Chapter 2
HYDROLOGY

Synopsis

Hydrologic studies are required to develop appropriate input data for hydraulic calculations to evaluate the impact of land development. Current conditions must be compared to predictions for post-construction conditions to assess the impact of the construction. This chapter describes techniques for estimating peak flood discharges and flood hydrographs recommended for use in Metro Nashville and Davidson County. Alternative methods of hydrologic analysis may be used with the approval ofMWS. The objectives of this chapter may generally be met using a systematic approach to arrive at the required results. The organization of this chapter is designed to facilitate such an approach and is outlined as follows;

1. Based on requirements (e.g., peak flow only, peak flow and runoff volume, or complete runoff hydrograph) and watershed characteristics (e.g., area, length, slope, and ground cover), select an appropriate hydrologic procedure from Section 2.1.

2. Identify rainfall data requirements for appropriate design storm conditions from Section 2.2. If required for hydrograph generation, develop a rainfall hyetograph for the design storm event using the method described in Section 2.2.

3. Estimate rainfall excess using Rational Method runoff coefficients or Natural Resource Conservation Service (NRCS) (formerly Soil Conservation Service (SCS)) curve numbers as outlined in Section 2.3.

4. Compute the watershed time of concentration using the procedures in Section 2.4.

5. Compute the peak runoff rate using methods described in Section 2.5, as appropriate for the procedure selected in Step 1. If required, generate a complete runoff hydrograph using one of the methods from Section 2.6.

6. Based on watershed characteristics, such as detention storage, open channel flow path length and slope, and channel roughness, determine if detention storage or channel routing is required. If appropriate, conduct hydrologic routing using methods described in Section 2.7.

2.1 Procedure Selection

The guidelines discussed in this section and summarized in Table 2-1 are recommended for selecting hydrologic procedures. A consideration of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm sewers or open channels. However, if the design must include flood routing (e.g., storage basins or complex conveyance networks and Table 2-1 timing of peak runoff), a flood hydrograph is usually required.

Because streamflow measurements for determining peak runoff rates for pre-project conditions are generally not available, accepted practice is to perform flood hydrology calculations using several methods. Results can then be compared (not averaged), and the method that best reflects project conditions selected and documented. When streamflow data are available, they should be obtained and analyzed before a hydrologic method is selected.

The Rational Method (see Section 2.5.2) is subject to the following limitations:

1. Only peak design flows can be estimated.

2. Time of concentration, t_c, is greater than or equal to 5 minutes and less than or equal to 30 minutes (5 minutes $\leq t_c \leq$ 30 minutes).

3. Drainage area, DA $\leq$ 100 acres.

Beyond these limits, results should be compared using other methods, and approval by MWS is required.

The SCS TR-55 (1986) graphical method (see Section 2.5.4) is subject to the following limitations:

1. Estimates of peak design flows only.

2. Design storm = SCS Type II 24-hour distribution.

3. Time of concentration, t_c, of 0.1 hour $\leq t_c \leq$ 10 hours.

4. The method was developed from results of computer analyses performed using TR-20 (USDA, SCS, 1983) for a 1-square mile homogeneous (describable by one CN value) watershed.

5. Curve number, CN, of 40 $\leq$ CN $\leq$ 98,

6. Ratio of initial abstraction to precipitation, I_a/P, of 0.1 $\leq$ I/P $\leq$ 0.5.

7. Unit hydrograph shape factor of 484.

8. Only one main stream channel in the watershed or, if more than one exists, nearly equal times of concentration for the branches.

9. Use of the 1986 version of TR-55 in place of the 1975 procedures.

10. No consideration of hydrologic channel routing.

The SCS TR-55 (1986) tabular method (see Section 2.6.4) can be used to estimate flood hydrographs and to approximate the effects of hydrologic channel routing, subject to the following limitations:

1. Design storm = SCS Type II 24-hour distribution.

2. Time of concentration, t_c, of 0.1 hour $\leq t_c \leq$ 2 hours.

3. DAs of individual subareas that do not differ by a factor of 5 or more. The procedure was developed for a DA of 1 square mile.

4. Curve number, CN, of 40 $\leq$ CN $\leq$ 98.

5. Ratio of initial abstraction to precipitation, I_a/P, of 0.1 $\leq I_a/P \leq$ 0.5.

6. Unit hydrograph shape factor of 484.

7. Reach travel time, t_T, of 0 to 3 hours.

8. Use of the 1986 version of TR-55 in place of the 1975 procedures.

U.S. Geological Survey (USGS) regional regression equations (see Section 2.5.3) have been prepared for Nashville and Davidson County for small, ungaged, rural and urban watersheds. These regression equations are subject to the following limitations:

1. Estimates of peak flows only.

2. DAs from 0.15 to 850 square miles for rural equations and from 0.15 to 30 square miles for urban equations.

3. Imperviousness less than or equal to 20 percent for rural equations and ranging from 20 to 80 percent for urban equations.

4. No extensive drainage improvements that alter the basin lagtime incorporated into the watershed.

Because a statistical estimate of expected error in predicted peak discharge is available for these regression equations, they are very useful for comparing results from other hydrologic methods. The statistical error estimates do not apply to watersheds that are outside the ranges of area and imperviousness listed above, however, and, as a result, should not be used for predicting peak discharge from such watersheds.

Unit hydrograph theory (see Section 2.6.1) provides a generally applicable procedure for developing flood hydrographs using a basin-specific unit hydrograph and an appropriate rainfall hyetograph. Many computer models use unit hydrograph theory. With careful development of a basin-specific unit hydrograph, this versatile method can be adapted to a wide range of conditions.

Inman's dimensionless hydrograph (see Section 2.6.2) can be used to develop flood hydrographs with peak runoff rates and a basin lagtime from other hydrologic methods. Lagtime as used in Inman's dimensionless hydrograph is defined as the difference between the center of mass of rainfall excess and the center of mass of runoff. Inman's hydrograph is applicable to both rural and urban watersheds, subject to the following limitations:

1. Rural watershed drainage areas between 0.17 and 481 square miles, inclusive, and imperviousness less than 4 percent.

2. Urban watershed drainage areas between 0.47 and 64 square miles, inclusive, and imperviousness between 4 and 48 percent, inclusive.

Computer modeling is appropriate when limitations of simpler methods are exceeded, complex situations are being studied, or more detailed information is required. HEC-1 or HEC-HMS developed by the U.S. Army Corps of Engineers, (1990 and 1998), or SWMM-RUNOFF developed by the U.S. Environmental Protection Agency, (Huber et al, 1992; Roesner et al 1994) calibrated to basin-specific data, is the recommended model. MWS has prepared a model for many Davidson County watersheds.

2.2 Rainfall Data

Rainfall data required for hydrologic studies include total rainfall depth and areal and time distribution for design or historical storm conditions. Data developed specifically for Metro Nashville include intensity-duration-frequency (IDF) curves and depth-duration-frequency data, which are required for predicting peak discharge rates and for developing runoff hydrographs.

2.2.1 Intensity-Duration-Frequency Relationships

Precipitation frequency estimates were obtained from NOAA Atlas 14, *Precipitation-Frequency Atlas of the United States* (Volume 2, Version 3). The data is available online from the Precipitation Frequency Data Server at "www.nws.noaa.gov/oh/hdsc/pfds/". Intensity-Duration-Frequency (IDF) curves for durations up to 60-days are presented in Figure 2-1 for return periods of 1, 2, 5, 10, 25, 50, and 100 years. Corresponding depth-duration-frequency data for durations up to 24 hours are included in Figure 2-1. The rainfall intensities and depths shown in Figure 2-

1 are provided for the Nashville WSO Airport gage (Site Identification 40-6402); however, as the drainage area increases, the intensity of precipitation should be reduced as recommended by the NWS. Areal reduction curves from TP-40 (Hershfield, 1961), which are appropriate for use with all recurrence intervals, are shown in Figure 2-2.

2.2.2 Rainfall Hyetographs

The rainfall data presented in Section 2.2.1 identify average depth or intensity over specific durations. To develop a flood hydrograph, however, a time variable distribution (hyetograph) is required.

The balanced storm approach (see Volume 3) was used to develop hyetographs for Metro Nashville for -a 24-hour storm duration. A dimensionless hyetograph for a 24-hour storm is shown in Figure 2-3. Tabular data for the dimensionless hyetograph along with the 2-, 10-, 25-, and 100-year return frequency hyetographs are presented in Table 2-2. A hyetograph can be developed for any return frequency by multiplying the ratio from the dimensionless hyetograph by the total 24-hour duration rainfall (see Figure 2-1) for the return frequency in question (see Example 2-1).

The tabular hyetographs in Table 2-2 are for 15-minute time intervals. If smaller time intervals are required, additional data points may be obtained from the dimensionless hyetograph curve in Figure 2-3 or interpolated directly from the tabular data.

2.2.3 Example Problem

Example 2-1. Hyetograph Development

Develop a hyetograph for a 5-year return frequency, 24-hour duration storm event. Assume 1-hour time intervals are required.

1. From Figure 2-1, the 5-year, 24-hour rainfall depth is 4.11 inches.

2. From Table 2-2, for a time of I hour, the P/P ratio is 0.0010.

3. The resulting hyetograph ordinate is determined by multiplying the 24-hour rainfall depth by the P/P_{24} ratio, or

   $P_{1\ hour}$ = 4.11 inches x 0.0010

   $P_{1\ hour}$ = 0.00411 inches

Steps 2 and 3 are repeated for each hourly time interval through 24 hours to develop the following 5-year, 24-hour hyetograph:

Time (hours)	P/P_{24} Ratio	Hyetograph Ordinates (inches)
0.00	0.0000	0.0000
1.00	0.0010	0.00411
2.00	0.0030	0.012
3.00	0.0065	0.027
4.00	0.0100	0.041
5.00	0.0250	0.103
6.00	0.0400	0.164
7.00	0.0600	0.247
8.00	0.0800	0.329
9.00	0.1080	0.444
10.00	0.1500	0.617
11.00	0.2200	0.904
12.00	0.5000	2.06
13.00	0.7900	3.25
14.00	0.8600	3.53
15.00	0.8950	3.68
16.00	0.9180	3.77
17.00	0.9365	3.85
18.00	0.9550	3.93
19.00	0.9675	3.98
20.00	0.9800	4.03
21.00	0.9875	4.06
22.00	0.9950	4.09
23.00	0.9975	4.10
24.00	1.0000	4.11

2.3 Rainfall Excess

Rainfall excess is the depth of precipitation that runs off an area during or immediately following a rainstorm, or the water depth remaining when abstractions are subtracted from the total precipitation. Abstractions (described in Chapter 2 of Volume 3) include evaporation, infiltration, transpiration, interception, and depression storage. Because the complexity of the actual process precludes a detailed determination of each abstraction, several methods are available to approximate the combined effects based on watershed characteristics. Either the Rational Method runoff coefficient or the SCS curve number can be used to estimate rainfall excess. Each approach is expressed mathematically as shown below:

Rational Method Runoff Coefficient

$$R_T = C_T P_T \qquad (2\text{-}1)$$

SCS Curve Number

$$R_T = \frac{(P_T - 0.2S)^2}{P_T + 0.8S} \qquad (2\text{-}2)$$

$$S = \frac{1000}{CN} - 10 \qquad (2\text{-}3)$$

where:

R_T = Rainfall excess for return period T, in inches, by the Rational Method or SCS method

C_T = Runoff coefficient for return period T, dimensionless

P_T = Precipitation depth for return period T, in inches

S = Maximum soil storage, in inches

CN = Watershed curve number

Procedures for determining the runoff coefficient and SCS curve number are discussed below. Variables that should be considered for either procedure include soil type, land use, antecedent moisture conditions, and precipitation volume.

Runoff coefficients or SCS curve numbers may be adjusted slightly if calibration data demonstrate a different value is justified. However, in the absence of adequate field data, the general procedures described in this section should be used.

2.3.1 Rational Method Runoff Coefficient

Runoff coefficients are generally determined from tabular values for a range of land cover or land use classifications as shown in Table 2-3. Runoff coefficients for various land uses, soil types, and watershed slopes in Table 2-3 apply when a design storm with a return period of 10 years or less is considered.

Runoff coefficients can be taken directly from the table for homogeneous land use. However, for mixed land uses, a weighted C value should be calculated as follows;

$$\overline{C} = \frac{\sum_{i=1}^{n} C_i A_i}{A_T} \qquad (2\text{-}3)$$

where:

$\overline{C}$ = Weighted composite runoff coefficient

n = Total number of areas with uniform runoff coefficients

C_i = Runoff coefficient for subarea i from Table 2-3

A_i = Land area contained in subarea i with uniform land use conditions, in acres or square miles

A_T = Total area of watershed, in acres or square miles

For return periods of more than 10 years, the coefficients from Table 2-3 should be multiplied by the frequency factors from Table 2-4. The following relationship is used to combine the data presented in Tables 2-3 and 2-4:

$$C_T = C_{10}\, X_T$$

where:

C_T = Runoff coefficient for return period T, dimensionless
C_{10} = Runoff coefficient for a design storm return period of 10 years or less (Table 2-3)
X_T = Design storm frequency factor for the return period T (Table 2-4)

The value of C_T should never be increased above 1.0 (see Example 2-2).

2.3.2 SCS Curve Numbers

The procedure for determining the SCS curve number uses soil survey information published by the SCS. Selection of an appropriate SCS curve number depends on land use, soil type, and antecedent moisture condition and is conducted in the following steps:

1. Identify soil types using the SCS soil survey report (1981) for Nashville and Davidson County.

2. Assign a hydrologic group to each soil type. The SCS has classified more than 4,000 soil series into four hydrologic soil groups, denoted by the letters A, B, C, and D. Soils in the A group have the lowest runoff potential; soils in the D group have the highest.

The hydrologic soil group classification considers only the soil properties that influence the minimum rate of infiltration obtained for a bare soil after prolonged wetting.

3. Identify land use conditions by categories for which CN values are available.

4. Identify drainage areas with combinations of uniform hydrologic group and land use conditions.

5. Use tables to select curve number values for each uniform drainage area identified in Step 4. A curve number value for Antecedent Moisture Condition II (AMC II) can be selected using Tables 2-5 and 2-6. Table 2-5 provides curve numbers for selected urban and suburban land uses; Table 2-6 gives information on rural land uses. Several special factors should be considered when curve numbers are being developed for an urban area, including the degree to which heavy equipment may compact the soil, the degree of surface and subsurface soil mixing caused by grading, and the depth to bedrock. In addition, the amount of barren pervious area (with little sod established) should be evaluated. Any one of these factors could move a soil normally placed in hydrologic group A or B to group B or C, respectively. The SCS soil survey report (1981) for Nashville and Davidson County provides additional information on hydrologic groups.

6. Calculate a composite curve number for the watershed using the equation:

$$\overline{CN} = \frac{\sum_{i=1}^{n} CN_i A_i}{A_T} \qquad (2\text{-}6)$$

where:

$\overline{CN}$ = Composite curve number for the watershed

n = Total number of areas with combinations of uniform hydrologic group and land use conditions

CN_i = Curve number for subarea i with a given combination of uniform hydrologic group and land use conditions (from Tables 2-5 and 2-6)

A_i = Land area for subarea i with combination of uniform hydrologic group and land use conditions, in acres or square miles

A_T = Total area of watershed, in acres or square miles

2.3.3 Example Problems

Example 2-2. Runoff Excess Using the Rational Method Runoff Coefficient

A 50-acre wooded watershed with an average overland and shallow channel slope of about 4 percent and good ground cover on both sandy and clay soils is to be developed as follows:

1. Undisturbed woodland on sandy soil- (hydrologic soil group A) = 10 acres

2. Undisturbed woodland on clay soil (hydrologic soil group D) = 10 acres

3. Multi-family residential (RM8 zoning classification) on sandy soil (hydrologic soil group B) = 20 acres

4. Industrial (IR zoning classification) on sandy soil (hydrologic soil group D) = 10 acres

Calculate the rainfall excess for proposed conditions from a 25-year, 24-hour storm using the Rational Method runoff coefficient.

1. From Figure 2-1, the 25-year, 24-hour rainfall depth is 5.53 inches.

2. The composite weighted runoff coefficient is computed from Equation 2-4 (repeated below)

$$\overline{C} = \frac{\sum_{i=1}^{n} C_i A_i}{A_T}$$

as follows:

a. From Table 2-3, for rolling (2-7 percent) woodland areas on sandy soil and assuming mid-range values, $C_1 = 0.17$

b. From Table 2-3, for rolling (2-7 percent) woodland areas on clay soil and assuming mid-range values, $C_2 = 0.22$

c. From Table 2-3, for RM8 zoning classification and assuming mid-range values, $C_3 = 0.70$

d. From Table 2-3, for IR zoning classification and assuming mid-range values, $C_4 = 0.85$

Sub-Area (i)	Area in Acres (A_i)	Runoff Coefficient (C_i)	Runoff Coefficient x Area (C_iA_i)
1	10	0.17	1.7
2	10	0.22	2.2
3	20	0.70	14.0
4	10	0.85	8.5
Total	50		26.4

$$\overline{C} = \frac{26.4}{50}, \quad \overline{C} = 0.53$$

3. From Equation 2-5 and Table 2-4, for a 25-year return period, the runoff coefficient is

$C_{25} = \overline{C}\,(X_{25})$

$C_{25} = 0.53\ (1.1)$, $C_{25} = 0.58$

4. Rainfall excess is computed with Equation 2-1:

$R_{25\ (RM)} = 0.58\ (5.53)$ inches

$R_{25\ (RM)} = 3.2$ inches

Example 2-3. Rainfall Excess Using the SCS Curve Number

Using the watershed and proposed development from Example 2-2, calculate the rainfall excess for proposed conditions from a 10-year, 12-hour storm using the SCS curve number.

1. From Figure 2-1, the 10-year, 12-hour rainfall depth is 3.92 inches.

2. The composite weighted curve number is computed from Equation 2-6 (repeated below)

$$\overline{CN} = \frac{\sum_{i=1}^{n} CN_i A_i}{A_T}$$

as follows:

a. From Table 2-6, for woodland areas with good ground cover on hydrologic soil group A, $CN_1 = 30$

b. From Table 2-6, for woodland areas with good ground cover on hydrologic soil group D, $CN_2 = 77$

c. From Table 2-5, for residential areas with 1/8-acre average lot size on hydrologic soil group B, $CN_3 = 85$

d. From Table 2-5, for industrial areas on hydrologic soil group D, $CN_4 = 93$

Sub-Area (i)	Area in Acres (A_i)	Curve Coefficient (CN_i)	Curve Number x Area (CN_iA_i)
1	10	30	300
2	10	77	770
3	20	85	1,700
4	10	93	930
Total	50		3,700

$$\overline{CN} = \frac{3{,}700}{50}, \quad \overline{CN} = 74$$

3. From Equation 2-3, the maximum soil storage in inches is

$$S = \frac{1{,}000}{74} - 10, \quad S = 3.51 \text{ inches}$$

4. The rainfall excess is computed using Equation 2-2:

$$R_{10\,(SCS)} = \frac{[3.92 - 0.2(3.51)]^2}{3.92 + 0.8(3.51)}$$

$$R_{10\,(SCS)} = \frac{(3.2)^2}{6.7}$$

$$R_{10\,(SCS)} = 1.54 \text{ inches}$$

2.4 Time of Concentration

To calculate the time of concentration of a watershed, at least three runoff components should be considered: overland, shallow channel (typically rill or gutter), and main channel. The Velocity Method is a segmental approach that can be used to account for each of these components by considering the average velocity for each flow segment being evaluated, and by calculating a travel time using the equation:

$$t_i = \frac{L_i}{(60)v_i} \qquad (2\text{-}7)$$

where:

t_i = Travel time for flow segment i, in minutes

L_i = Length of the flow path for segment i, in feet

v_i = Average flow velocity for segment i, in feet/second

The sum of the flow path segment lengths must equal the length of the watershed measured from the outlet to the hydrologically most distant point.

The time of concentration is then calculated, expressed as

$$t_c = t_1 + t_2 + t_3 + \ldots + t_i \qquad (2\text{-}8)$$

where:

t_c = Time of concentration, in minutes

t_1 = Overland flow travel time, in minutes

t_2 = Shallow channel (typically rill or gutter flow) travel time, in minutes

t_3 = Main channel travel time, in minutes

t_i = Travel time for the i^{th} segment, in minutes

Procedures for estimating the average flow velocity are discussed in subsequent sections.

2.4.1 Overland Flow

The length of the overland flow segment generally should be limited to 300 feet (Engman, 1983). The kinematic wave equation developed by Ragan (1971) is recommended for calculating the travel time for overland conditions. Figure 2-4 presents a nomograph that can be used to solve this equation, which is expressed as:

$$t_1 = 0.93 \left(\frac{L^{0.6} n^{0.6}}{I^{0.4} S^{0.3}} \right) \qquad (2\text{-}9)$$

where:

t_1 = Overland flow travel time, in minutes

L = Overland flow length, in feet

n = Manning's roughness coefficient for overland flow (see Table 2-7)

I = Rainfall intensity, in inches/hour (i on Figure 2-4)

S = Average slope of overland flow path, in feet/foot

Manning's n values reported in Table 2-7 were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations.

Equation 2-9 generally entails a trial and error process using the following steps;

1. Assume a trial value of rainfall intensity, I, for the watershed t_c as obtained by Equation 2-8.

2. Find the overland travel time, t_1, using Figure 2-4.

3 Use t_1 from Step 2 in Equation 2-8 to find the actual rainfall intensity for a storm duration of t_c (see Figure 2-1).

4. Compare the trial and actual rainfall intensities. If they are not similar, select a new trial rainfall intensity and repeat the process until the actual and trial rainfall intensities agree.

The SCS TR-55 method uses a non-iterative approximation to the overland flow travel time for flow paths of less than 300 feet. This approximation is expressed as:

$$t_1 = 0.42 \left(\frac{(nL)^{0.8}}{P_2^{0.5} S^{0.4}} \right) \qquad (2\text{-}10)$$

where:

t_1 = Overland flow travel time, in minutes

P_2 = 2-year, 24-hour rainfall, in inches

(remaining terms are defined in Equation 2-9)

Equation 2-10 is based on a single rainfall intensity from a 2-year, 24-hour rainfall event. For many cases, this approximate method will yield acceptable results; however, overland flow travel time should be checked using the iterative method for Equation 2-9 with results of the SCS TR-55 method as a starting point.

2.4.2 Shallow Channel Flow

Average velocities for shallow channel flow in rills and gutters can be obtained directly from Figure 2-5, if the slope of the flow segment in percent is known. Knowing the flow path length and average flow velocity, the travel time is estimated using Equation 2-7. Other types of shallow channel flow can be evaluated using the conventional form of Manning's Equation (see Chapter 3). Alternative procedures for evaluating gutter flow velocity are presented in Chapter 4. More than one segment of shallow channel flow can be considered to represent changing conditions.

2.4.3 Main Channel Flow

Average velocities for main channel flow should be evaluated with Manning's Equation (see Chapter 3). More than one main channel flow segment should be used where needed to account for varying main channel slope, roughness, or cross section.

2.4.4 Example Problem

Example 2-4. Time of Concentration Computation

The hydrologic flow path of the watershed described in Example 2-2 is about 2,000 feet in length with a total elevation change of about 40 feet. This flow path may be divided into the following three segments:

Segment No.	Type of Flow	Segment Length (ft)	Elevation Change (ft)	Slope (%)
1	Overland (woodland)	250	25	10
2	Shallow Channel	750	13	1.7
3	Main Channel	1,000	2	0.20

n = 0.025
Width = 10 feet
Depth = 2 feet
Approximately rectangular channel

Compute watershed time of concentration for a 10-year storm.

1. Compute the overland flow travel time, t_1, using the SCS TR-55 method from Equation 2-10 (repeated below).

$$t_1 = 0.42 \left(\frac{nL^{0.8}}{P_2^{\,0.5} S^{0.4}} \right)$$

From Figure 2-1, the 2-year, 24-hour rainfall, P_2, is 3.37 inches.

From Table 2-7, for woodlands, n = 0.45.

$$t_1 = 0.42 \left(\frac{(0.45 \times 250)^{0.8}}{(3.37)^{0.5}(0.10)^{0.4}} \right)$$

t_1 = 25 minutes

2. Compute the shallow channel flow travel time using the gutter flow curve in Figure 2-5.

From Figure 2-5, for a slope of 1.7 percent, the flow velocity is 2.6 feet/second.

By Equation 2-7,

$$t_2 = \frac{750}{(60)(2.6)}$$

t_2 = 4.8 minutes

3. Compute the main channel flow travel time.

The flow velocity is given by Manning's Equation,

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \quad \text{(see Chapter 3)}$$

For a rectangular channel with a 10-foot bottom width and an estimated depth of 2 feet,

R = (2 x 10)/[2(2)+10]

R = 20/14 = 1.43 feet

$$v = \frac{1.49}{0.025} (1.43)^{0.67} (0.002)^{0.5}$$

v = 3.4 feet/second

By Equation 2-7,

$$t_3 = \frac{1{,}000}{60(3.4)}$$

t_3 = 4.9 minutes

4. Compute the watershed time of concentration,

   From Equation 2-8,
   $t_c = 25 + 4.8 + 4.9$

   t_c = 35 minutes

5. Check the time of concentration using the kinematic wave equation (Equation 2-9, repeated below).

$$t_1 = 0.93\left(\frac{L^{0.6} n^{0.6}}{I^{0.4} S^{0.3}}\right)$$

From Step 1, n = 0.45. From Figure 2-1, for t_1 = 25 minutes from Step 1, the 10-year return frequency rainfall intensity is 3.6 inches/hour.

Assume the rainfall intensity to be 3.6 inches/hour (see Figure 2-4).

$$t_1 = 0.93\left(\frac{(250)^{0.6}(0.45)^{0.6}}{(3.6)^{0.4}(0.10)^{0.3}}\right)$$

t_1 = 18.9 minutes

From Step 2, t_2 = 4.8 minutes. From Step 3, t_3 = 4.9 minutes. From Equation 2-8,

$t_c = 18.9 + 4.8 + 4.9$

t_c = 29 minutes

From Figure 2-1, for t_c = 29 minutes, the 10-year return frequency rainfall intensity is 3.4 inches/hour.

Trial rainfall intensity and computed rainfall intensity do not agree.

6. Repeat computation assuming rainfall intensity is 3.4 inches/hour.

$$t_1 = 0.93\left(\frac{(250)^{0.6}(0.45)^{0.6}}{(3.4)^{0.4}(0.10)^{0.3}}\right)$$

t_1 = 19.3 minutes

t_c = 29 minutes

From Figure 2-1, for t_c = 29 minutes, rainfall intensity is 3.4 inches/hour.

Trial rainfall intensity and computed rainfall intensity agree.

7. Use t_c = 29 minutes.

Note: For this steep slope example, the SCS TR-55 method overestimates the time of concentration by about 20 percent. This demonstrates the need to check results from TR-55 for extreme cases.

2.5 Peak Runoff Rates

2.5.1 Gaged Sites

Streamflow and flood frequency data for gaged watersheds are available from the USGS. Locations in Metro Nashville and Davidson County for which streamflow information is currently being collected are presented in Table 2-8. In the event that streamflow measurements have not been analyzed to develop appropriate flood frequency curves, guidelines presented by the U.S. Water Resources Council (1981) should be followed. A brief discussion of the fundamentals behind the statistical analysis of streamflow data is presented in Volume 3, Chapter 2.

Flood frequency for gaged watersheds may be estimated by combined use of actual station data and regression equations, when applicable. A record-length-weighted average peak discharge estimate for a given recurrence interval may be computed using the equivalent years of record for the regression equation (see Table 2-9) and the number of years of actual station data.

Peak runoff rates for pre-project conditions should be determined from observed data when available. Otherwise, the synthetic procedures presented in the following sections should be used. Post-project conditions for gaged sites must be estimated with synthetic procedures. Synthetic procedures recommended for developing peak runoff rates at ungaged sites include the Rational Method, USGS regression equations, SCS TR-55 (1986), and computer modeling.

2.5.2 Rational Method

In this manual, the Rational Method is expressed in the equation:

$$Q_T = C_T\, I_{tc}\, A \qquad (2\text{-}11)$$

Where:

Q_T = Peak runoff rate for return period T, in cubic feet per second (cfs)

C_T = Runoff coefficient for return period T, expressed as the dimensionless ratio of rainfall excess to total rainfall (see Section 2.3.1)

I_{tc} = Average rainfall intensity, in inches/hour, during a period of time equal to t_c or the return period T

t_c = Time of concentration (see Section 2.4), in minutes

A = Watershed drainage area, in acres, tributary to the design point

The following procedure is recommended for using the Rational Method:

1. Collect watershed data.
2. Calculate time of concentration using information in Section 2.4.
3. Use the IDF curves in Figure 2-1 to determine the average rainfall intensity for the return period T and the time of concentration, t_c, from Step 2.
4. Obtain a runoff coefficient for the return period T, using the information in Section 2.3.1.
5. Compute the peak runoff rate for the return period T, using Equation 2-11.

2.5.3 USGS Regression Equations

Randolph and Gamble completed a study of Tennessee watersheds in 1976. In the central Tennessee area (called Hydrologic Area 3), they used 47 gage sites in both rural and urban watersheds to develop rural regression equations. The equations take the following general form:

$$Q_T = CR_T\, A^{XT} \qquad (2\text{-}12)$$

Where:

Q_T = Peak runoff rate for return period T, in cfs

CR_T = Regression constant for return period T (see Table 2-9)

A = Contributing drainage area, in square miles

XT = Regression exponent for return period T (see Table 2-9)

The rural regression equations should provide reasonable peak runoff rate estimates for areas between 0.15 and 850 square miles, inclusive, where the total impervious area is less than or equal to 20 percent.

For imperviousness greater than 20 percent, Robbins (1984b) has developed urban regression equations. The form of these equations has been slightly modified for Nashville and Davidson County by realizing that the 2-year, 24-hour rainfall is a constant (see Table 2-9 for constant values). The form of these equations is:

$$Q_T = CR_T\, A^{XT}\, IA^{YT} \qquad (2\text{-}13)$$

Where:

Q_T = Peak runoff rate for return period T, in cfs

CR_T = Regression constant for return period T (see Table 2-9)

A = Contributing drainage area, in square miles

XT = Regression exponent for return period T (see Table 2-9)

IA = Percent total imperviousness

YT = Regression exponent for return period T (see Table 2-9)

The urban regression equations should provide reasonable peak runoff rate estimates for areas between 0.15 and 30 square miles, inclusive, and total imperviousness up to about 80 percent, although extrapolations are permissible for purposes of comparison with other methods only.

Figure 2-6 presents an example solution of the rural and urban equations for Nashville and Davidson County for the l00-year storm. Equations 2-12 and 2-13 can be used to obtain peak flow estimates for other return periods within the specified ranges. The regression equations do not apply where basin lagtime is significantly altered, for example, by a great amount of detention or by paving of much of the collection system. See Volume 3 for further explanation of the derivation of the regression equations.

2.5.4 SCS TR-55 Graphic Method

The SCS has developed a graphical peak discharge method for estimating the peak runoff rate from watersheds with a single homogeneous land use. The method is based on the results of computer analyses performed using TR-20 (USDA, SCS, 1983) and is subject to certain limitations. A description of the SCS procedure and details on limitations are contained in SCS TR-55 (1986).

The graphical peak discharge method described in Chapter 4 of SCS TR-55 is based on the following equation:

$$Q_t = q_u \, A_m \, R_T \, F_p$$

where:

Q_t = Peak runoff rate for return period T, in cfs

q_u = Unit peak discharge, in cubic feet per second per square mile per inch (csm/inch)

A_m = Drainage area, in square miles

R_T = Runoff, in inches

F_p = Pond and swamp adjustment factor

Computation using the graphical peak discharge method proceeds as follows;

1. The 24-hour rainfall depth is determined from Figure 2-1 for the selected return frequency.

2. The runoff curve number, CN, and total rainfall runoff, R_T, are estimated using the procedures in Section 2.3.2.

3. The CN value is used to determine the initial abstraction, I_a, from Table 2-10 and the ratio I_a/P is then computed.

4 The watershed time of concentration is computed using the procedures in Section 2.4 and is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 2-7. If the ratio I_a/P lies outside the range shown in Figure 2-7, either the limiting values or another peak discharge method should be used.

5. The pond and swamp adjustment factor, F_p, is estimated from below (TR-55, USDA, SCS, 1986):

Pond and Swamp Areas (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

6. The peak runoff rate is computed using Equation 2-14.

Accuracy of the graphical peak discharge method is subject to specific limitations, including the following factors presented in TR-55:

1. The watershed must be hydrologically homogeneous and describable by a single CN value.

2. The watershed may have only one main stream, or if more than one, the individual branches must have nearly equal times of concentration.

3. Hydrologic routing cannot be considered.

4. The pond and swamp adjustment factor, F_p , applies only to areas located away from the main flow path.

5. Accuracy is reduced if the ratio I_a/P is outside the range given in Figure 2-7.

6. The weighted CN value must be greater than or equal to 40 and less than or equal to 98.

7. The same procedure should be used to estimate pre- and post-development time of concentration when computing pre- and post-development peak discharge.

8. The watershed time of concentration must be between 0.1 and 10 hours.

The 1986 version of TR-55 includes extensive revisions to the 1975 version, which is no longer appropriate for use in Metro Nashville and Davidson County. The 1986 version can be obtained from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580. Microcomputer diskettes with TR-55 procedures are also available under catalog number PB87-101598.

2.5.5 Other Techniques

Other methods may be used for computation of design flow rates, subject to the approval of MWS. The computer model HEC-HMS, developed by the U.S. Army Corps of Engineers , (1998), or SWMM Runoff block developed by the U.S. Environmental Protection Agency , (Huber et al, 1992; Roesner et al 1994) are recommended for complex hydrologic conditions.

2.5.6 Example Problems

<u>Example 2-5. Rational Method Peak Runoff Rate</u>

Use the Rational Method to compute the peak runoff rate from the watershed in Example 2-2 for a 25-year, 24-hour storm event.

1. Total area of watershed = 50 acres.

2. From Example 2-2, the 25-year runoff coefficient, C_{25}, is 0.58.

3. From Example 2-4, the watershed time of concentration is

   a. By TR-55 (Equation 2-10), t_c = 35 minutes
   b. By kinematic wave equation (Equation 2-9), t_c = 29 minutes

4. From Figure 2-1, the rainfall intensity for a 25-year storm is

   a. For t_c = 35 minutes, I = 3.5 inches/hour
   b. For t_c = 29 minutes, I = 4.0 inches/hour

5. The peak runoff rate is computed from Equation 2-11 as follows:

   a. Using the SCS TR-55 method for t_c:

   Q_{25} = (0.58) (3.5) (50)

   Q_{25} = 101.5 cfs

   b. Using the kinematic wave equation for t_c:

   Q_{25} = (0.58) (4.0) (50)

   Q_{25} = 116 cfs

Note: For this example, the SCS TR-55 method (Equation 2-10) results in a peak runoff rate 11 percent lower than that obtained using the kinematic wave equation (Equation 2-9) for watershed time of concentration.

Example 2-6. USGS Regression Equation Peak Runoff Rate

A 1,200-acre watershed has an imperviousness of 40 percent resulting from urbanization. Determine the 10-, 25-, and 100-year peak runoff rates using the USGS regression equations.

1. Determine if the watershed characteristics are within the limits of applicability of the USGS regression equations.

   a. Area = 1,200 acres or 1.9 square miles
   b. Imperviousness = 40 percent

   Since the area is between 0.15 and 30 square miles and the imperviousness is between 20 and 80 percent, the USGS urban regression equations are applicable.

2. From Table 2-9, the peak runoff rates are

   a. $Q_{10} = 168\ (1.9)^{0.75}\ (40)^{0.43}$
   $Q_{10} = 1{,}328$ cfs

   b. $Q_{25} = 234\ (1.9)^{0.75}\ (40)^{0.39}$
   $Q_{25} = 1{,}596$ cfs

   c. $Q_{100} = 305\ (1.9)^{0.75}\ (40)^{0.40}$
   $Q_{100} = 2{,}159$ cfs

Example 2-7. SCS TR-55 Graphical Method Peak Runoff Rate

Use the SCS graphical peak discharge method to compute the peak runoff rate from a 25-year, 24-hour storm event from the watershed in Example 2-2. Use the watershed time of concentration computed in Example 2-4.

1. From Figure 2-1, the 25-year, 24-hour rainfall depth is 5.53 inches.

2. From Example 2-3, the curve number, CN, is 74.

3. From Example 2-3, Step 3, the soil storage, S, is 3.51 inches.

$$R_{25\ (SCS)} = \frac{[5.53 - 0.2(3.51)]^2}{5.53 + 0.8(3.51)}$$

4. Using Equation 2-2, the rainfall excess is

   $R_{25\ (SCS)} = 2.8$ inches

5. From Table 2-10, the initial abstraction, I_a, is 0.703 inches.

6. The I_a/P ratio is

   $I_a/P = 0.703/5.53$

   $I_a/P = 0.13$

7. From Figure 2-7, for the time of concentration, t_c, from Example 2-4 of 0.6 hour (35 minutes) and with an I_a/P ratio of 0.13, the unit peak discharge, q_u, is 475 csm/inch of runoff.

8. The pond and swamp adjustment factor, F_P, is 1.0 since no pond or swamp area exists.

9. The peak runoff rate is computed using Equation 2-14, as follows:

   $Q_{25} = (475)\ (50/640)\ (3.3)\ (1.0)$

   $Q_{25} = 122$ cfs

2.6 Flood Hydrographs

Flood hydrograph procedures presented include unit hydrograph theory, Inman's dimensionless hydrograph, the rational hydrograph method, and the SCS TR-55 (1986) tabular method.

2.6.1 Unit Hydrographs

Unit hydrographs should be developed using observed rainfall and streamflow records when they are available. Procedures for deriving unit hydrograph parameters from observed data are well-documented in publications by Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), Chow (1964), and the USDOT, FHWA (HEC-19, 1984). When observed data are not available for deriving unit hydrograph parameters, as is often the case, synthetic procedures are required. The SCS dimensionless unit hydrograph approach is presented below.

Two types of dimensionless unit hydrographs were developed by the SCS as shown in Figure 2-8; the first has a curvilinear shape and the second is a triangular approximation to that curvilinear shape. In both cases, once the time to peak and peak flow for a particular unit hydrograph have been defined, the entire shape can be estimated using the dimensionless unit hydrograph ratios in Table 2-11.

The procedure for using the SCS curvilinear dimensionless unit hydrograph is as follows:

1. Estimate the time of concentration, t_c, using an appropriate method (see Section 2.4).

2. Calculate the incremental duration of runoff producing rainfall,)D, using the equation:

   $$DD = 0.133\ t_c \qquad (2\text{-}15)$$

   where:

   DD = Incremental duration of runoff producing rainfall, in minutes

   t_c = Time of concentration, in minutes

3. Calculate time to peak, t_p, using the equation:

   $$t_p = \frac{DD}{2} + 0.6\ t_c \qquad (2\text{-}16)$$

   where:

   t_p = Time to peak, in minutes

   DD = Incremental duration of runoff producing rainfall, in minutes

   t_c = Time of concentration, in minutes

4. Calculate peak flow rate, q_p, from the equation:

   $q_p = 60\ (BA)/t_p$
   where:

   q_p = Peak flow rate, in cfs

   B = Hydrograph shape factor, ranging from 300 for flat swampy areas to 600 in steep terrain. The SCS standard B value of 484 should be used in Metro Nashville unless another value is approved by MWS.

   A = Drainage area, in square miles

   t_p = Time to peak, in minutes

5. List the hydrograph time, t, in increments of DD and calculate t/t_p .

6. Using Table 2-11 or Figure 2-8, find the q/q_p ratio for the appropriate t/t_p ratios from Step 5.

7. Calculate the appropriate unit hydrograph ordinates by multiplying the q/q_p ratios by q_p .

8. Determine the volume under the unit hydrograph to ensure that it is equal to 1 inch.

The SCS triangular dimensionless unit hydrograph procedure is identical to the curvilinear procedure presented above. However, to draw the required unit hydrograph, only t/t_p ratios of 0, 1, and 2.67 are needed. When applying the triangular dimensionless unit hydrograph, the time of concentration, t_c, is computed using Equation 2-15, the time to peak, t_p, is computed using Equation 2-16, and the time base, t_b, is computed as follows:

$$t_b = 2.67\ t_p \qquad (2\text{-}18)$$

where:

t_p = Time to peak, in minutes

t_b = Time base, in minutes

If a short-duration unit hydrograph is used to develop a long-duration synthetic hydrograph, the actual shape of the unit hydrograph is not nearly as important as its time to peak and peak flow rate. Therefore, a triangular unit hydrograph would likely produce approximately the same synthetic runoff hydrograph as a curvilinear unit hydrograph. A flood hydrograph can be developed through the following steps using unit hydrograph theory (see Example 2-9):

1. Develop a unit hydrograph for the subject watershed using the SCS procedure.

2. Develop a design storm hyetograph using the time interval for which the unit hydrograph was developed (as presented in Section 2.2).

3. Develop a rainfall excess hyetograph using an appropriate procedure as presented in Section 2.3.

4. Route the rainfall excess hyetograph through the subject watershed by multiplying the ordinates of the unit hydrograph by the respective rainfall excess increments. Each increment of rainfall excess will produce a routed incremental hydrograph. Each routed incremental hydrograph is delayed by the design storm time interval.

5. Develop the composite synthetic runoff hydrograph by summing the ordinates of each routed incremental hydrograph from Step 4 at each time interval of the hydrograph.

6. Check to ensure that the volume of the synthetic runoff hydrograph is equal to the volume of rainfall excess, using the equation:

$$V = \frac{12 \Delta t \Sigma q_i}{A(43{,}560)} \qquad (2\text{-}19)$$

where:

V = Volume under the hydrograph, in inches

Δt = Time increment of the runoff hydrograph ordinates, in seconds

$\sum q_i$ = Sum of the runoff hydrograph ordinates, in cfs, for each time increment i

A = Watershed drainage area, in acres

2.6.2 Inman's Dimensionless Hydrograph

Inman's dimensionless hydrograph presented in Table 2-12 can be used to develop a flood hydrograph using the following steps (see Example 2-10):

1. Determine the watershed drainage area and main channel length.

2. Considering the limitations of hydrologic method, compute the peak discharge using any applicable method presented in Section 2.5. If the USGS regression equations are used, a statistical estimate of expected error may also be developed.

3. If the watershed is urbanized, estimate the percentage of impervious area.

4. Considering the limitations of the lagtime regression equations, compute the basin lagtime, defined as the difference between the center of mass of rainfall excess and the center of mass of runoff, using the appropriate equation as follows.

   For rural basins (imperviousness $\leq$20 percent) with drainage area from 0.15 to 850 square miles and channel length from 0.56 to 74 miles:

   $$RLT = 0.94\,(CL)^{0.86} \qquad (2\text{-}20)$$

where:

RLT = Rural basin lagtime, in hours (time between the center of mass of rainfall excess and runoff), with a standard error of estimate of ±39.2 percent

CL = Channel length, in miles, from the discharge site to the hydrologically most distant point, measured along the main channel

For urban basins with imperviousness greater than 20 and less than 80 percent, drainage area from 0.15 to 30 square miles, and channel length from 0.65 to 17 miles:

$$ULT = 1.64\,(CL)^{0.49}\ IA^{-0.16} \qquad (2\text{-}21)$$

where:

ULT = Urban basin lagtime, in hours (time between the center of mass for rainfall excess and runoff), with a standard error of estimate of ±15.9 percent

CL = Channel length, in miles, from the discharge site to the hydrologically most distant point, measured along the main channel

IA = Effective impervious area directly connected to the drainage system, in percent

Consider using another hydrologic method, such as TR-55 or unit hydrograph method, if the watershed characteristics are outside the ranges listed.

The errors of estimate given above for basin lagtime do not apply outside the ranges modeled.

5. Compute the coordinates of the flood hydrograph by multiplying the value of lagtime from Step 4 by the time ratios in Table 2-12 and the value of peak discharge from Step 2 by the discharge ratios in Table 2-12.

2.6.3 Rational Hydrograph Method

A rational hydrograph method may be used for small homogeneous watersheds when attenuation is insignificant. A small paved parking lot is one example where this method may be appropriate.

The method presented uses a rainfall hyetograph that is developed using a balanced storm approach (see Volume 3) with time increments equal to the watershed time of concentration. Incremental rainfall runoff depth is computed using the Rational Method.

The following procedure is used for this method (see Example 2-11):

1. Determine appropriate design storm for facilities being evaluated.

2. Estimate the runoff coefficient (Section 2.3.1).

3. Compute the watershed time of concentration (see Section 2.4).

4. Divide the design storm duration into intervals using the watershed time of concentration as an approximate time interval.

5. Determine the design storm rainfall intensity in inches per hour from Figure 2-1 using the time at the end of each interval as the duration in Figure 2-1.

6. Multiply the rainfall intensity by the time interval to obtain total accumulated rainfall.

7. Subtract the preceding value of total accumulated rainfall to obtain the incremental rainfall for each time interval.

8. Distribute or "balance" the incremental rainfall about the center of the storm duration by placing the largest incremental rainfall at the center, the second largest before the center, the third largest after the center, the fourth largest before the second largest, the fifth largest after the third largest, etc., until the "balanced" storm is completed for the duration in question.

9. Determine the rainfall runoff rate during the time interval, in cfs, by multiplying the incremental runoff volume from Step 8 by the runoff coefficient and area and dividing by the length of the time interval.

2.6.4 NRCS TR-55 Tabular Method

The NRCS (formerly SCS) has developed a tabular hydrograph method for developing flood hydrographs from watersheds that can be divided into relatively homogeneous land uses. The method is based on the results of computer analyses performed using TR-20 (USDA, SCS, 1983) and is subject to certain limitations. A description of the SCS procedure and details on limitations are contained in NRCS TR-55 (1986).

Since the 1986 version of TR-55 includes extensive revisions to the 1975 version, the earlier version is no longer appropriate for use in Metro Nashville and Davidson County. The 1986 version can be obtained from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580. Microcomputer diskettes with TR-55 procedures are available under catalog number PB87-101598.

2.6.5 Other Methods

Other methods of developing flood hydrographs may be used subject to approval by MWS. Other methods may be used for computation of design flow rates, subject to the approval of MWS. The computer model HEC-HMS, developed by the U.S. Army Corps of Engineers (1998), or SWMM Runoff block developed by the U.S. Environmental Protection Agency (Huber et al, 1992; Roesner et al 1994) are recommended for complex hydrologic conditions.

2.6.6 Example Problems

<u>Example 2-8. SCS Dimensionless Unit Hydrograph</u>

Develop a synthetic unit hydrograph for the watershed in Example 2-2 using the SCS curvilinear approach.

1. From Example 2-4, the watershed time of concentration, t_c, is 35 minutes.

2. From Equation 2-14, the incremental duration of runoff producing rainfall, ΔD, is

   $\Delta D = 0.133\ (35)$
   $\Delta D = 4.6$ minutes or 0.08 hours

3. From Equation 2-16, the time to peak, t_p, is

   $$t_p = \frac{4.6}{2} + 0.6\ (35) = 23 \text{ minutes}$$

4. From Equation 2-17, the unit hydrograph peak flow rate, q_p, is

   $$q_p = 60 \left(\frac{484(50/640)}{23} \right)$$

   $q_p = 99$ cfs

5. From Figure 2-8 or Table 2-11, determine the q/q_p ratio for appropriate t/t_p ratios and calculate the unit hydrograph ordinates by multiplying the q/q_p ratio by q_p as follows:

Time t (hours)	t/t_p (t/0.4 hours)	q/q_p (q/99 cfs)	q (cfs)
0.00	0.00	0.000	0
0.08	0.20	0.100	10
0.16	0.40	0.310	31
0.24	0.60	0.660	65
0.32	0.80	0.930	92
0.40	1.00	1.000	99
0.48	1.20	0.930	92
0.56	1.40	0.780	77
0.64	1.60	0.560	55
0.72	1.80	0.390	39
0.80	2.00	0.280	28

Time t (hours)	t/t_p (t/0.4 hours)	q/q_p (q/99 cfs)	q (cfs)
0.88	2.20	0.207	20
0.96	2.40	0.147	15
1.04	2.60	0.107	11
1.12	2.80	0.077	8
1.20	3.00	0.055	5
1.28	3.20	0.040	4
1.36	3.40	0.029	3
1.44	3.60	0.021	2
1.52	3.80	0.015	1
1.60	4.00	0.011	1
1.68	4.20	0.008	1
1.76	4.40	0.005	0
1.84	4.60	0.002	0
1.92	4.80	0.001	0
2.00	5.00	0.000	0
			659

6. Check that the unit hydrograph volume equals 1 inch using Equation 2-19:

$$V = \frac{12(0.08)(3{,}600)(659)}{(50)(43{,}560)}$$

V = 1.05 @1 (close enough)

Example 2-9. Flood Hydrograph Using Unit Hydrograph Theory

Develop a synthetic runoff hydrograph for a 25-year, 2-hour design storm for the watershed described in Example 2-2 using the unit hydrograph developed in Example 2-8.

1. Develop a balanced storm hyetograph and cumulative mass curve using the IDF curve for a 25-year storm from Figure 2-1 as follows:

Time, t (hours)	Intensity, i (inches/hour)	Rainfall Depth (inches)	Incremental Depth (inches)	Balanced Depth (inches)	Cumulative Depth (inches)
0.00	0.00	0.00	0.00	0.00	0.00
0.08	8.00	0.64	0.64	0.04	0.04
0.16	6.60	1.06	0.42	0.04	0.09
0.24	5.62	1.35	0.29	0.05	0.14
0.32	4.95	1.58	0.24	0.05	0.19
0.40	4.50	1.80	0.22	0.06	0.26
0.48	4.08	1.96	0.16	0.07	0.33
0.56	3.76	2.11	0.15	0.09	0.42
0.64	3.50	2.24	0.13	0.11	0.52
0.72	3.26	2.35	0.11	0.13	0.65
0.80	3.07	2.46	0.11	0.16	0.81
0.88	2.90	2.55	0.09	0.24	1.05
0.96	2.74	2.64	0.09	0.42	1.46
1.04	2.61	2.72	0.08	0.64	2.10
1.12	2.49	2.79	0.07	0.29	2.40
1.20	2.39	2.86	0.07	0.22	2.61
1.28	2.29	2.93	0.06	0.15	2.76
1.36	2.20	2.99	0.06	0.11	2.88
1.44	2.11	3.04	0.05	0.09	2.97
1.52	2.03	3.09	0.05	0.08	3.05
1.60	1.96	3.14	0.05	0.07	3.12
1.68	1.90	3.19	0.05	0.06	3.18
1.76	1.84	3.23	0.04	0.05	3.23
1.84	1.78	3.28	0.04	0.05	3.27
1.92	1.73	3.32	0.04	0.04	3.32
2.00	1.68	3.36	0.04	0.04	3.36

2. Develop a rainfall excess hyetograph using the SCS curve number approach (Equation 2-2). From Example 2-3, CN is 74 and S is 3.51 inches. From Figure 2-1, the 25-year, 2-hour rainfall depth is 2.97 inches.

Apply Equation 2-2 to the cumulative depth as follows to obtain the rainfall excess hyetograph shown below:

Time, t (hours)	Cumulative Depth (inches)	Cumulative Rainfall Excess (inches)	Rainfall Excess Hytegraph (inches)
0.00	0.00	0.00	0.00
0.08	0.04	0.00	0.00
0.16	0.09	0.00	0.00
0.24	0.14	0.00	0.00
0.32	0.19	0.00	0.00
0.40	0.26	0.00	0.00
0.48	0.33	0.00	0.00
0.56	0.42	0.00	0.00
0.64	0.52	0.00	0.00
0.72	0.65	0.00	0.00
0.80	0.81	0.00	0.00
0.88	1.05	0.03	0.03
0.96	1.46	0.14	0.10
1.04	2.10	0.40	0.26
1.12	2.40	0.55	0.15
1.20	2.61	0.67	0.12
1.28	2.76	0.76	0.09
1.36	2.88	0.83	0.07
1.44	2.97	0.89	0.06
1.52	3.05	0.94	0.05
1.60	3.12	0.98	0.05
1.68	3.18	1.02	0.05
1.76	3.23	1.06	0.03
1.84	3.27	1.09	0.02
1.92	3.32	1.12	0.01
2.00	3.36	1.15	0.01

3. Route the rainfall excess hyetograph through the watershed using the unit hydrograph developed in Example 2-8. Each increment of rainfall excess from the design storm is multiplied by the unit hydrograph ordinates. This routed incremental hydrograph begins at the time interval during which the rainfall excess occurred. The rainfall excess hydrograph is obtained by summing the ordinates of each routed incremental hydrograph, as shown in Table 2-13.

4. Check that hydrograph volume is equal to the rainfall excess. From Equation 2-19,

$$V = \frac{12(0.08)(3{,}600)(703)}{(50)(43{,}560)} = 1.1 \quad \text{(close enough)}$$

Example 2-10. Inman's Dimensionless Hydrograph

Develop a runoff hydrograph for a 25-year, 2-hour design storm for the watershed described in Example 2-2 using Inman's dimensionless hydrograph method. Use the peak runoff rate from Example 2-9.

1. From Example 2-2, the watershed area is 50 acres, and from Example 2-4, the channel length is 2,000 feet or 0.38 mile. This watershed length is just below the lower limit for the lagtime regression equation (0.65 mile). Example calculations are presented for demonstration purposes.

2. From Example 2-9, the peak runoff rate is 69 cfs.

3. The imperviousness is estimated to be about 30 percent using land use from Example 2-2.

4. From Equation 2-21, for urban basins, the basin lagtime between the center of mass of rainfall excess and runoff is

   $ULT = 1.64\ (0.38)^{0.49}\ (30)^{-0.16}$
   $ULT = 0.6$ hour

5. Compute the runoff hydrograph ordinates by multiplying the peak runoff rate from Step 2 by the discharge ratios from Table 2-12 as follows:

Runoff Time, t (hours)	Time Ratio (t/LT)	Discharge Ratio (Q/Q_P)	Runoff Hydrograph Ordinate (cfs)
0.15	0.25	0.12	8.3
0.30	0.50	0.40	27.6
0.45	0.75	0.84	58.0
0.60	1.00	0.99	68.3
0.75	1.25	0.74	51.1
0.90	1.50	0.47	32.4
1.05	1.75	0.30	20.7
1.20	2.00	0.20	13.8
1.35	2.25	0.14	9.7
1.50	2.50	0.09	6.2

$LT = 0.6$ hour
$Q_P = 69$ cfs

Example 2-11. Rational Hydrograph Method

Develop a runoff hydrograph for the watershed described in Example 2-2 using the rational hydrograph method for a 25-year, 2-hour design storm.

1. Watershed characteristics determined from previous examples

    a. Area = 50 acres
    b. Time of concentration, t_c @30 minutes
    c. Runoff coefficient, C = 0.58

2. Develop a balanced storm for time increments equal to the time of concentration using the procedure from Example 2-9, Step 1.

Time, t (hours)	Intensity, i (inches/hour)	Rainfall Depth (inches)	Incremental Depth (inches)	Balanced Storm (inches)
0	0	0	0	0.28
0.5	4.00	2.00	2.00	0.66
1.0	2.66	2.66	0.66	2.00
1.5	2.05	3.08	0.42	0.42
2.0	1.68	3.36	0.28	0

3. Develop a runoff hydrograph by multiplying the balanced storm ordinate by the runoff coefficient and the watershed area, and divide the results by the time increment (time of concentration):

Time t (hours)	Balanced Storm (inches)	$\frac{C \times A}{t_c}$ (acres/hours)	Runoff Hydrograph Ordinate (cfs)
0	0.28	58	16
0.5	0.66	58	38
1.0	2.00	58	116
1.5	0.42	58	24
2.0	0	58	0

2.7 Hydrologic Channel Routing

The Muskingum Method of hydrologic channel routing is recommended when computer-based procedures are not used. A tabular method presented by the SCS in TR-55 (1986) is appropriate for preliminary desktop calculations.

2.7.1 Muskingum Method

The Muskingum Method is applied with the following steps:

1. Select a representative flow rate for evaluating the parameters K and X. Use 75 percent of the inflow hydrograph peak. If this flow exceeds the channel capacity, use the channel capacity as representative.

2. Find the velocity of a small kinematic wave in the channel using the equation:

$$v = \frac{1}{B}\left(\frac{Q(Y+\Delta Y) - Q(Y)}{\Delta Y}\right) \qquad (2\text{-}22)$$

where:

v = Velocity of a small kinematic wave, in feet/second

$Q(Y)$ = A representative flow rate for channel routing at representative depth Y, in cfs

ΔY = A small increase in the representative depth of flow in the channel

$Q(Y+\Delta Y)$ = Flow rate at the new depth $Y + \Delta Y$, in cfs

B = Top width of water surface, in feet

3. Estimate the minimum channel length allowable for the routing, using the following equation, and make sure that ΔL is greater than ΔL_{min}:

$$\Delta L_{\min} = \frac{Q}{BS_o v} \qquad (2\text{-}23)$$

where:

ΔL_{min} = Minimum channel length for routing calculations, in feet

Q = Flow rate, in cfs

B = Top width of water surface, in feet

S_o = Slope of channel bottom, in feet/foot

v = Velocity of a small kinematic wave, in feet/second

4. Estimate a value of K using the following equation (make sure that K is less than the time of rise for the inflow hydrograph):

$$K = \frac{DL}{v} \tag{2-24}$$

where:

K = Muskingum channel routing time constant for a particular channel segment

DL = Channel routing segment length, in feet

v = Velocity of a small kinematic wave, in feet/second

5. Estimate the value of X using the equation:

$$X = 0.5\left(1 - \frac{Q}{BS_o vDL}\right) \tag{2-25}$$

where:

X = Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume

Q = Flow rate, in cfs

B = Top width of water surface, in feet

S_o = Slope of channel bottom, in feet/foot

v = Velocity of a small kinematic wave, in feet/second

DL = Channel routing segment length, in feet

6. Select a reasonable channel routing time period,)t, using the criteria expressed by the following inequality:

$$\frac{K}{3} \le \Delta t \le K \qquad (2\text{-}26)$$

7. Determine coefficients C_0, C_1, and C_2 using the following equations (make sure that $C_0 + C_1 + C_2 = 1.0$)

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (2\text{-}27)$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (2\text{-}28)$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (2\text{-}29)$$

where:

K = Muskingum channel routing time constant for a particular segment

X = Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume

Δt = Routing time period, in hours

8. Determine an initial outflow, O_1 , then calculate an ending outflow, O_2, using the equation:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \qquad (2\text{-}30)$$

where:

O_2 = Outflow rate at the end of routing time period Δt, in cfs

I_2 = Inflow rate at the end of routing time period Δt, in cfs

I_1 = Inflow rate at the beginning of routing time period Δt, in cfs

O_1 = Outflow rate at the beginning of routing time period Δt, in cfs

The routing is performed by repetitively solving Equation 2-30, assigning the current value of O_2 to O_1, and determining a new value of O_2. This sequence continues until the entire inflow hydrograph is routed through the channel.

2.7.2 NRCS TR-55 Tabular Method

The NRCS (formerly SCS) has developed a tabular method that can be used to develop a runoff hydrograph and to evaluate channel routing conditions. Consult TR-55 (1986) for a description of the method and the limitations of its application. The newer version of TR-55 supersedes the 1976 version and should be used in place of the older publication.

Table 2-1
GUIDELINES FOR SELECTING HYDROLOGIC PROCEDURES

Hydrologic Method		Section of Manual	Peak Flow	Hydrograph
1.	Rational Method[a]	2.5.2	Yes	No
2.	SCS TR-55 Graphical	2.5.4	Yes	No
3.	SCS TR-55 Tabular	2.6.4	Yes	Yes
4.	USGS Regression Equations	2.5.3	Yes	No
5.	Unit Hydrograph Theory	2.6.1	Yes	Yes
6.	Inman's Dimensionless Hydrograph	2.6.2	Yes	Yes

Table 2-1 (continued)
Limits of Application

		Design Storm	Time of Concentration (t_c)	Drainage Area (DA)	Impervious (IMP)	I_a/P
1.	Rational Method[a]	t_c	5 min. $\leq t_c \leq$ 30 min	$\leq$ 100 acres	0-100%	N/A
2.	SCS TR-55 Graphical	24 hr Type II	0.1 hr $\leq t_c \leq$ 10 hr	b	40 $\leq$ CN $\leq$ 98	.1-.5
3.	SCS TR-55 Tabular	24 hr Type II	0.1 hr $\leq t_c \leq$ 2 hr	c	40 $\leq$ CN $\leq$ 98	.1-.5
4.	USGS Regression Equations					
	Rural	N/A	N/A	0.15 sq. mi. $\leq$ DA $\leq$ 850 sq. mi.	$\leq$ 20%	N/A
	Urban	N/A	N/A	0.15 sq. mi. $\leq$ DA $\leq$ 30 sq. mi.	20% < IMP $\leq$ 80%	N/A
5.	Unit Hydrograph	Any	> 0	> 0	0-100%	N/A
6.	Inman's Dimensionless Hydrograph[d]					
	Rural	N/A	N/A	0.17 sq. mi. $\leq$ DA $\leq$ 481 sq. mi.	<4%	N/A
	Urban	N/A	N/A	0.47 sq. mi. $\leq$ DA $\leq$ 64 sq. mi.	4-48%	N/A

[a]Use of the Rational Method beyond the limits shown requires approval by MWS, and results should be compared using other methods.

[b]A single homogeneous watershed is required. The procedure was developed from results of TR-20 (USDA, SCS, 1983) computer analysis with a DA of 1 square mile.

[c]Drainage areas of individual subareas cannot differ by a factor of 5 or more. The procedure was developed from results of TR-20 (USDA, SCS, 1983) computer analysis with a DA of 1 square mile.

[d]Drainage area and impervious limitations apply to lagtime estimates used in Inman's method; additional limitations may apply based on the method used to predict peak discharge.

N/A = Not Applicable

Table 2-2
BALANCED STORM RAINFALL HYETOGRAPH DATA
FOR METRO NASHVILLE

		Cumulative Rainfall (inches)			
Time (hr)	P/P_{24} Ratio	2-yr	10-yr	25-yr	100-yr
0.00	0.0000	0.000	0.000	0.000	0.000
0.25	0.0003	0.001	0.001	0.002	0.002
0.50	0.0005	0.002	0.003	0.003	0.004
0.75	0.0008	0.003	0.004	0.005	0.006
1.00	0.0010	0.003	0.005	0.006	0.008
1.25	0.0015	0.005	0.008	0.009	0.011
1.50	0.0020	0.007	0.010	0.012	0.015
1.75	0.0025	0.008	0.013	0.015	0.019
2.00	0.0030	0.010	0.016	0.018	0.023
2.25	0.0039	0.013	0.020	0.024	0.029
2.50	0.0048	0.016	0.025	0.029	0.036
2.75	0.0056	0.019	0.029	0.035	0.042
3.00	0.0065	0.022	0.034	0.040	0.049
3.25	0.0074	0.025	0.039	0.045	0.056
3.50	0.0083	0.028	0.043	0.051	0.062
3.75	0.0091	0.031	0.048	0.056	0.069
4.00	0.0100	0.034	0.052	0.062	0.075
4.25	0.0138	0.047	0.072	0.085	0.104
4.50	0.0175	0.059	0.092	0.108	0.132
4.75	0.0213	0.072	0.111	0.131	0.160
5.00	0.0250	0.085	0.131	0.154	0.188
5.25	0.0288	0.097	0.150	0.177	0.216
5.50	0.0325	0.110	0.170	0.200	0.245
5.75	0.0363	0.123	0.190	0.223	0.273
6.00	0.0400	0.136	0.209	0.246	0.301
6.25	0.0450	0.153	0.235	0.277	0.339
6.50	0.0500	0.169	0.262	0.308	0.377
6.75	0.0550	0.186	0.288	0.339	0.414
7.00	0.0600	0.203	0.314	0.370	0.452
7.25	0.0650	0.220	0.340	0.400	0.489
7.50	0.0700	0.237	0.366	0.431	0.527
7.75	0.0750	0.254	0.392	0.462	0.565
8.00	0.0800	0.271	0.418	0.493	0.602
8.25	0.0870	0.295	0.455	0.536	0.655
8.50	0.0940	0.319	0.492	0.579	0.708
8.75	0.1010	0.342	0.528	0.622	0.761
9.00	0.1080	0.366	0.565	0.665	0.813
9.25	0.1185	0.402	0.620	0.730	0.892
9.50	0.1290	0.437	0.675	0.795	0.971
9.75	0.1395	0.473	0.730	0.859	1.050

Table 2-2 (continued)
BALANCED STORM RAINFALL HYETOGRAPH DATA
FOR METRO NASHVILLE

		Cumulative Rainfall (inches)			
Time (hr)	P/P_{24} Ratio	2-yr	10-yr	25-yr	100-yr
10.00	0.1500	0.509	0.785	0.924	1.130
10.25	0.1675	0.568	0.876	1.032	1.261
10.50	0.1850	0.627	0.968	1.140	1.393
10.75	0.2025	0.686	1.059	1.247	1.525
11.00	0.2200	0.746	1.151	1.355	1.657
11.25	0.2450	0.831	1.281	1.509	1.845
11.50	0.2800	0.949	1.464	1.725	2.108
11.75	0.3900	1.322	2.040	2.402	2.937
12.00	0.5000	1.695	2.615	3.080	3.765
12.25	0.6080	2.061	3.180	3.745	4.578
12.50	0.7150	2.424	3.739	4.404	5.384
12.75	0.7570	2.566	3.959	4.663	5.700
13.00	0.7900	2.678	4.132	4.866	5.949
13.25	0.8075	2.737	4.223	4.974	6.080
13.50	0.8250	2.797	4.315	5.082	6.212
13.75	0.8425	2.856	4.406	5.190	6.344
14.00	0.8600	2.915	4.498	5.298	6.476
14.25	0.8688	2.945	4.544	5.352	6.542
14.50	0.8775	2.975	4.589	5.405	6.608
14.75	0.8863	3.004	4.635	5.459	6.673
15.00	0.8950	3.034	4.681	5.513	6.739
15.25	0.9008	3.054	4.711	5.549	6.783
15.50	0.9065	3.073	4.741	5.584	6.826
15.75	0.9123	3.093	4.771	5.619	6.869
16.00	0.9180	3.112	4.801	5.655	6.913
16.25	0.9226	3.128	4.825	5.683	6.947
16.50	0.9273	3.143	4.850	5.712	6.982
16.75	0.9319	3.159	4.874	5.740	7.017
17.00	0.9365	3.175	4.898	5.769	7.052
17.25	0.9411	3.190	4.922	5.797	7.087
17.50	0.9458	3.206	4.946	5.826	7.121
17.75	0.9504	3.222	4.970	5.854	7.156
18.00	0.9550	3.237	4.995	5.883	7.191
18.25	0.9581	3.248	5.011	5.902	7.215
18.50	0.9613	3.259	5.027	5.921	7.238
18.75	0.9644	3.269	5.044	5.941	7.262
19.00	0.9675	3.280	5.060	5.960	7.285
19.25	0.9706	3.290	5.076	5.979	7.309
19.50	0.9738	3.301	5.093	5.998	7.332
19.75	0.9769	3.312	5.109	6.018	7.356

Table 2-2 (continued)
BALANCED STORM RAINFALL HYETOGRAPH DATA
FOR METRO NASHVILLE

		Cumulative Rainfall (inches)			
Time (hr)	P/P_{24} Ratio	2-yr	10-yr	25-yr	100-yr
20.00	0.9800	3.322	5.125	6.037	7.379
20.25	0.9819	3.329	5.135	6.048	7.394
20.50	0.9837	3.335	5.145	6.060	7.408
20.75	0.9856	3.341	5.155	6.071	7.422
21.00	0.9875	3.348	5.165	6.083	7.436
21.25	0.9894	3.354	5.174	6.095	7.450
21.50	0.9912	3.360	5.184	6.106	7.464
21.75	0.9931	3.367	5.194	6.118	7.478
22.00	0.9950	3.373	5.204	6.129	7.492
22.25	0.9956	3.375	5.207	6.133	7.497
22.50	0.9963	3.377	5.210	6.137	7.502
22.75	0.9969	3.379	5.214	6.141	7.506
23.00	0.9975	3.382	5.217	6.145	7.511
23.25	0.9981	3.384	5.220	6.148	7.516
23.50	0.9987	3.386	5.223	6.152	7.521
23.75	0.9994	3.388	5.227	6.156	7.525
24.00	1.0000	3.390	5.230	6.160	7.530

Table 2-3
RUNOFF COEFFICIENTS[a] FOR A DESIGN STORM RETURN PERIOD OF 10 YEARS OR LESS

Slope	Typical Land Use	Sandy Soils Min.	Sandy Soils Max.	Clay Soils Min.	Clay Soils Max.
Flat	Woodlands	0.10	0.15	0.15	0.20
(0-2%)	Pasture, grass, and farmland[b]	0.15	0.20	0.20	0.25
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavements[c]	0.75	0.95	0.90	0.95
Rolling	Woodlands	0.15	0.20	0.20	0.25
(2-7%)	Pasture, grass, and farmland[b]	0.20	0.25	0.25	0.30
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavements[c]	0.80	0.95	0.90	0.95
Steep	Woodlands	0.20	0.25	0.25	0.30
(7%+)	Pasture, grass, and farmland[b]	0.25	0.35	0.30	0.40
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavements[c]	0.85	0.95	0.90	0.95

[a]Weighted coefficient based on percentage of impervious surfaces and green areas must be selected for each site.

[b]Coefficients assume good ground cover and conservation treatment.

[c]Depends on depth and degree of permeability of underlying strata.

Table 2-3 (continued)

Specific Zoning Classification	Runoff Coefficients
Residential	
AR2a, R2a	0.25 – 0.35
RS40, R40, RS30, R30, RS20, R20	0.40 – 0.50
RS15, R15	0.45 – 0.55
RS10, R10, RS8, R8	0.55 – 0.65
RM8, RM6, RS6, R6	0.65 – 0.75
Commercial	
CH, CSL, CS, CG, CF, CC	0.80 – 0.90
OP, OG, MUL, MU, MRO, MO	0.70 – 0.80
Industrial	
IR, IG	0.80 – 0.90

Note: For specific zoning classifications, the lowest range of runoff coefficients should be used for flat areas (areas where the majority of the grades and slopes are 2 percent and less). The average range of runoff coefficients should be used for intermediate areas (areas where the majority of the grades and slopes are from 2 percent to 7 percent). The highest range of runoff coefficients should be used for steep areas (areas where the majority of the grades and slopes are greater than 7 percent).

Reference: Coefficient values adapted from DeKalb County (1976). Zoning classification data derived from Zoning Regulations of the Metro Government of Nashville and Davidson County, Tennessee (September 1987).

Table 2-4
DESIGN STORM FREQUENCY FACTORS
FOR PERVIOUS AREA RUNOFF COEFFICIENTS

Return period (years)	Design Storm Frequency Factor, X_T
2 to 10	1.0
25	1.1
50	1.2
100	1.25

Reference: Wright-McLaughlin Engineers (1969).

Table 2-5
RUNOFF CURVE NUMBERS FOR URBAN AREAS[a]

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	Average Percent Impervious Area[b]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawn, parks, golf courses, cemeteries, etc.)[c]:					
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
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
¼ acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
½ acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing Urban Areas					
Newly graded areas (previous areas only, no vegetation)[d]		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 2-6)					

[a]Average runoff condition, and $I_a = 0.2S$.

[b]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.

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

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

Reference: USDA, SCS, TR-55 (1986).

Table 2-6
RUNOFF CURVE NUMBERS FOR RURAL AREAS[a]

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type	Hydrologic Condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing[b]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay	--	30	58	71	78
Brush—brush—weed—grass mixture with brush the major element[c]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[d]	48	65	73
Woods—grass combination (orchard or tree farm)[e]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[f]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[d]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots	--	59	74	82	86

[a]Average runoff condition, and $I_a = 0.2S$.

[b]Poor: <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[c]Poor: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[d]Actual curve number is less than 30; use CN = 30 for runoff computations.

[e]CNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pastures.

[f]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.

Reference: USDA, SCS, NEH-4 (1972).

Table 2-7
OVERLAND FLOW MANNING'S n VALUES

	Recommended Value	Range of Values
Concrete	.011	.01 – .013
Asphalt	.012	.01 - .015
Bare sand[a]	.010	.010 - .016
Graveled surface[a]	.012	.012 - .030
Bare clay-loam (eroded) [a]	.012	.012 - .033
Fallow (no residue) [b]	.05	.006 - .16
Chisel plow (<1/4 ton/acre residue)	.07	.006 - .17
Chisel plow (1/4 – 1 ton/acre residue)	.18	.07 - .34
Chisel plow (1 – 3 tons/acre residue)	.30	.19 - .47
Chisel plow (>3 tons/acre residue)	.40	.34 - .46
Disk/harrow (<1/4 ton/acre residue)	.08	.008 - .41
Disk/harrow (1/4 – 1 ton/acre residue)	.16	.10 - .25
Disk/harrow (1 – 3 tons/acre residue)	.25	.14 - .53
Disk/harrow (>3 tons/acre residue)	.30	-- --
No till (<1/4 ton/acre residue)	.04	.03 - .07
No till (1/4 – 1 ton/acre residue)	.07	.01 - .13
No till (1 –3 tons/acre residue)	.30	.16 - .47
Plow (fall)	.06	.02 - .10
Coulter	.10	.05 - .13
Range (natural)	.13	.01 - .32
Range (clipped)	.08	.02 - .24
Grass (bluegrass sod)	.45	.39 - .63
Short grass praire[a]	.15	.10 - .20
Dense grass[c]	.24	.17 - .30
Bermudagrass[c]	.41	.30 - .48
Woods	.45	-- --

[a]Woolhiser (1975).

[b]Fallow has been idle for one year and is fairly smooth.

[c]Palmer (1946). Weeping lovegrass, bluegrass, buffalo grass, blue gramma grass, native grass mix (OK), alfalfa, lespedeza.

Note: These values were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations. See Chapter 3 for open channel flow procedures.

Reference: Engman (1983), unless noted otherwise.

Table 2-8
CURRENT (2000) STREAMFLOW MONITORING SITES
METRO NASHVILLE AND DAVIDSON COUNTY[a]

03426310	Cumberland River at Old Hickory Dam (Tw), TN
03426385	Mansker Creek above Goodlettsville, TN
03426470	Dry Creek near Edenwold, TN
03426500	Cumberland River below Old Hickory, TN
03430100	Stones River below J. Percy Priest Dam, TN
03430118	McCrory Creek at Ironwood Drive at Donelson, TN
03430147	Stoners Creek near Hermitage, TN
03430550	Mill Creek near Nolensville, TN
03430600	Mill Creek at Hobson Pike near Antioch, TN
03430700	Indian Creek at Pettus Road at Nashville, TN
03431000	Mill Creek near Antioch, TN
03431020	Sorgum Branch at Antioch Pike near Antioch, TN
03431040	Sevenmile Creek at Blackman Road near Nashville, TN
03431060	Mill Creek at Thompson Lane near Woodbine, TN
03431062	Mill Creek Tributary at Glenrose Avenue at Woodbine, TN
03431080	Sims Branch at Elm Hill Pike near Donelson, TN
03431100	W. F. Browns Creek at Glendale Lane at Nashville, TN
03431120	W. F. Browns Creek at General Bates Drive, at Nashville, TN
03431160	M. F. Browns Creek at Overbrook Drive at Nashville, TN
03431200	Browns Creek at Berry Lane at Nashville, TN
03431240	E. F. Browns Creek at Baird-Ward Printing Co., Nashville, TN
03431300	Browns Creek at State Fairgrounds at Nashville, TN
03431340	Browns Creek at Factory Street at Nashville, TN
03431490	Pages Branch at Avondale, TN
03431500	Cumberland River at Nashville, TN
03431505	Cumberland River at Woodland Street at Nashville, TN
03431517	Cummings Branch at Lickton, TN
03431520	Claylick Creek at Lickton, TN
03431530	Whites Creek at Old Hickory Blvd. at Whites, Creek, TN
03431550	Earthman Fork at Whites Creek, TN
03431560	Whites Creek at Whites Creek Pike at Whites Creek, TN
03431573	Ewing Creek at Richmond Hill Drive at Parkwood, TN
03431575	Ewing Creek at Brick Church Pike at Parkwood, TN
03431578	Ewing Creek at Gwynwood Drive near Jordania, TN
03431580	Ewing Creek at Knight Road near Bordeaux, TN
03431581	Ewing Creek below Knight Road near Bordeaux, TN
03431599	Whites Creek near Bordeaux, TN
03431600	Whites Creek at Tucker Road near Bordeaux, TN

Table 2-8 (continued)

03431610	Eaton Creek at Cato Road near Bordeaux, TN
03431630	Richland Creek at Lynnwood Blvd. at Belle Meade, TN
03431640	Belle Meade Branch at B M Blvd., Belle Meade, TN
03431650	Vaughns Gap Br at Percy Warner Belle, Meade, TN
03431660	Jocelyn Hollow Br at Post Rd at Belle Meade, TN
03431670	Richland Creek at Fransworth Dr. at Belle Meade, TN
03431677	Sugartree Creek at YMCA Access Road at Green Hills, TN
03431679	Sugartree Creek at Abbott Martin Road at Green Hills, TN
03431680	Sugartree Creek at Cross Creek Rd at Nashville, TN
03431700	Richland Creek at Charlotte Avenue at Nashville, TN
03433500	Harpeth River at Bellevue, TN

[a]Additional information and data from these monitoring sites can be downloaded from the USGS web site at http://waterdata.usgs.gov/.

Table 2-9
USGS REGRESSION EQUATION PARAMETERS

Rural Regression Equations

T	CR_T	XT	Standard Error of Estimate (%)	Equivalent Years of Record
2	319	0.733	33	3
5	512	0.725	30	4
10	651	0.723	30	6
25	836	0.720	31	8
50	977	0.720	32	8
100	1,125	0.719	34	9

Reference: Randolph and Gamble (1976).

Urban Regression Equations

T	CR_T	XT	YT	Standard Error of Estimate (%)	Equivalent Years of Record
2	76.4	0.74	0.48	44	2
5	132	0.75	0.44	39	3
10	168	0.75	0.43	37	4
25	234	0.75	0.39	36	6
50	266	0.75	0.40	37	7
100	305	0.75	0.40	39	8

Note: See Section 2.5.3 for details regarding the equations.

Reference: Robbins (1984b).

Table 2-10
I_a VALUES FOR RUNOFF CURVE NUMBERS

Curve Number	I_a (inches)	Curve Number	I_a (inches)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Reference: USDA, SCS, TR-55 (1986).

Table 2-11
SCS DIMENSIONLESS UNIT HYDROGRAPH RATIOS

Time Ratios (t/t_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Qa/Q)
0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.012
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.055	.999
5.0	.000	1.000

Reference: USDA, SCS, NEH-4 (1972).

Table 2-12
TIME AND DISCHARGE RATIOS OF INMAN'S DIMENSIONLESS HYDROGRAPH

Time Ratio (t/LT)	Discharge Ratio (Q_t/Q_p)
0.25	0.12
.30	.16
.35	.21
.40	.26
.45	.33
.50	.40
.55	.49
.60	.58
.65	.67
.70	.76
.75	.84
.80	.90
.85	.95
.90	.98
.95	1.00
1.00	.99
1.05	.96
1.10	.92
1.15	.86
1.20	.80
1.25	.74
1.30	.68
1.35	.62
1.40	.56
1.45	.51
1.50	.47
1.55	.43
1.60	.39
1.65	.36
1.70	.33
1.75	.30
1.80	.28
1.85	.26
1.90	.24
1.95	.22
2.00	.20
2.05	.19
2.10	.17
2.15	.16
2.20	.15
2.25	.14

Table 2-12 (continued)
TIME AND DISCHARGE RATIOS OF INMAN'S DIMENSIONLESS HYDROGRAPH

Time Ratio (t/LT)	Discharge Ratio (Q_t/Q_p)
2.30	.13
2.35	.12
2.40	.11

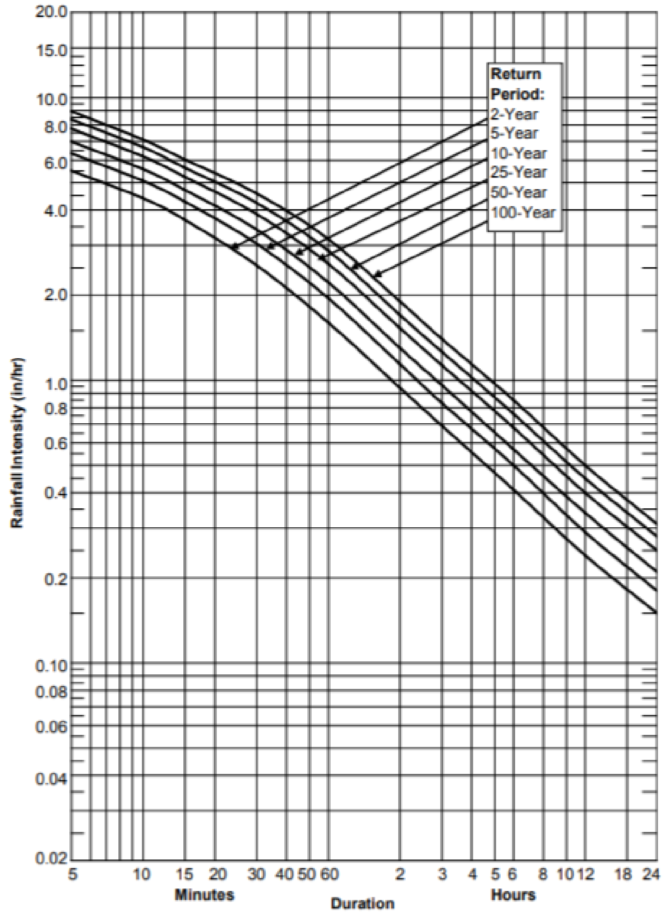
Reference: Robbins (1986).

Table 2-13
HYDROGRAPH COMPUTATION, EXAMPLE 2-9

UNIT HYDROGRAPH (cfs/in)		0	10	31	65	92	99	92	77	55	39	28	20	15	11	8	5	4	3	2	1	1	1	0	0	0	0
TIME t (hour)	RUNOFF[a] TIME (min)	0	5	10	14	19	24	29	34	38	43	48	53	58	62	67	72	77	82	86	91	96	101	106	110	115	120
0.00	Incremental																										
0.08	Rainfall																										
0.16	Excess																										
0.24	(in)																										
0.32	0.03	0.0	0.3	0.9	1.8	2.5	2.7	2.5	2.1	1.5	1.1	0.8	0.6	0.4	0.3	0.2	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.40	0.10		0.0	1.0	3.2	6.8	9.6	10.4	9.6	8.1	5.8	4.1	2.9	2.1	1.6	1.2	0.8	0.5	0.4	0.3	0.2	0.1	0.1	0.1	0.0	0.0	0.0
0.48	0.26			0.0	2.6	8.2	17.2	24.3	26.1	24.3	20.3	14.5	10.3	7.4	5.3	4.0	2.9	2.1	1.3	1.1	0.8	0.5	0.3	0.3	0.3	0.0	0.0
0.56	0.15				0.0	1.5	4.7	9.9	14.0	15.0	14.0	11.7	8.3	5.9	4.2	3.0	2.3	1.7	1.2	0.8	0.6	0.5	0.3	0.2	0.2	0.2	0.0
0.64	0.12					0.0	1.2	3.8	7.9	11.2	12.0	11.2	9.4	6.7	4.7	3.4	2.4	1.8	1.3	1.0	0.6	0.5	0.4	0.2	0.1	0.1	0.1
0.72	0.09						0.0	0.9	2.8	5.8	8.2	8.8	8.2	6.8	4.9	3.5	2.5	1.8	1.3	1.0	0.7	0.4	0.4	0.3	0.2	0.1	0.1
0.80	0.07							0.0	0.7	2.1	4.5	6.3	6.8	6.3	5.3	3.8	2.7	1.9	1.4	1.0	0.8	0.6	0.3	0.3	0.2	0.1	0.1
0.88	0.06								0.0	0.6	1.8	3.7	5.2	5.6	5.2	4.4	3.1	2.2	1.6	1.1	0.9	0.6	0.5	0.3	0.2	0.2	0.1
0.96	0.05									0.0	0.5	1.6	3.4	4.8	5.2	4.8	4.0	2.9	2.0	1.5	1.0	0.8	0.6	0.4	0.3	0.2	0.2
1.04	0.05										0.0	0.5	1.4	2.9	4.2	4.5	4.2	3.5	2.5	1.8	1.3	0.9	0.7	0.5	0.4	0.2	0.2
1.12	0.05											0.0	0.5	1.6	3.3	4.6	5.0	4.6	3.9	2.8	2.0	1.4	1.0	0.8	0.6	0.4	0.3
1.20	0.03												0.0	0.3	0.9	2.0	2.8	3.0	2.8	2.3	1.7	1.2	0.8	0.6	0.5	0.3	0.2
1.28	0.02													0.0	0.2	0.6	1.3	1.8	2.0	1.8	1.5	1.1	0.8	0.6	0.4	0.3	0.2
1.36	0.01														0.0	0.1	0.3	0.7	0.9	1.0	0.9	0.8	0.6	0.4	0.3	0.2	0.2
1.44	0.01															0.0	0.1	0.3	0.7	0.9	1.0	0.9	0.8	0.6	0.4	0.3	0.2
1.52																											
1.60	TOTAL[b]																										
1.68	RUNOFF																										
1.76	(cfs)	0	0	2	8	19	35	52	63	69	68	63	57	51	45	40	34	29	23	18	14	10	7	5	4	3	2
1.84																											
1.92																											
2.00																											

[a]Beginning at 53 minutes.

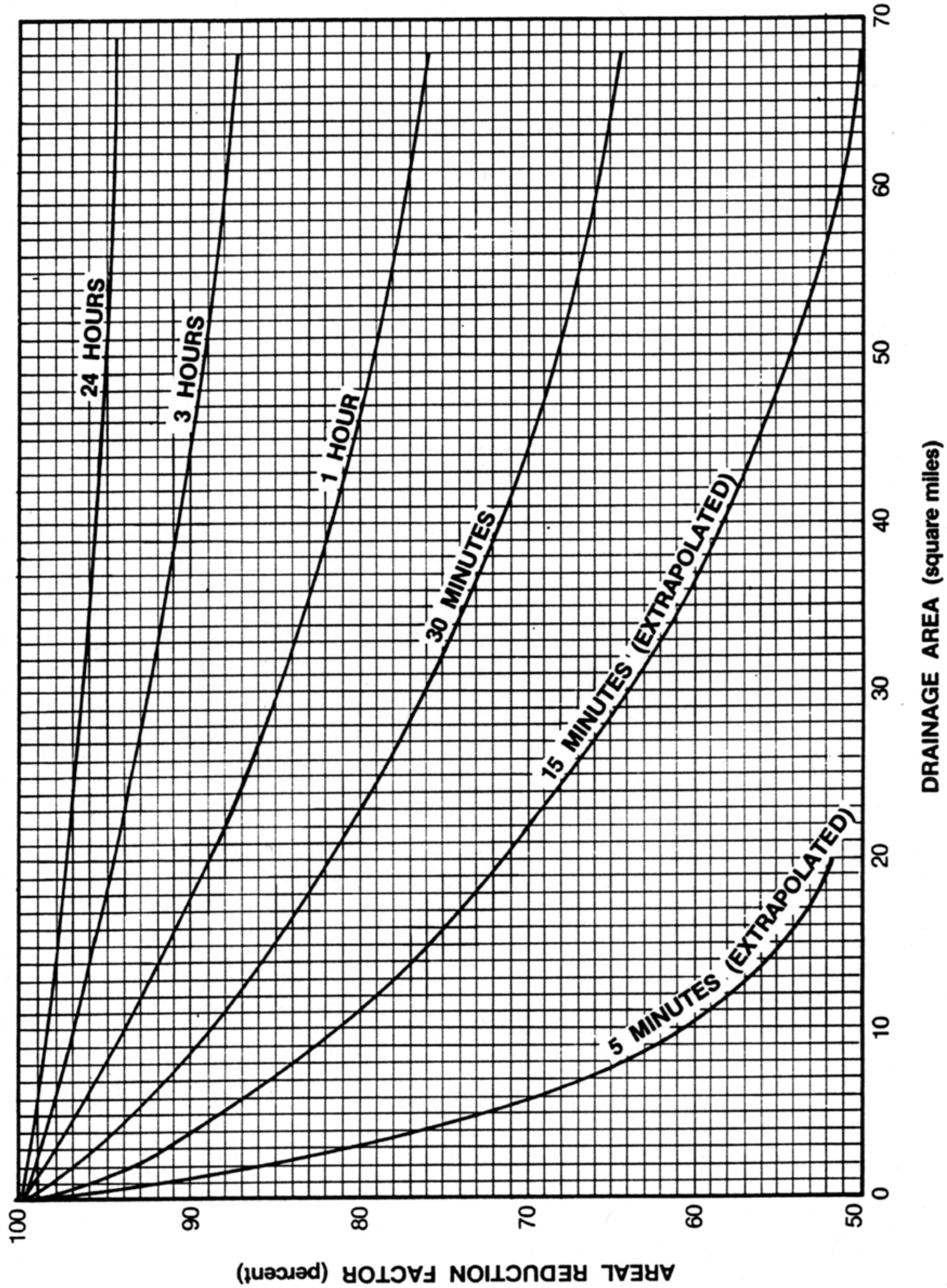
[b]Sum of incremental flow rates = 703 cfs.



RAINFALL VOLUME (inches)										
	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1-Year	0.38	0.61	0.76	1.04	1.30	1.54	1.67	2.00	2.37	2.83
2-Year	0.45	0.71	0.90	1.24	1.56	1.83	1.99	2.38	2.82	3.37
5-Year	0.51	0.82	1.04	1.48	1.90	2.23	2.41	2.88	3.42	4.11
10-Year	0.57	0.91	1.15	1.67	2.17	2.54	2.76	3.31	3.92	4.70
25-Year	0.64	1.01	1.29	1.90	2.54	2.97	3.23	3.90	4.62	5.53
50-Year	0.69	1.09	1.39	2.09	2.83	3.31	3.61	4.38	5.19	6.20
100-Year	0.74	1.17	1.48	2.27	3.12	3.67	4.01	4.88	5.79	6.89

RAINFALL INTENSITY (inches/hour)										
	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1-Year	4.57	3.65	3.04	2.08	1.30	0.77	0.56	0.34	0.20	0.12
2-Year	5.35	4.28	3.59	2.48	1.56	0.92	0.66	0.40	0.23	0.14
5-Year	6.17	4.94	4.16	2.96	1.90	1.11	0.80	0.48	0.28	0.17
10-Year	6.82	5.45	4.60	3.33	2.17	1.27	0.92	0.55	0.33	0.20
25-Year	7.63	6.08	5.14	3.81	2.54	1.48	1.08	0.65	0.38	0.23
50-Year	8.24	6.56	5.54	4.17	2.83	1.66	1.20	0.73	0.43	0.26
100-Year	8.83	7.02	5.92	4.53	3.12	1.83	1.33	0.82	0.48	0.29

Figure 2-1
Intensity-Duration-Frequency Curves and Depth-Duration Data
(Source: NOAA Atlas 14, Volume 2, Version 3)



Reference: Hershfield (1961).

Figure 2-2
Areal Reduction Factors for Precipitation Durations from 5 Minutes to 24 Hours

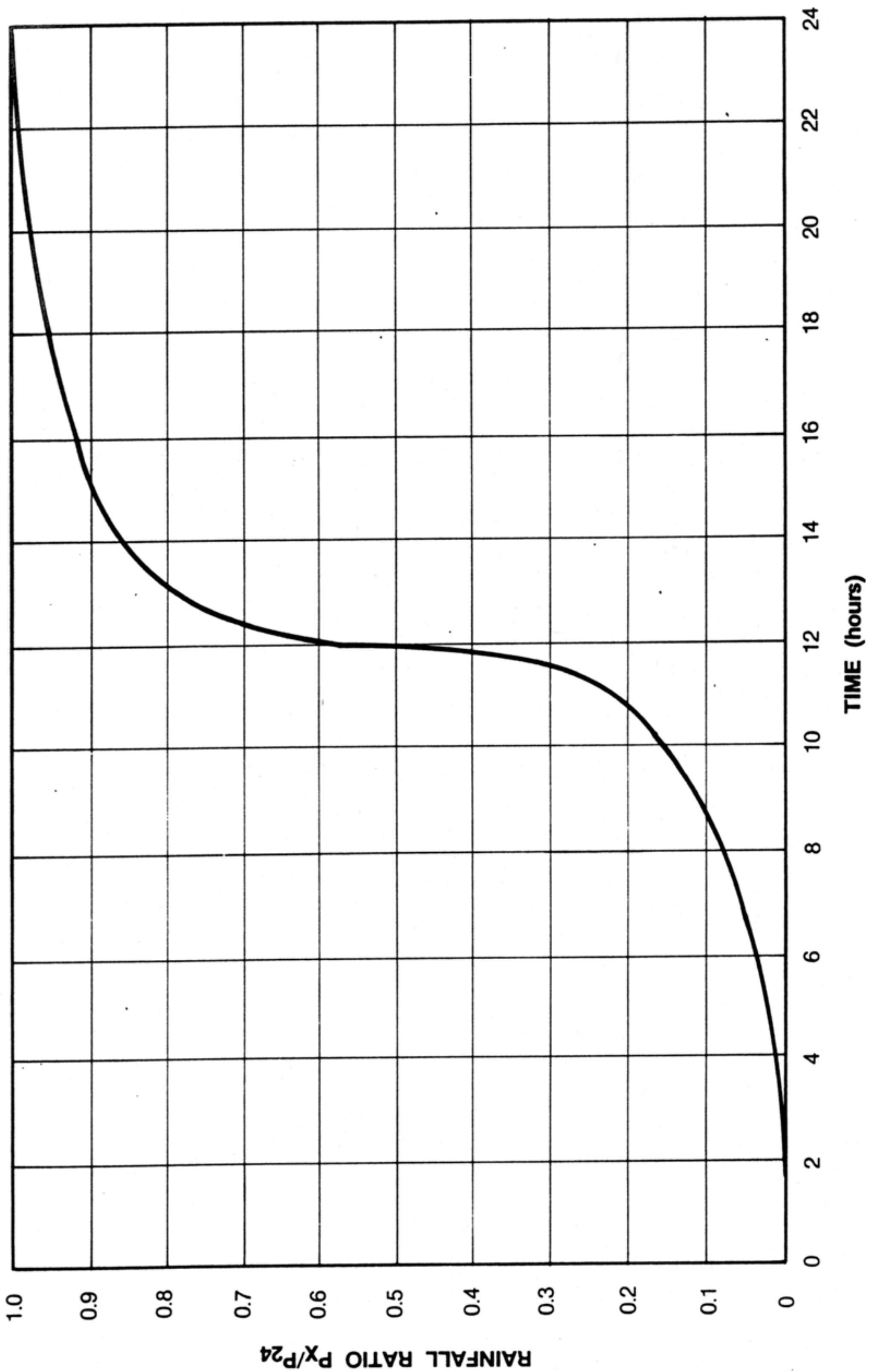


Figure 2-3
24-Hour Rainfall Hyetograph for Metro Nashville Area

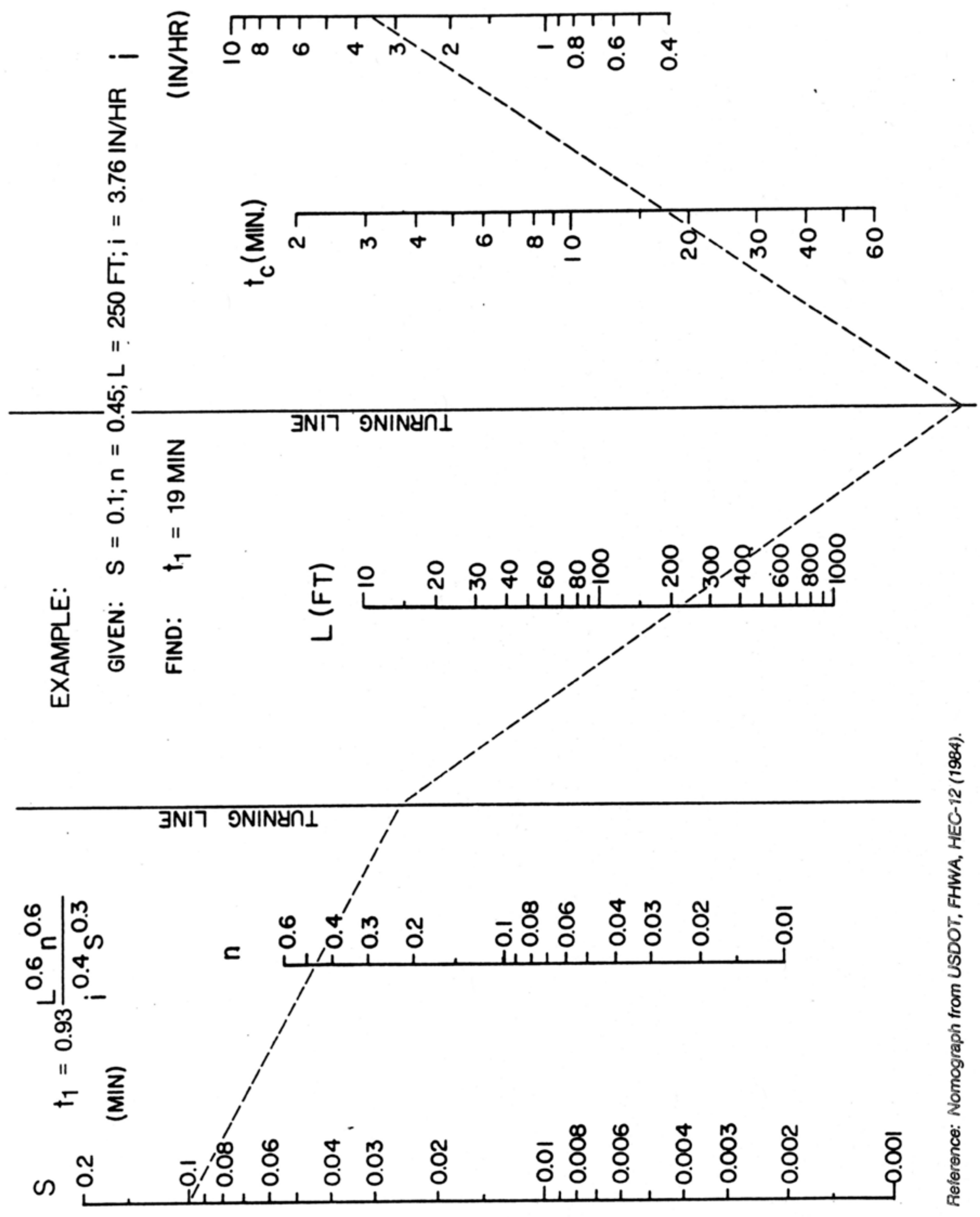
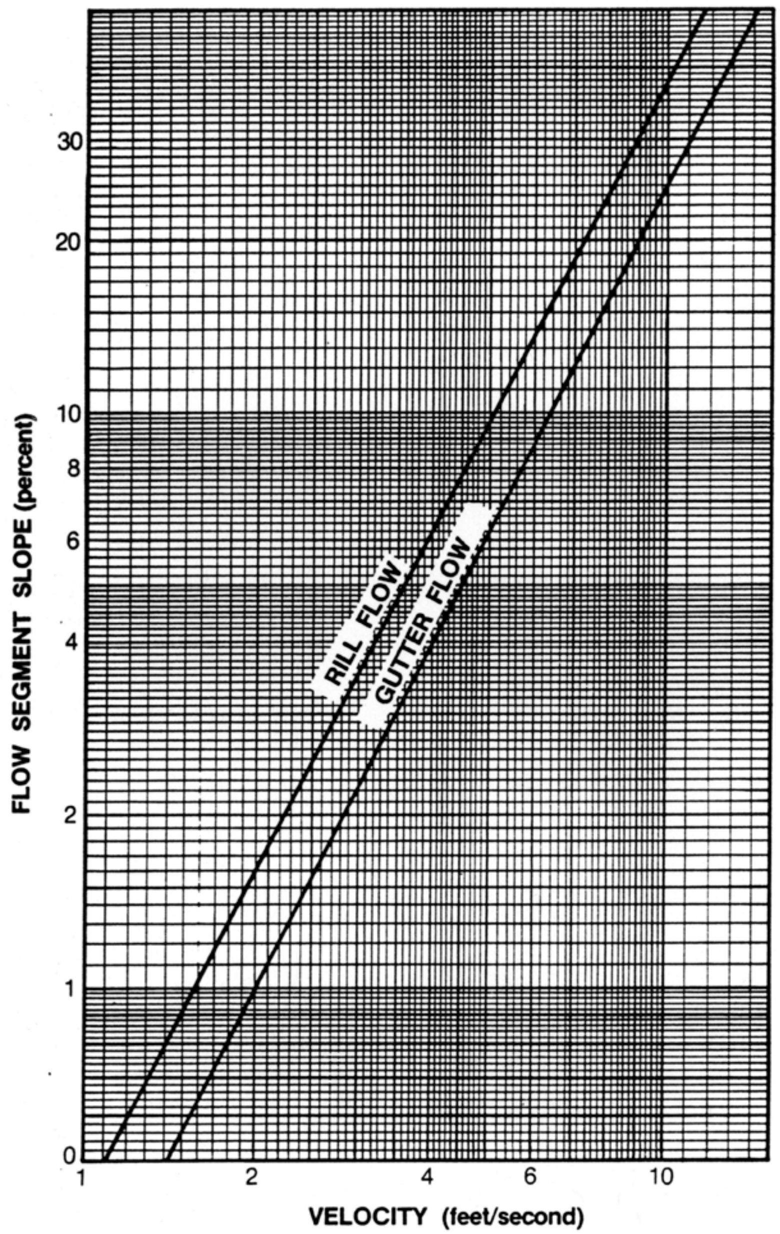


Figure 2-4
Kinematic Wave Nomograph for Estimating Overland Flow Travel Time



Reference: USDA, SCS, TR-55 (1986).

Figure 2-5
Average Velocities for Estimating Travel Time for Shallow Channel Flow

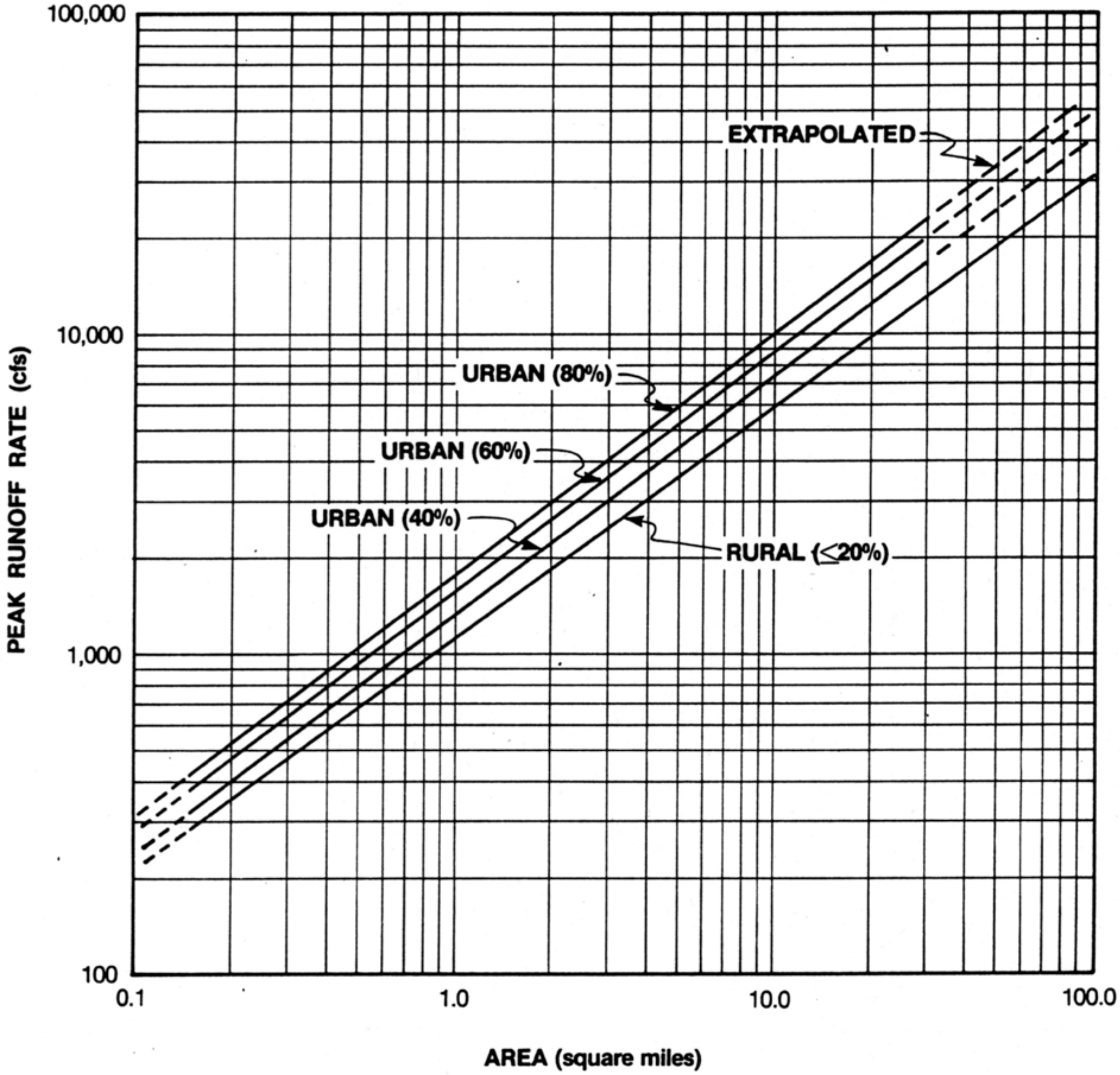


Figure 2-6
100-Year Return Period
Peak Runoff Rate Regression Equations

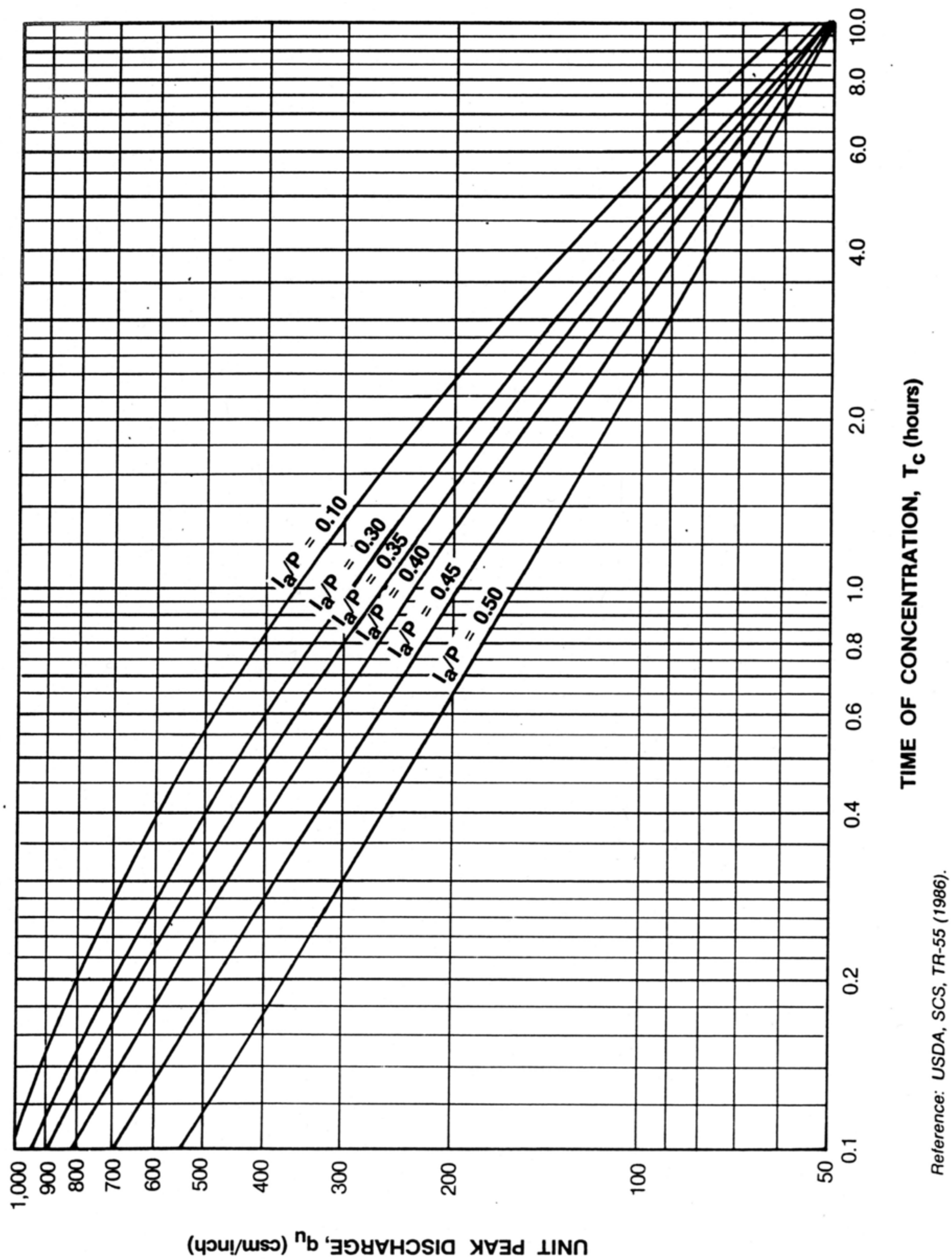
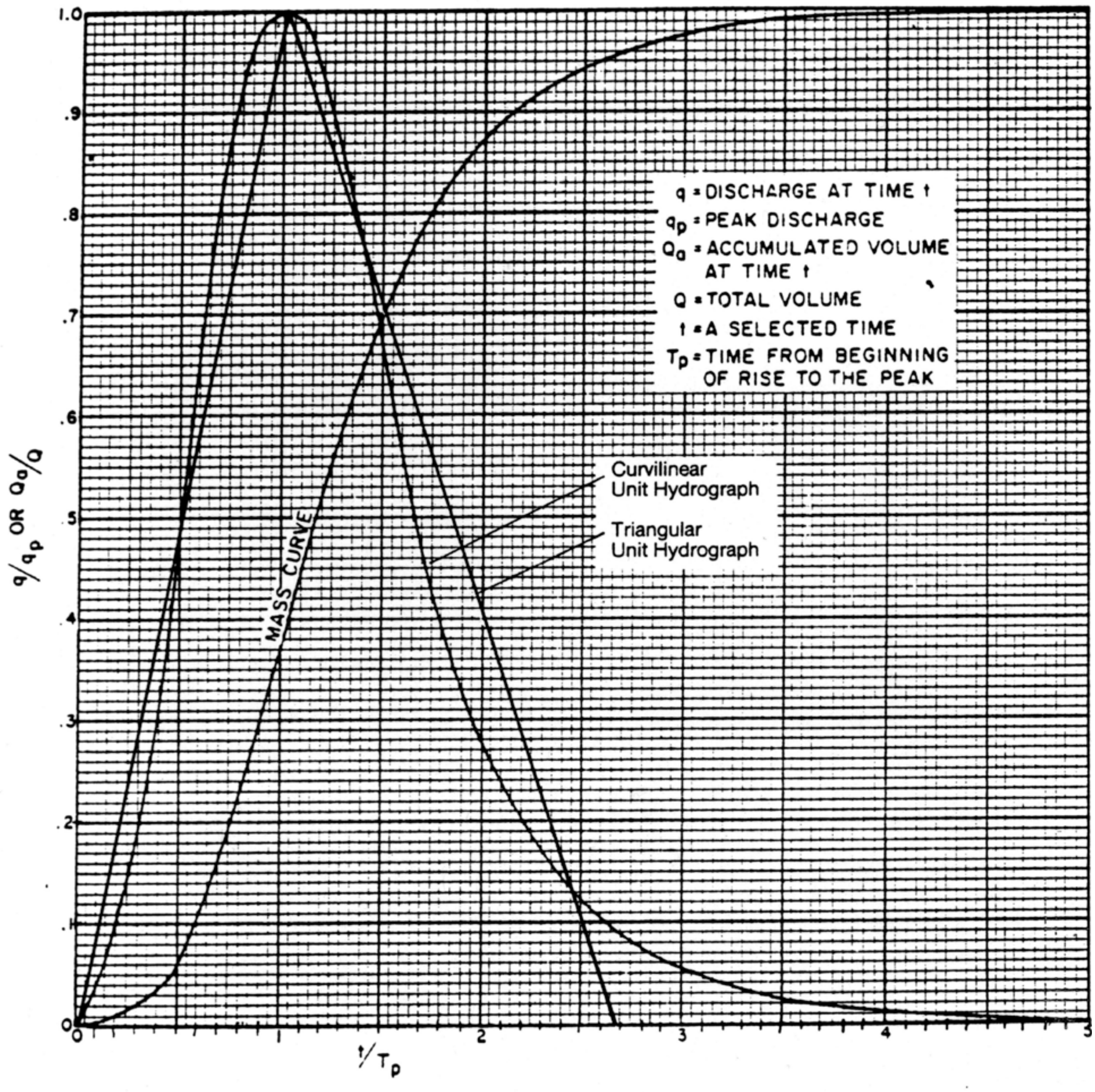


Figure 2-7
Unit Peak Discharge, q_u, for SCS Type II Rainfall Distribution



Reference: USDA, SCS, NEH-4 (1972).

Figure 2-8
SCS Dimensionless Unit Hydrograph and Mass Curve