)	Runoff Coefficient
Paved	5.4	0.90
Lawn	1.6	0.15
Unimproved Area	18.6	0.25
Grass	17.7	0.22
Total	43.3	-

Step 1. Determine weighted C for existing (unimproved) conditions.

Using equation 4.1:

Weighted $C = \Sigma (C_x A_x) / A = [(22.1)(0.25) + (21.2)(0.22)] / 43.3 = 0.235$

Step 2. Determine weighted C for proposed (improved) conditions.

Using equation 4.1:

Weighted C = [(5.4) (0.90) + (1.6) (0.15) + (18.6) (0.25) + (17.7) (0.22)] / 43.3 = 0.315

Solution: The weighted runoff coefficients, C, for existing and proposed conditions are 0.235 and 0.315, respectively.

Table 4.1. Runoff coefficients for the Rational Method (ASCE 1960).

Land Use Category	Type of Drainage Area	Runoff Coefficient, C*
Business	Downtown areas	0.70 - 0.95
	Neighborhood areas	0.50 - 0.70
Residential	Single-family areas	0.30 - 0.50
	Multi-units, detached	0.40 - 0.60
	Multi-units, attached	0.60 - 0.75
	Suburban	0.25 - 0.40
	Apartment dwelling areas	0.50 - 0.70
Industrial	Light areas	0.50 - 0.80
	Heavy areas	0.60 - 0.90
Open	Parks, cemeteries	0.10 - 0.25
	Playgrounds	0.20 - 0.40
	Railroad yard areas	0.20 - 0.40
	Unimproved areas	0.10 - 0.30
Lawns	Sandy soil, flat, 2%	0.05 - 0.10
	Sandy soil, average, 2 - 7%	0.10 - 0.15
	Sandy soil, steep, 7%	0.15 - 0.20
	Heavy soil, flat, 2%	0.13 - 0.17
	Heavy soil, average, 2 - 7%	0.18 - 0.22
	Heavy soil, steep, 7%	0.25 - 0.35
Streets	Asphaltic	0.70 - 0.95
	Concrete	0.80 - 0.95
	Brick	0.70 - 0.85
Other impervious	Drives and walks	0.75 - 0.85
	Roofs	0.75 - 0.95

*Higher values are generally appropriate for steeply sloped areas and less frequent AEPs because infiltration and other losses have a proportionally smaller effect on runoff in these cases.

4.2.2.2 Rainfall Intensity

The Rational Method uses rainfall intensity, duration, and frequency curves. Federal, State, and local agencies have developed IDF curves for locations across the country and State highway agency drainage manuals typically document those applicable within their jurisdiction. NOAA and NRCS have created regional rainfall intensity curves that can also be used to create IDF relationships for design.

4.2.2.3 Time of Concentration

Designers use many methods to estimate time of concentration including the velocity or segment method. The velocity method calculates the flow velocity within individual segments of the flow path, e.g., sheet flow, shallow concentrated flow, and open channel flow. The time of concentration can be calculated as the sum of the travel times within the various consecutive flow segments. For additional discussion on establishing the time of concentration for inlets and drainage systems, see Section 9.2.2 of this manual.

Sheet flow is the shallow runoff on a planar surface with a uniform depth across the sloping surface. This usually occurs at the headwater of streams over relatively short distances, rarely more than about 300 ft, but most often less than 100 ft (NRCS 2010). Ragan (1971) suggests sheet flow occurs for distances 72 feet or less. Designers commonly estimate sheet flow with a version of the kinematic wave equation (FHWA 2002):

$$t_t = \frac{K_u}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8} \tag{4.3}$$

where:

t_t	=	Sheet flow travel time, min
n	=	Roughness coefficient
L	=	Flow length, ft (m)
P_2	=	2-year, 24-hour rainfall depth, inches (mm)
S	=	Surface slope, ft/ft (m/m)
K_u	=	Unit conversion constant, 0.42 in CU (5.5 in SI)

Table 4.2 summarizes Manning's roughness coefficients. Equation 4.3 is the modified version of the sheet flow equation. An iterative version of the equation replaces rainfall depth with rainfall intensity (FHWA 2002).

Shallow concentrated flow develops as sheet flow concentrates in rills and then gullies of increasing proportions. Designers can estimate the velocity of such flow using a relationship between velocity and slope, as described in HDS-2 (FHWA 2002) as follows:

$$V = K_u k S_p^{0.5} \tag{4.4}$$

where:

V	=	Velocity, ft/s (m/s)
k	=	Intercept coefficient
S_p	=	Slope, percent
K_u	=	Unit conversion constant, 3.28 in CU (1.0 in SI)

Table 4.3 summarizes intercept coefficients for shallow concentrated flow.

Table 4.2. Manning's roughness coefficient (n) for overland sheet flow (FHWA 2002).

Surface Description	n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipe	0.024
Cement rubble surface	0.024
Fallow (no residue)	0.05
Cultivated soils: Residue cover # 20%	0.06
Cultivated soils: Residue cover > 20%	0.17
Cultivated soils: Range (natural)	0.13
Short grass prairie	0.15
Dense grasses	0.24
Bermuda grass	0.41
Woods: Light underbrush *	0.40
Woods: Dense underbrush *	0.80

*When selecting n, consider cover to a height of about 1 inch (30 mm). This is only part of the plant cover that will obstruct sheet flow.

Table 4.3. Intercept coefficients for velocity vs. slope relationship (FHWA 2002).

Land Cover/Flow Regime	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow); alluvial fans in western mountain regions	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

Open channel and pipe flow occurs when water further collects in gullies, channels, or pipes. Such channels can be identified when they are shown on maps or are visible on aerial photographs. Velocity calculations typically use cross-section geometry and roughness for all channel reaches in the watershed. Designers commonly use Manning's equation to estimate average flow velocities in pipes and open channels as follows:

$$V = \frac{K_u}{n} R^{2/3} S^{1/2} \tag{4.5}$$

where:

n	=	Roughness coefficient
V	=	Velocity, ft/s (m/s)
R	=	Hydraulic radius (measured as the flow area divided by the wetted perimeter), ft (m)
S	=	Slope, ft/ft (m/m)
K_u	=	Unit conversion constant, 1.49 in CU (1 in SI)

Table 4.4 provides representative values of Manning's roughness coefficient for channels and pipes. For a circular pipe flowing full, the hydraulic radius equals one-fourth of the diameter.

Table 4.4. Typical range of Manning's coefficient (n) for channels and pipes.

Conduit Category	Conduit Material	Manning's n *
Closed conduits	Concrete pipe	0.010 - 0.015
	CMP	0.011 - 0.037
	Plastic pipe (smooth)	0.009 - 0.015
	Plastic pipe (corrugated)	0.018 - 0.025
Pavement/gutter sections	Concrete, asphalt	0.012 - 0.016
Small open channels	Concrete	0.011 - 0.015
	Rubble or riprap	0.020 - 0.035
	Vegetation	0.020 - 0.150
	Bare soil	0.016 - 0.025
	Rock cut	0.025 - 0.045
Natural channels/streams (top width at flood stage less than 100 ft (30 m)	Fairly regular section	0.025 - 0.050
	Irregular section with pools	0.040 - 0.150

*Lower values usually apply to well-constructed and maintained (smoother) pipes and channels.

For a wide rectangular channel (W > 10 d), the hydraulic radius approximately equals the depth. To calculate the travel time:

$$t = \frac{L}{60V} \tag{4.6}$$

where:

t	=	Travel time for a given segment, min
L	=	Flow length for the given segment, ft (m)
V	=	Velocity for the given segment, ft/s (m/s)

Example 4.2: Calculation of time of concentration.

Objective: Estimate time of concentration, t_c, for the given area.

Given: The following flow path characteristics:

Flow Segment	Length (ft)	Slope (ft/ft)	Segment Description
1 (sheet flow)	223	0.010	Bermuda grass
2 (shallow conc.)	259	0.006	Grassed waterway
3 (flow in conduit)	479	0.008	15-inch (380 mm) concrete pipe

The 2-yr 24-h rainfall depth is 4.35 inches.

Step 1. Calculate travel times for each segment, starting at the downstream end, using the 0.1 AEP (10-year) IDF curve.

Step 1a. Calculate travel time for segment 1.

Obtain Manning's n roughness coefficient from Table 4.2:

n = 0.41

Determine the sheet flow travel time using equation 4.3:

$$t_t = \frac{K_u}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8} = \frac{0.42}{(4.35)^{0.5}}\left(\frac{0.41(223)}{\sqrt{0.01}}\right)^{0.8} = 47.1\,\text{min}\ \ (\text{use 47 minutes})$$

Step 1b. Calculate travel time for segment 2.

Obtain intercept coefficient, k, from Table 4.3: k = 0.457 and K_u = 3.281

Determine the concentrated flow velocity from equation 4.4:

$V = K_u\, k\, S_p^{0.5} = (3.281)\,(0.457)\,(0.6)^{0.5} = 1.16$ ft/s

Determine the travel time from equation 4.6:

$t_{t2} = L / (60\ V) = 259/[(60)(1.16)] = 3.7$ min

Step 1c. Calculate travel time for segment 3.

Obtain Manning's n roughness coefficient from Table 4.3: n = 0.011

Determine the pipe flow velocity from equation 4.5 (assuming full flow):

$V = (1.49 / 0.011)\,(1.25 / 4)^{0.67}\,(0.008)^{0.5} = 5.58$ ft/s

Determine the travel time:

$t_{t3} = L/(60\ V) = 479/[(60)(5.58)] = 1.4$ min

Step 2. Determine the total travel time by summing the individual travel times.

$t_c = t_{t1} + t_{t2} + t_{t3} = 47.1 + 3.7 + 1.4 = 52.2$ min; use 52 min

Solution: The estimated time of concentration is 52 minutes.

Example 4.3: Rational Method peak flow.

Objective: Estimate 0.1 AEP (10-year) peak flow using the Rational Method and the IDF curve shown in Figure 4.1.

Given: Land use conditions from example 4.1 and the following times of concentration:

Condition	Time of concentration, t_c (min)	Weighted C (from example 4.1)
Existing (unimproved)	88	0.235
Proposed (improved)	66	0.315

Area = 43.36 ac (17.55 ha)

Step 1. Determine rainfall intensity, I, from the 0.1 AEP IDF curve for each time of concentration.

Rainfall intensity:

Existing condition (unimproved) 1.9 in/h

Proposed condition (improved) 2.3 in/h

Step 2. Determine peak flow rate, Q.

Existing condition (unimproved):

$Q = CIA / K_u = (0.235)(1.9)(43.3) / 1 = 19.3$ ft^3/s

Proposed condition (improved):

$Q = CIA / K_u = (0.315)(2.3)(43.3) / 1 = 31.4$ ft^3/s

Solution: The existing and proposed condition peak flows are 19.3 ft^3/s (0.55 m^3/s) and 31.4 ft^3/s (89 m^3/s), respectively.

4.2.3 USGS Regression Equations

Designers commonly use regression equations to estimate peak flows at ungaged sites or sites with limited data. The USGS has developed and compiled regional regression equations which are included in a computer program called the National Streamflow Statistics program (NSS) and in StreamStats. NSS allows quick and easy estimation of peak flows throughout the United States (Reis 2007). Local equations may also be available. HDS-2 provides additional information on regression equations (FHWA 2002).

4.2.3.1 Rural Equations

The rural equations are based on watershed and climatic characteristics within specific regions of each State that can be obtained from topographic maps, rainfall reports, and atlases. These regression equations generally take the form:

$$RQ_T = a\, A^b B^c\, C^d \quad (4.7)$$

where:

RQ_T	=	T-year rural peak flow, ft^3/s (m^3/s)
a	=	Regression constant
b, c, d	=	Regression coefficients
A, B, C	=	Basin or meteorological characteristics

The USGS, State highway, and other agencies conducted a series of studies to develop rural equations for all States. These equations do not apply where dams and other hydrologic modifications have a significant effect on peak flows. HDS-2 presents other limitations (FHWA 2002).

4.2.3.2 Urban Equations

Designers can adapt a rural peak flow for urban conditions with the three-parameter or seven-parameter nationwide urban regression equations developed by USGS. NSS can calculate peak flows with both versions. The USGS urban equations are based on urban runoff data from 269 basins in 56 cities and 31 States and validated at 78 additional sites in the southeastern United States. The equations provide reasonable estimates of peak flows with recurrence intervals between 2 and 500 years (Sauer et al. 1983). The USGS has quantified the accuracy of the urban equations, like the rural equations, with standard errors that are in the range of 35 to 50 percent when compared to site-specific estimates from gage records. HDS-2 (FHWA 2002) provides more detail on urban regression equations.

4.3 Design Hydrographs

This section discusses methods used to develop a design hydrograph. Application of hydrograph methods often necessitates computer programs such as the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), the Stormwater Management Model (SWMM), WinTR-20, WinTR-55, among others, to generate runoff hydrographs. Practitioners perform hydrographic analyses when flow routing is important such as in the design of stormwater detention, other water quality facilities, and pump stations. Large storm drainage systems might also use hydrographic analyses to evaluate flow routing and more precisely reflect flow peaking conditions in each segment of complex systems. HDS-2 contains additional information on hydrographic methods (FHWA 2002).

4.3.1 Unit Hydrograph Methods

A unit hydrograph is the direct runoff hydrograph resulting from a unit volume of excess rainfall. The unit volume is 1 inch in CU and 1 millimeter in SI system. Although the unit volume is given in units of depth, users of the method call it a volume because it is that unit depth applied over the area of the watershed. Depth times area is volume.

The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is also equal to that unit volume, as described in HDS-2 (FHWA 2002). As do many methods, application of a unit hydrograph assumes uniform rainfall intensity and duration over the entire watershed. Therefore, the modeler chooses a design storm (rainfall intensity and duration) suited for the size of watershed under consideration. Additionally, storm movement can affect the runoff characteristics of the watershed. Storms moving down long and narrow watersheds produce a higher peak runoff rate and a longer time to peak. Acknowledging these assumptions, the drainage area limitation of unit hydrograph applications is a maximum of

1,000 mi^2 (FHWA 2002). This section discusses the NRCS dimensionless unit hydrograph and Snyder unit hydrograph methods.

4.3.1.1 NRCS Dimensionless Unit Hydrograph

The NRCS developed a synthetic unit hydrograph procedure widely used in their conservation and flood control work. This unit hydrograph is based upon an analysis of many natural unit hydrographs from a broad range of geographic locations and hydrologic regions. To construct the standard NRCS unit hydrograph, the designer estimates the peak flow and the time to peak.

For the development of the NRCS Unit Hydrograph, the curvilinear unit hydrograph is approximated by a triangular unit hydrograph with similar characteristics. Figure 4.4 compares the two dimensionless unit hydrographs. Even though the time base of the triangular unit hydrograph is 8/3 of the time to peak and the time base of the curvilinear unit hydrograph is five times the time to peak, the area under the two unit hydrograph types is the same.

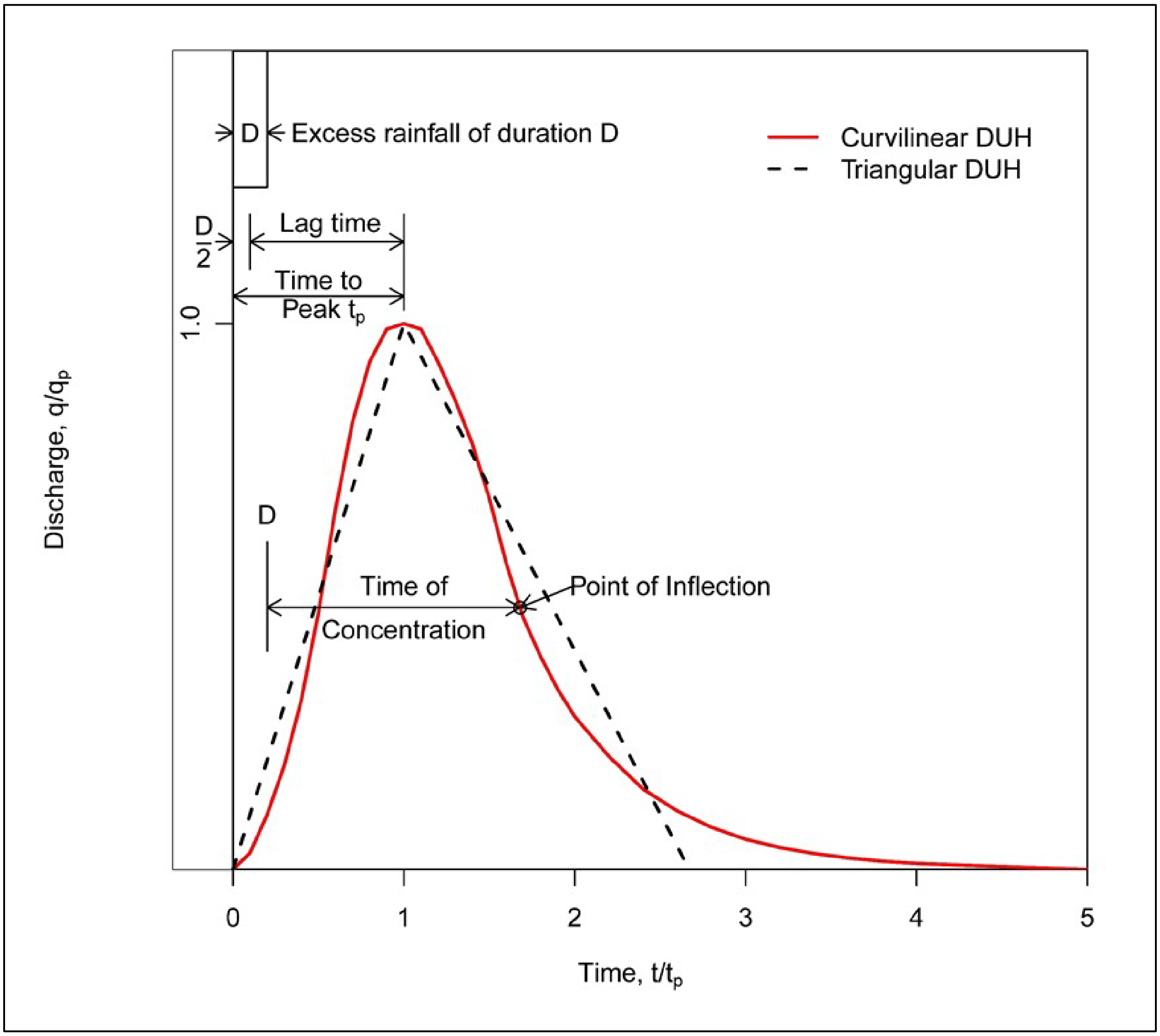

Figure 4.4. Dimensionless curvilinear NRCS unit hydrograph and equivalent triangular hydrograph.

The area under a hydrograph equals the volume of direct runoff, Q_D, which is 1 inch (millimeter) for a unit hydrograph. To calculate the peak flow:

$$q_p = \frac{K_u K_p A Q_D}{t_p} \tag{4.8}$$

where:

q_p	=	Peak flow, ft^3/s (m^3/s)
A	=	Drainage area, mi^2 (km^2)
Q_D	=	Volume of direct runoff (= 1 for unit hydrograph), inch (mm)
t_p	=	Time to peak, h
K_p	=	Peaking constant equal to 484, dimensionless
K_u	=	Unit conversion constant, 1 in CU (0.0043 in SI)

The peaking constant reflects a unit hydrograph with 3/8 of its area under the rising limb. For mountainous watersheds, the fraction could be expected to be greater than 3/8, and therefore the constant may be near 600. For flat, swampy areas, the constant may be on the order of 300.

Time to peak, t_p, can be expressed in terms of time of concentration, t_c, as follows:

$$t_p = \frac{2}{3} t_c \tag{4.9}$$

Expressing q_p in terms of t_c rather than t_p yields:

$$q_p = \frac{K_u K_p A Q_D}{t_c} \tag{4.10}$$

where:

K_u	=	Unit conversion constant, 1.5 in CU (0.00645 in SI)

Example 4.4:	NRCS dimensionless unit hydrograph.
Objective:	Estimate the peak and time to peak for the NRCS dimensionless unit hydrograph.

Given: The following watershed conditions:

Watershed is commercially developed.

Watershed area = 0.463 mi^2 (1.2 km^2)

Time of concentration = 1.34 h

Q_D = 1.0 inch (for unit hydrograph, 1 mm for SI)

Step 1. Calculate peak flow using equation 4.10.

$q_p = (K_u\ K_p\ A\ Q_D) / t_c$ = [1.5 (484) (0.463) (1.0)] / 1.34 = 251 ft^3/s

Step 2. Calculate time to peak using equation 4.9.

$t_p = (2/3)\ t_c$ = (2/3) (1.34) = 0.89 h

Solution:	The key parameters for this application are a peak flow of 251 ft^3/s (7.1 m^3/s) and a time to peak of 0.89 hours.

4.3.1.2 Snyder Unit Hydrograph

The Snyder method uses empirical terms and the physiographic characteristics of the drainage basin to determine a unit hydrograph. Figure 4.5 illustrates the key parameters that determine the hydrograph shape (lag time, unit hydrograph duration, peak flow, and hydrograph time widths of 50 percent and 75 percent of the peak flow). The designer adjusts the selected parameters so that the volume of the hydrograph equals one inch (millimeter) of direct runoff. Symbols used in the figure are:

T_R	=	Duration of unit excess rainfall, h
t_L	=	Lag time from the centroid of the unit rainfall excess to the peak of the unit hydrograph, h
t_p	=	Time to peak flow of the unit hydrograph, h
t_b	=	Time base of the unit hydrograph, h
W_{50}, W_{75}	=	Time width of unit hydrograph at discharge equal to 50 and 75 percent, h

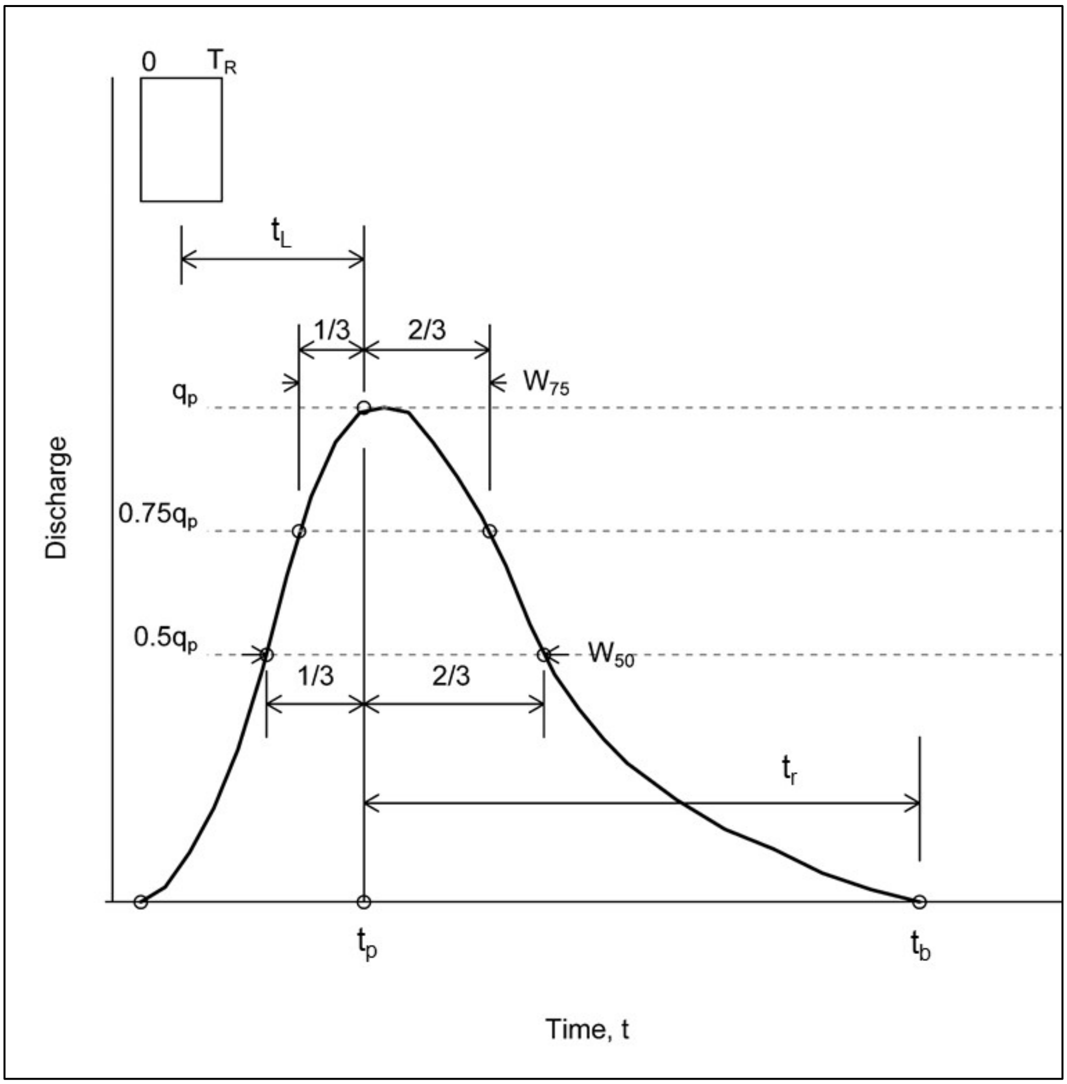

Figure 4.5. Snyder synthetic hydrograph definition.

Snyder initially applied this unit hydrograph for watersheds in the Appalachian highlands; however, the general method has been successfully applied throughout the country. HDS-2 provides additional information and an example problem that describes the procedures for computing the Snyder Synthetic Unit Hydrograph (FHWA 2002).

4.3.2 USGS Nationwide Urban Hydrograph

The USGS nationwide urban hydrograph method uses information developed by the USGS that approximates the shape and characteristics of hydrographs. Information needed for using this method include: 1) dimensionless hydrograph ordinates, 2) lag time, and 3) peak flow. HDS-2 (FHWA 2002) provides more detail on the USGS urban hydrograph method.

4.3.3 Continuous Simulation

In drainage design, peak flow and unit hydrographs typically meet the needs of designers but in some complex applications of stormwater management water quality analysis, designers use continuous simulation. Continuous simulation evaluates the entire hydrologic cycle based on the historical record and is distinctly different from single-event models as described in the preceding sections. Continuous simulation involves less simplification of processes and fewer assumptions and may produce more robust hydrologic estimates.

Depending on applicable design standards or requirements, or both, hydrologic design applications may rely on low flow statistics, flood-volume statistics, or flow duration curves. These applications use daily time series data. Modelers can use a calibrated continuous simulation model and a long-term precipitation record to produce a simulated streamflow series as described in HDS-2 (FHWA 2002). There are many implementations of continuous simulation.