VOLUME 2
Procedures
STORMWATER MANAGEMENT MANUAL

Prepared for

**METROPOLITAN GOVERNMENT
NASHVILLE AND DAVIDSON COUNTY**

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Contents

List of Tables

List of Figures

CHAPTER 1
INTRODUCTION

Chapter 1
INTRODUCTION

1.1 Purpose

This volume of the Nashville Stormwater Management Manual has been prepared by the Metropolitan Department of Water and Sewerage Services (MWS), as authorized by Ordinance No. 78-840 and approved by the Mayor, to establish technical guidelines to enforce the terms of that ordinance. It provides a compilation of readily available literature relevant to stormwater management activities in Nashville and Davidson County. Although it is intended to establish uniform design practices, it neither replaces the need for engineering judgment nor precludes the use of information not presented. It supports changes to Volume 1 - Regulations adopted in September 1999 and includes references to and from Volume 4 – Best Management Practices released in March 2000.

1.2 Format and Contents

The Nashville Stormwater Management Manual is presented in four volumes:

Volume 1 – Regulations
Volume 2 – Procedures
Volume 3 – Theory
Volume 4 – Best Management Practices (BMP) Manual

Volume 2 includes the following chapters:

1. Introduction
2. Hydrology
3. Open Channel Hydraulics
4. Gutter and Inlet Hydraulics
5. Culvert Hydraulics
6. Storm Sewer Hydraulics
7. Bridge Hydraulics
8. Detention/Retention Hydraulics
9. Erosion and Sediment Control
10. Outlet Protection
11. Data Collection
12. Computer Programs

Titles for Chapters 1 through 10 coincide with those for Volume 3, which does not include material related to the topics of Chapters 11 and 12. However, it should be noted that Volume 3 was not updated in the 1999/2000 revision period.

Tables and figures for Volume 2 are located at the end of each chapter, with tables preceding figures. Each chapter is introduced by a detailed table of contents that includes table and figure titles. Cited literature for the entire volume is listed alphabetically by author after the last chapter. Terms and symbols are defined immediately after each equation. Additional material appended to the manual but bound separately includes watershed master plans available from MWS. Example problems are provided at the end of selected sections to demonstrate the application of procedures. Application of site specific procedures addressing stormwater quality are presented in Volume 4 – Best Management Practices. A summary list of example problems is given below.

1.3 Limitations

The procedures in Volume 2 include original material as well as frequently used charts, equations, computation forms, and figures duplicated from readily available publications. Textual material from these publications has not been duplicated in its entirety, and the user is encouraged to obtain original or additional reference material, as appropriate. Only desktop procedures are developed in this manual. A guide to selected computer programs is provided in Chapter 12. Computer applications require appropriate user documentation and experience and should be considered for the following types of stormwater calculations:

1. Water surface profile computations, particularly for nonuniform flow conditions.

2. Development of runoff hydrographs.

3. Routing of runoff hydrographs through complex open channel, storm sewer, and storage basin facilities.

4. Bridge hydraulic evaluations.

Careful consideration should always be given to site conditions, project requirements, and engineering experience so that procedures are properly applied and adapted as needed.

1.4 Updating

This volume of the Nashville Stormwater Management Manual will be updated and revised, as necessary, to reflect up-to-date engineering practices and information applicable to Metropolitan Nashville and Davidson County.

Questions and comments should also be submitted to MWS:

Attn: Stormwater Development Review
Metro Water Services
800 Second Avenue S
Nashville, Tennessee 37210

Copies of this and other volumes of the manual may be downloaded from the MWS web site:

http://www.nashville.gov/stormwater

CHAPTER 2
HYDROLOGY

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 of MWS. 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

MWS has developed IDF curves for use in Nashville and Davidson County based on National Weather Service (NWS) data that replace previous data and precipitation records from the Metro Nashville Airport. IDF curves for durations up to 100 minutes are presented in Figure 2-1 for return periods of 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 representative for any single point in Davidson County; 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 4.50 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

   $P_{1\ hour}$ = 4.50 inches x 0.0010

   $P_{1\ hour}$ = 0.0045 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	0.0045
2.00	0.0030	0.014
3.00	0.0065	0.029
4.00	0.0100	0.045
5.00	0.0250	0.113
6.00	0.0400	0.180
7.00	0.0600	0.270
8.00	0.0800	0.360
9.00	0.1080	0.486
10.00	0.1500	0.675
11.00	0.2200	0.990
12.00	0.5000	2.25
13.00	0.7900	3.56
14.00	0.8600	3.87
15.00	0.8950	4.03
16.00	0.9180	4.13
17.00	0.9365	4.21
18.00	0.9550	4.30
19.00	0.9675	4.35
20.00	0.9800	4.41
21.00	0.9875	4.44
22.00	0.9950	4.48
23.00	0.9975	4.49
24.00	1.0000	4.50

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:

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

where:

$\overline{\mathrm{CN}}$ = 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 6.16 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:

   $R_{25\ (RM)} = 0.58\ (6.16)$ inches

   $R_{25\ (RM)} = 3.6$ 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 4.57 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:

$$R_{10\,(SCS)} = \frac{[4.57 - 0.2(3.51)]^2}{4.57 + 0.8(3.51)}$$

$$R_{10\,(SCS)} = \frac{(3.9)^2}{7.4}$$

$$R_{10\,(SCS)} = 2.1 \text{ inches}$$

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 3.39 inches.

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

$$t_1 = 0.42 \left(\frac{(0.45 \times 250)^{0.8}}{(3.39)^{0.5} (0.10)^{0.4}} \right)$$

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,

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

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 3.76 inches/hour.

Assume the rainfall intensity to be 3.76 inches/hour (see Figure 2-4).

$$t_1 = 0.93\left(\frac{(250)^{0.6}(0.45)^{0.6}}{(3.76)^{0.4}(0.10)^{0.3}}\right)$$

$t_1 = 19$ minutes

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

$t_c = 19 + 4.8 + 4.9$

$t_c = 29$ minutes

From Figure 2-1, for $t_c = 29$ minutes, the 10-year return frequency rainfall intensity is 3.48 inches/hour.

Trial rainfall intensity and computed rainfall intensity do not agree.

6. Repeat computation assuming rainfall intensity is 3.48 inches/hour.

$$t_1 = 0.93\left(\frac{(250)^{0.6}(0.45)^{0.6}}{(3.48)^{0.4}(0.10)^{0.3}}\right)$$

t_1 = 19.2 minutes

t_c = 29 minutes

From Figure 2-1, for t_c = 29 minutes, rainfall intensity is 3.48 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

Example 2-5. Rational Method Peak Runoff Rate

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 = 3.65 inches/hour
   b. For t_c = 29 minutes, I = 4.12 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) (3.65) (50)

   Q_{25} = 106 cfs

   b. Using the kinematic wave equation for t_c:

   Q_{25} = (0.58) (4.12) (50)

   Q_{25} = 119 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

<u>Example 2-7. SCS TR-55 Graphical Method Peak Runoff Rate</u>

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 6.16 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.

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

$$R_{25\,(SCS)} = \frac{[6.16 - 0.2(3.51)]^2}{6.16 + 0.8(3.51)}$$

$R_{25\,(SCS)} = 3.3$ inches

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

6. The I_a/P ratio is

   $I_a/P = 0.703/6.16$

   $I_a/P = 0.11$

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 0.11, 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:

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

where:

ΔD = 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:

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

where:

t_p = Time to peak, in minutes

ΔD = 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 ΔD 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

Example 2-8. SCS Dimensionless Unit Hydrograph

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 3.36 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, $t_c \cong 30$ 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:

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

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+ΔY) = Flow rate at the new depth Y + ΔY, 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}:

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

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):

$$K = \frac{\Delta L}{v} \qquad (2\text{-}24)$$

where:

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

ΔL = 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:

$$X = 0.5\left(1 - \frac{Q}{BS_o v \Delta L}\right) \qquad (2\text{-}25)$$

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

ΔL = 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:

$$O_2 = C_0I_2 + C_1I_1 + C_2O_1 \qquad (2\text{-}30)$$

where:

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

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

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

O_1 = Outflow rate at the beginning of routing time period)t, 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

		Sandy Soils		Clay Soils	
Slope	Typical Land Use	Min.	Max.	Min.	Max.
Flat (0-2%)	Woodlands	0.10	0.15	0.15	0.20
	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 (2-7%)	Woodlands	0.15	0.20	0.20	0.25
	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 (7%+)	Woodlands	0.20	0.25	0.25	0.30
	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

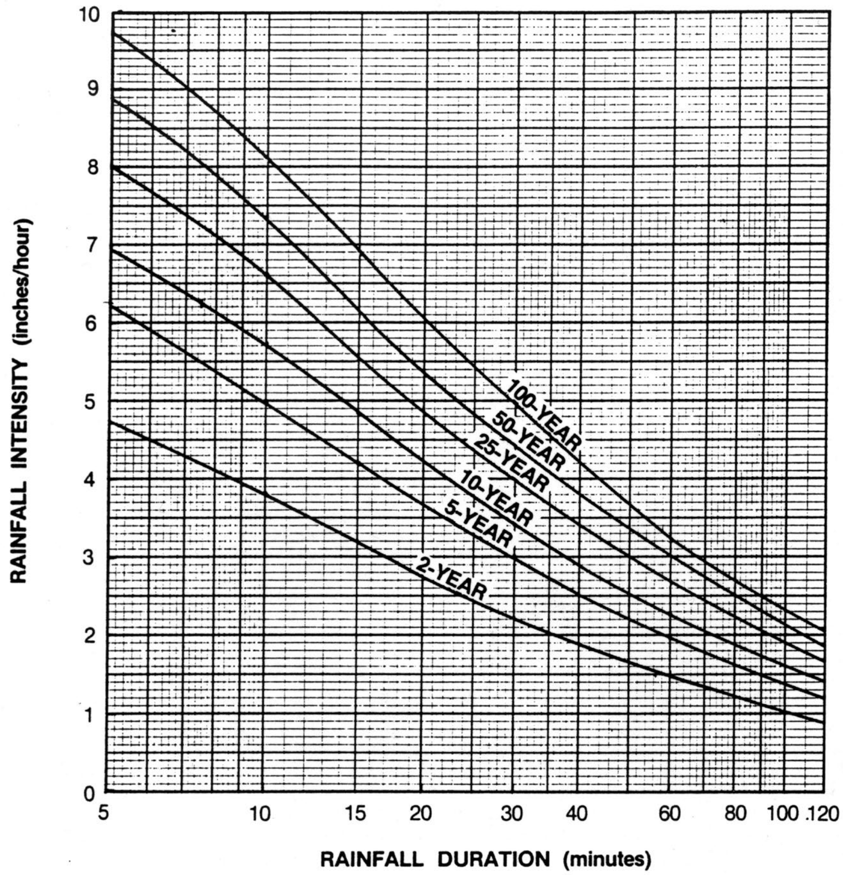
Reference: Robbins (1986).

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.

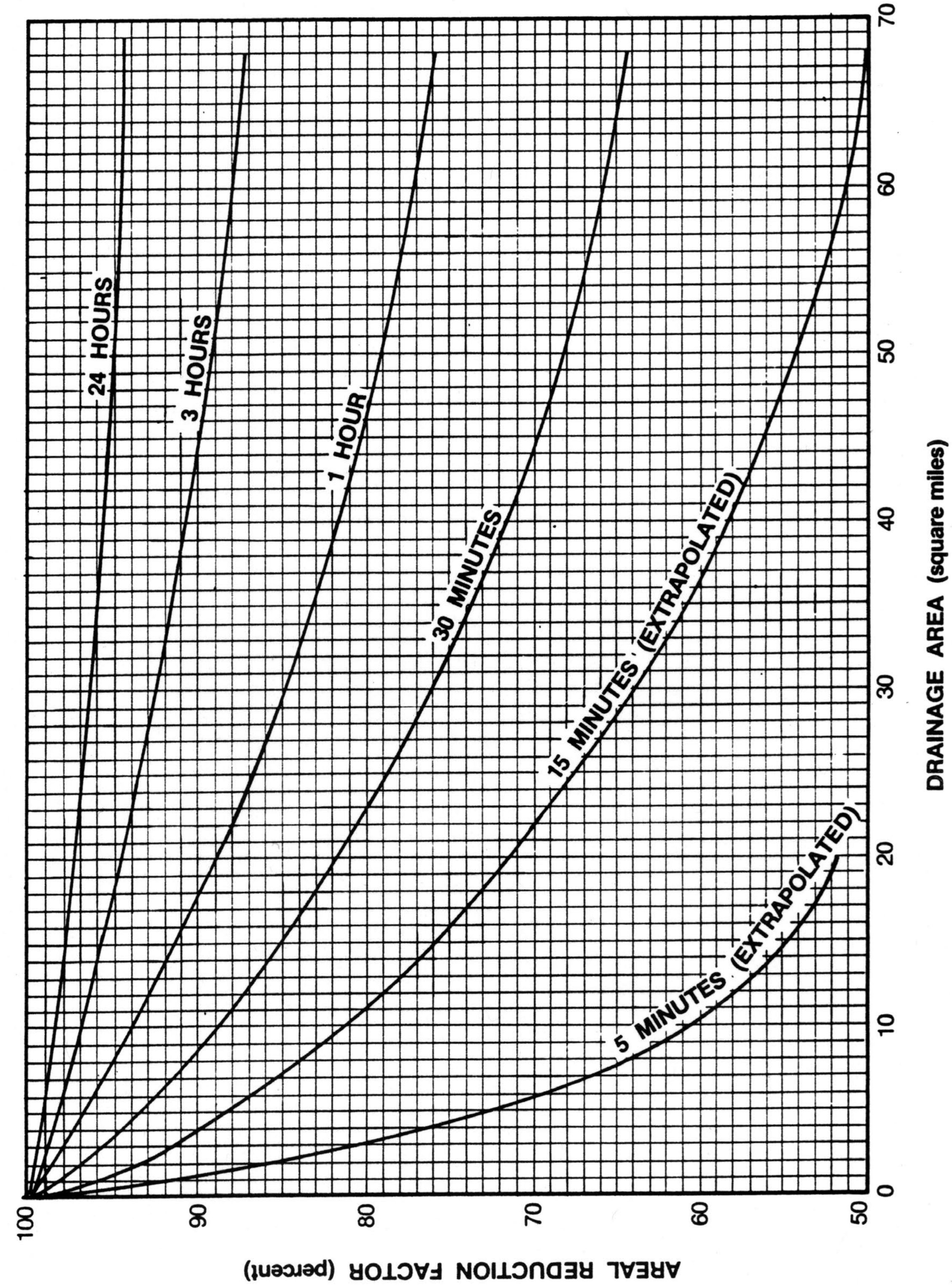


Reference: Frederick et al. (1977) and Data from Nashville Airport, 1949 to 1985, Climatic Data Center, NOAA.

RAINFALL VOLUME (inches)										
	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
2-Year	0.40	0.64	0.80	1.11	1.51	1.75	1.96	2.31	2.73	3.39
5-Year	0.52	0.83	1.05	1.50	1.97	2.39	2.71	3.30	3.83	4.50
10-Year	0.58	0.95	1.21	1.73	2.27	2.82	3.21	3.96	4.57	5.23
25-Year	0.67	1.10	1.38	2.00	2.66	3.36	3.84	4.79	5.49	6.16
50-Year	0.74	1.22	1.52	2.23	2.94	3.76	4.30	5.41	6.18	6.85
100-Year	0.81	1.35	1.72	2.50	3.21	4.16	4.77	6.02	6.86	7.53

RAINFALL INTENSITY (inches/hour)										
	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
2-Year	4.75	3.81	3.20	2.22	1.51	0.88	0.65	0.39	0.23	0.14
5-Year	6.25	4.99	4.20	3.00	1.97	1.20	0.90	0.55	0.32	0.19
10-Year	6.97	5.71	4.84	3.46	2.27	1.41	1.07	0.66	0.38	0.22
25-Year	8.00	6.60	5.50	4.00	2.66	1.68	1.28	0.80	0.46	0.26
50-Year	8.90	7.35	6.08	4.45	2.94	1.88	1.43	0.90	0.52	0.29
100-Year	9.72	8.08	6.88	4.99	3.21	2.08	1.59	1.00	0.57	0.31

Figure 2-1
Intensity-Duration-Frequency Curves and Depth-Duration Data



Reference: Hershfield (1961).

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

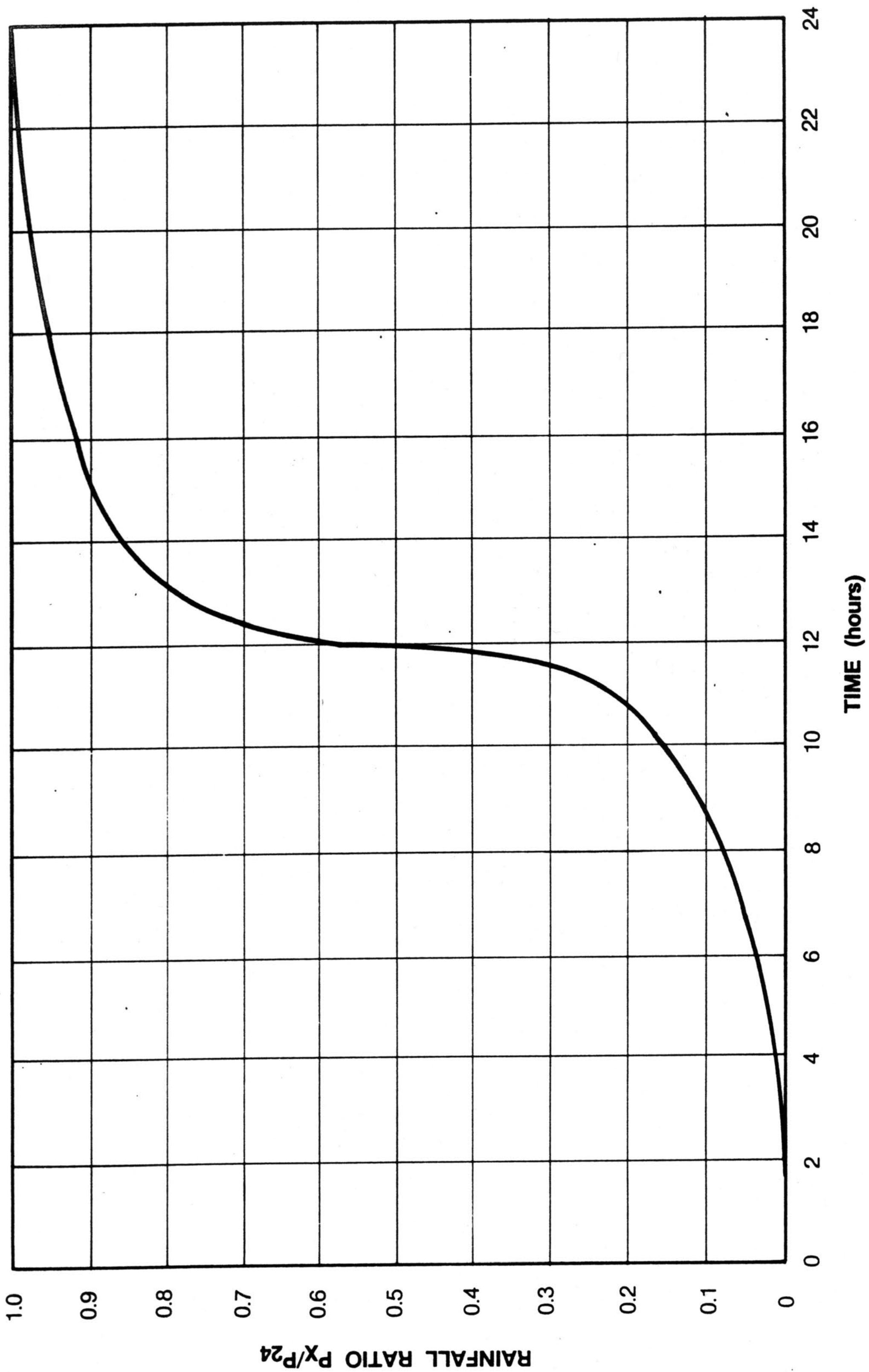


Figure 2-3
24-Hour Rainfall Hyetograph for Metro Nashville Area

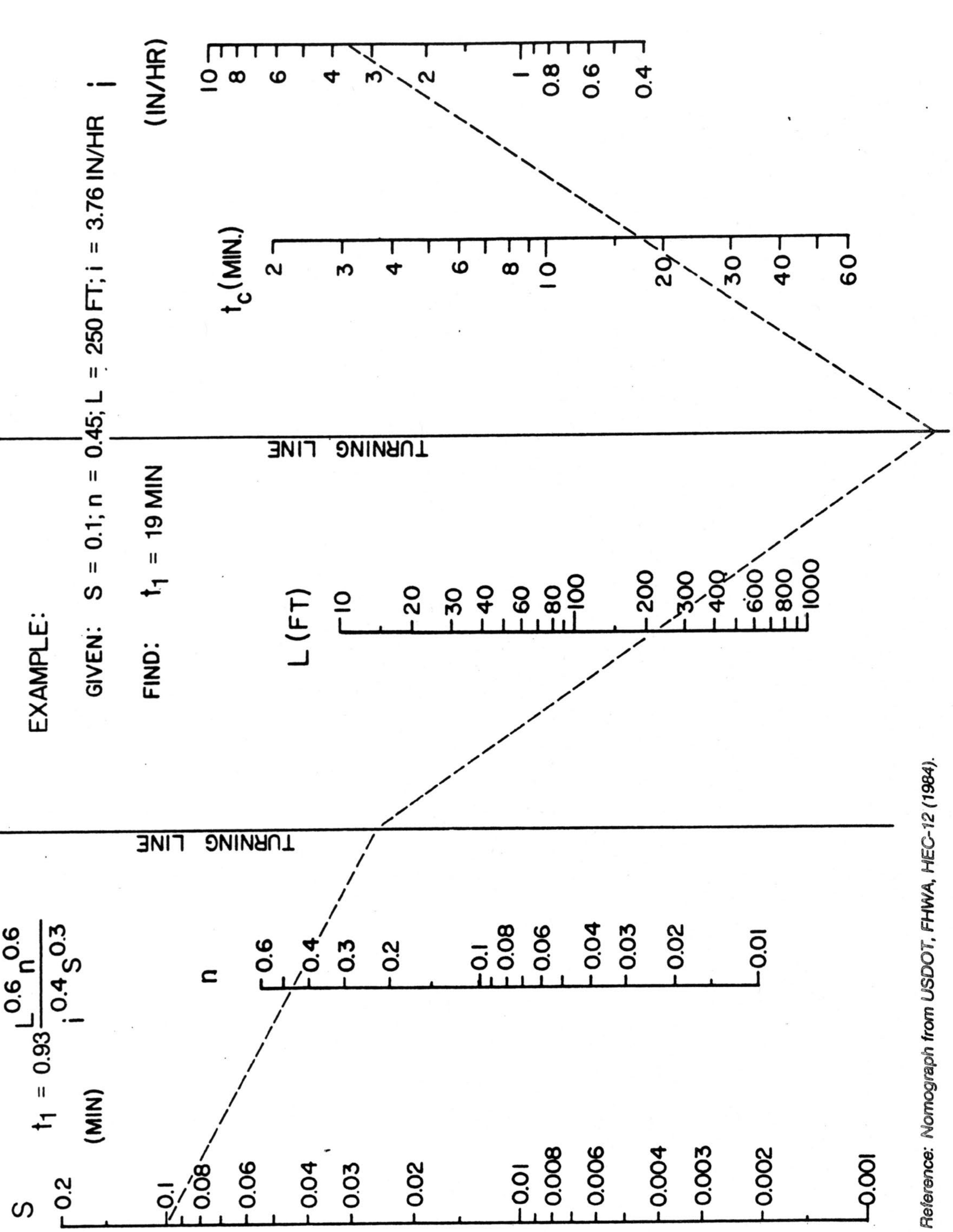
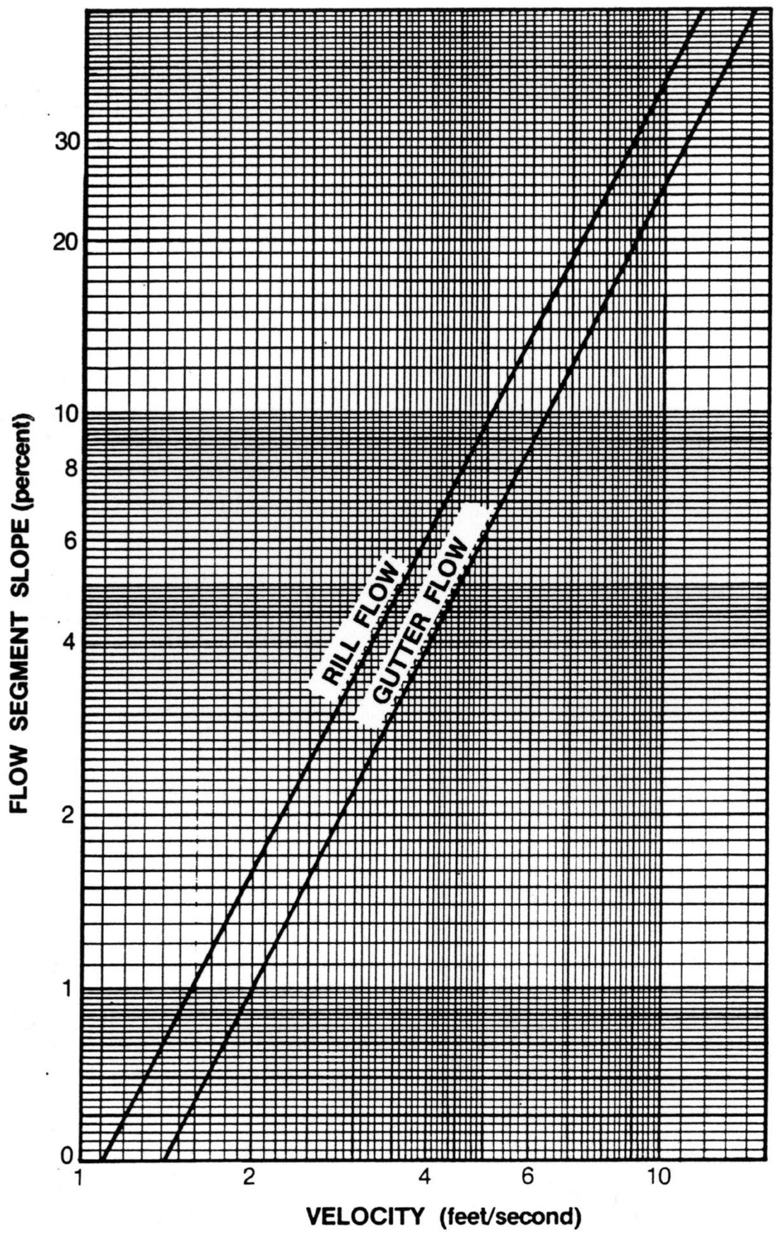


Figure 2-4
Kinematic Wave Nomograph for Estimating Overland Flow Travel Time



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

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

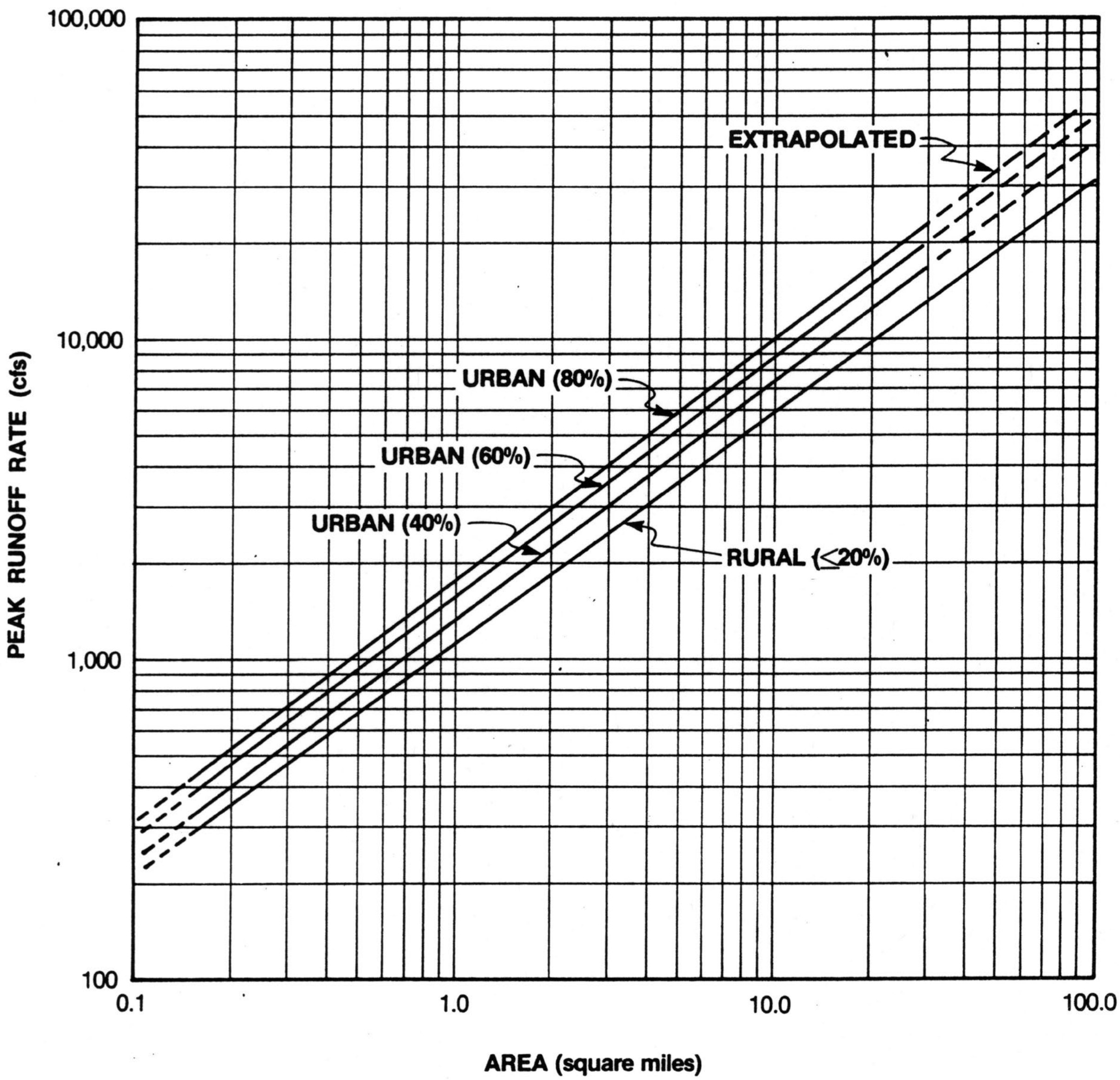
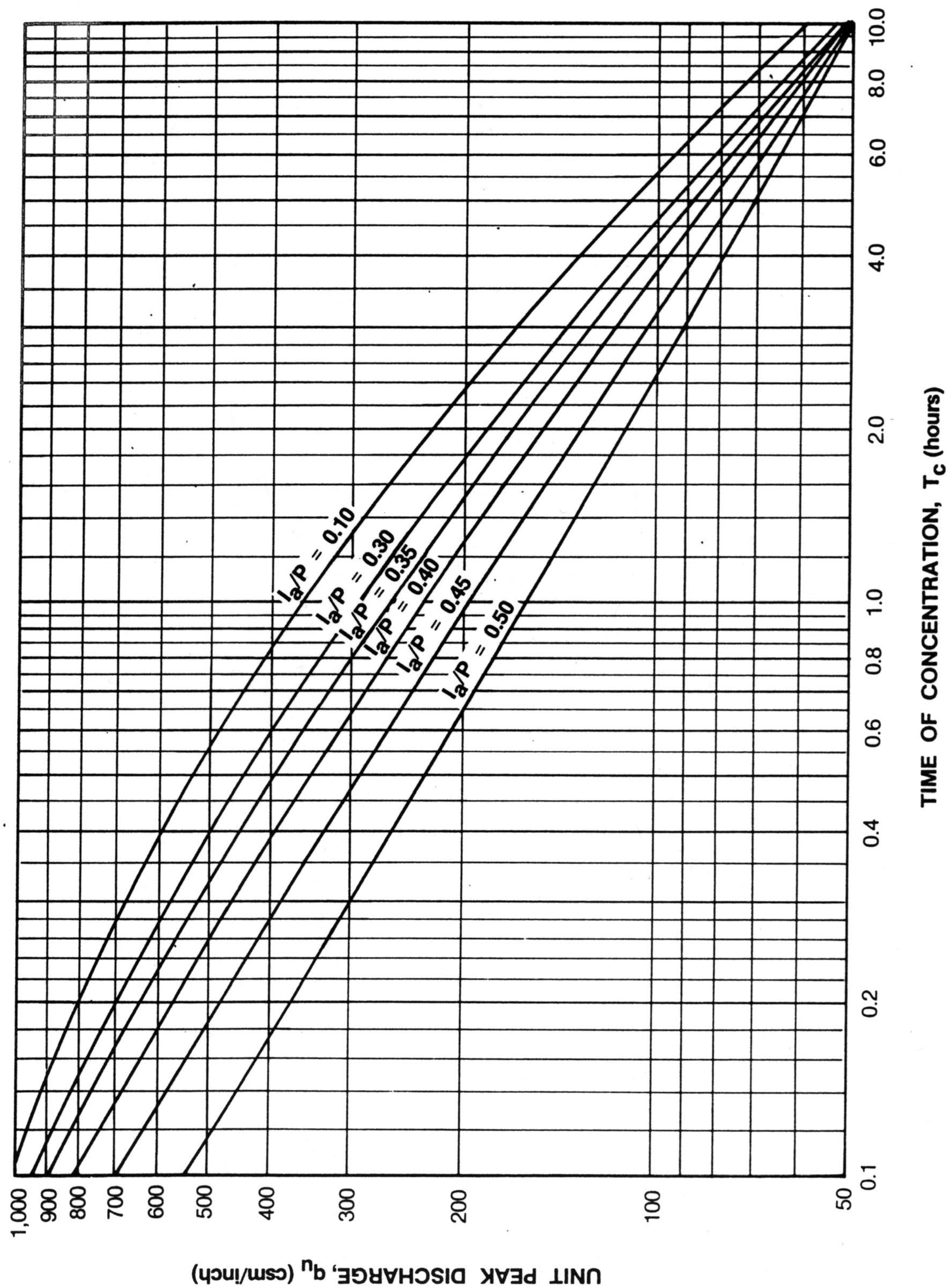
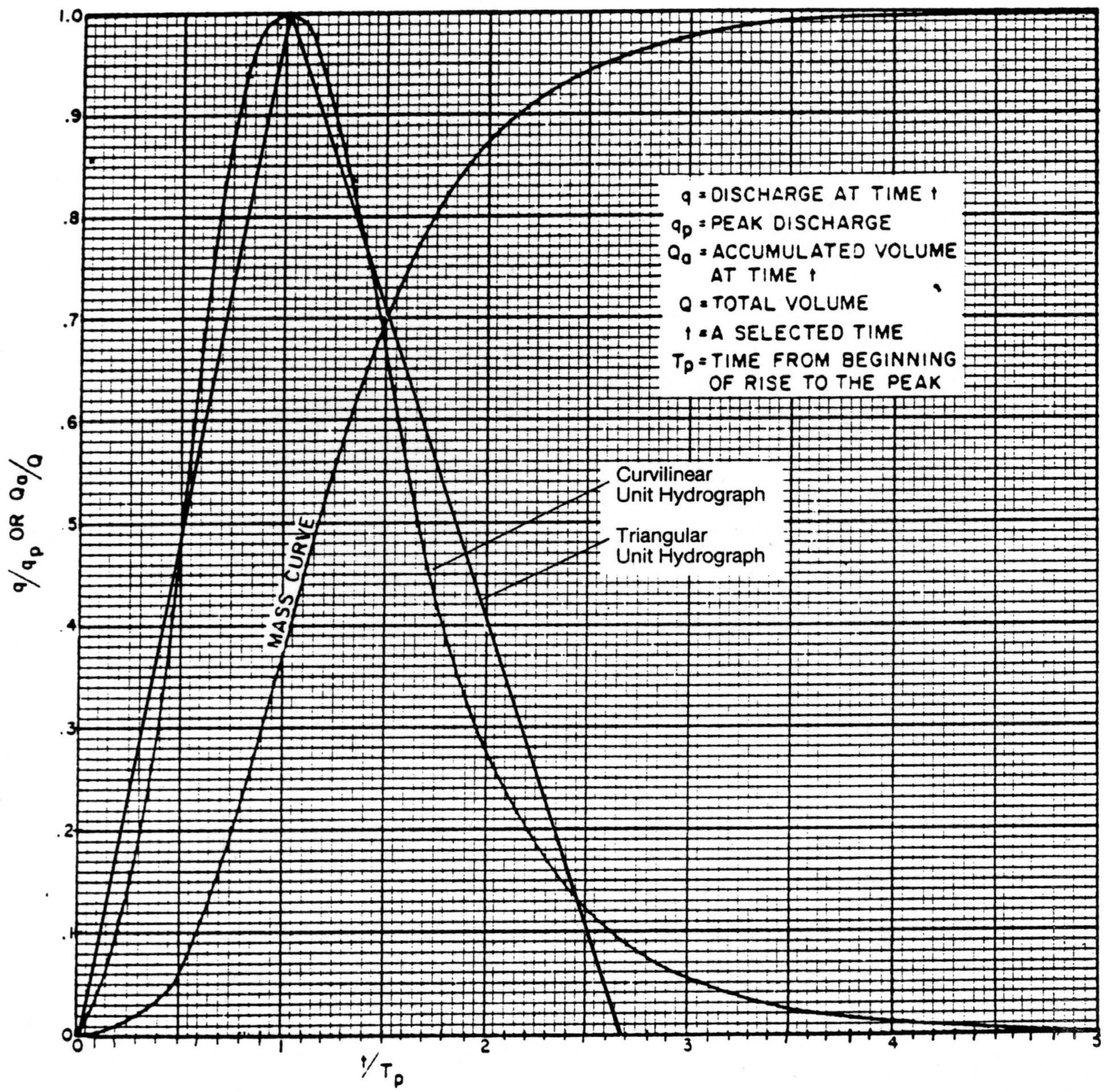


Figure 2-6
100-Year Return Period
Peak Runoff Rate Regression Equations



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

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

CHAPTER 3
OPEN CHANNEL HYDRAULICS

Chapter 3
OPEN CHANNEL HYDRAULICS

Synopsis

A consideration of open channel hydraulics is an integral part of projects in which artificial channels and improvements to natural channels are a primary concern. This chapter emphasizes procedures for performing uniform flow calculations that aid in the selection or evaluation of appropriate channel linings, depths, and grades. Allowable velocities are provided, along with procedures for evaluating channel capacity using Manning's Equation. For most artificial channels, the most desirable lining is vegetation with temporary measures to provide short-term erosion resistance during construction. If vegetation is infeasible, a flexible lining of riprap reinforced with a soil bioengineering technique is generally better than rigid paving for preventing erosion.

3.1 Linings

The three main classifications of open channel linings are vegetative, flexible, and rigid. Vegetative linings include grass with mulch, sod, and lapped sod. Rock riprap is a flexible lining. There are a number of soil bioengineering techniques that are a blend of vegetative and flexible lining techniques. For this discussion they will be presented with the flexible linings. Rigid linings are generally concrete.

3.1.1 Vegetation

Vegetation is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, and controls the movement of soil particles along the channel bottom. Conditions under which vegetation may not be acceptable, however, include but are not limited to:

1. Flow conditions in excess of the maximum shear stress for bare soils (see Section 3.2.4)

2. Standing or continuous flowing water

3. Lack of the regular maintenance necessary to prevent domination by taller vegetation

4. Lack of nutrients and inadequate topsoil

5. Excessive shade (under a small bridge or large culvert)

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of a healthy growth of grass. Soil testing should be performed and the results evaluated by an agronomist to determine soil treatment requirements for pH, nitrogen, phosphorus, potassium, and other factors. In many cases, temporary erosion control measures are required to provide time for the seeding to establish a viable vegetative lining.

Sodding should be staggered, to avoid seams in the direction of flow. Lapped or shingle sod should be staggered and overlapped by approximately 25 percent. Staked sod is usually only appropriate for use on steeper slopes to prevent sliding. Additional information on vegitation is presented in Volume 4 Best Management Practices fact sheet PESC-01: Permanent Grasses, Vines and other Vegetation. This fact sheet should be consulted for additional information on applying and design criteria for this practice.

3.1.2 Flexible

Rock riprap including rubble is the most common (while not the most preferred) type of flexible lining. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance.

They typically require use of filter fabric and allow the infiltration and exfiltration of water. The use of flexible lining may be restricted where right-of-way is limited, since the higher roughness values create larger cross sections. Additional information on riprap is presented in Volume 4 Best Management Practices fact sheet TCP-20: Riprap.

A “soft” blend of vegetation and flexible lining are applied in geotextiles reinforcement and “bioengineering” techniques. This is presented in more detail in Volume 4 – Best Management Practices fact sheets PESC-01: Permanent Grasses, Vines and other Vegetation, TCP-10 and PESC-02: Geotextiles, TCP-19: Bank Stabilization, and PESC-04: Soil Bioengineering and Bank Stabilization. These fact sheets should be consulted for additional information on applying and design criteria for these practices.

3.1.3 Rigid

Rigid linings are generally constructed of concrete and used where smoothness offers a higher capacity for a given cross-sectional area. They should only be applied when vegetative, flexible and “softer” techniques like soil bioengineering techniques have been thoroughly considered. Higher velocities, however, create the potential for scour at channel lining transitions. A rigid lining can be destroyed by flow undercutting the lining, channel headcutting, or the buildup of hydrostatic pressure behind the rigid surfaces. When properly designed, rigid linings may be appropriate where the channel width is restricted. Filter fabric may be required to prevent soil loss through pavement cracks. Under continuous base conditions when a vegetative lining alone would be inappropriate, a small concrete pilot channel could be used to handle the continuous low flows. Vegetation could then be maintained for handling larger flows. Additional information on channel linings are presented in Volume 4 Best Management Practices fact sheet TCP-21 and PESC-08: Channel Linings. This fact sheet should be consulted for additional information on applying and design criteria for this practice.

3.2 Design Criteria

3.2.1 General

In general, the following criteria shall be considered for open channels:

1. Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1.

2. Low flow sections shall be considered in the design of channels with large cross sections.

3. Channels with design flows greater than 100 cfs will be considered to have large cross sections.

4. Channel side slopes shall be stable throughout the entire length and shall consider the channel material. A maximum of 3:1 is allowed for vegetation and 2:1 for riprap, unless otherwise justified by calculations.

5. Superelevation of the water surface at horizontal curves shall be accounted for by increased free-board (see Section 3.3.5).

6. Parabolic cross sections are preferred while an alternative trapezoidal channel may be accepted. Triangular shapes should be avoided.

3.2.2 Channel Transitions

The following criteria shall be considered at channel transitions:

1. Transition to channel sections shall be smooth and gradual.

2. A straight line connecting flow lines at the two ends of the transition shall not make an angle greater than 12.5 degrees with the axis of the main channel.

3. Transition section length shall be considerably greater than the transition width.

4. Energy losses in transitions shall be accounted for as part of water surface profile calculations (see Section 3.4).

3.2.3 Return Period

Minor drainage systems shall be sized to handle a 10-year design storm; major systems shall be sized to handle a 100-year design storm. However, if the 10-year design flow exceeds 100 cfs, then the system shall be capable of passing the 100-year design flow within the drainage easement. Definitions of minor and major systems are provided in Volume 1 along with additional details on design policy.

Sediment transport requirements must be considered for conditions of flow below the design frequency. A low flow channel component within a larger channel can reduce maintenance by improving sediment transport in the channel.

Temporary open channel facilities used during construction should be designed to handle flood flows commensurate with risks. The recommended minimum frequency for temporary facilities and temporary lining of permanent facilities is 20 percent of the standard frequency for permanent facilities.

3.2.4 Velocity Limitations

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in Table 3-1. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 3-2. Vegetative lining calculations are presented in Section 3.3.5 and Volume 4: PESC-02, and rip-rap procedures are presented in Section 3.3.6 and Volume 4: TCP-20.

3.2.5 Manning's n Values

<u>General</u>

The following general factors should be considered when selecting the value of Manning's n:

1. As a general rule, retardance is increased when conditions tend to induce turbulence and reduced when they tend to minimize turbulence.

2. The physical roughness of the bottom and sides of the channel should be taken into account. Fine particle soils on smooth, uniform surfaces result in relatively low values of n. Coarse materials, such as gravel or boulders, and pronounced surface irregularity cause higher values of n.

3. The value of n will be affected by the height, density, and type of vegetation. Consideration should be given to density and distribution of the vegetation along the reach and the wetted perimeter, the degree to which the vegetation occupies or blocks the cross section of flow at different depths, and the degree to which the vegetation may be bent or "shingled" by flows of different depths. The n value will increase in the spring and summer, as vegetation grows and foliage develops, and diminish in the fall, as the dormant season approaches.

4. Channel shape variations, such as abrupt changes in channel cross sections or alternating small and large cross sections, will require somewhat larger n values than normal. These variations in channel cross section become particularly important if they cause the flow to meander from side to side.

5. A significant increase in the value of n is possible if severe meandering occurs in the alignment of a channel. Meandering becomes particularly important when frequent changes in the direction of curvature occur with relatively small radii of curvature.

6. Active channel erosion or sedimentation will tend to increase the value of n, since these processes may cause variations in the shape of a channel. The potential for future erosion or sedimentation in the channel should also be considered.

7. Obstructions such as log jams or deposits of debris will increase the value of n. The level of this increase will depend on the number, type, and size of obstructions.

8. To be conservative, it is better to use a higher resistance for capacity calculations and a lower resistance for stability calculations.

9. Proper assessment of natural channel n values requires field observations and experience. Special attention is required in the field to identify flood plain vegetation and evaluate possible variations in roughness with depth of flow.

All of these factors should be studied and evaluated with respect to type of channel, degree of maintenance, seasonal requirements, and other considerations as a basis for making a determination of an appropriate design n value. The probable condition of the channel when the design event is anticipated should be considered. Values representative of a freshly constructed channel are rarely appropriate as a basis for design capacity calculations.

<u>Artificial Channels</u>

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and riprap linings are given in Table 3-3. Recommended values for vegetative linings should be determined using Figure 3-1, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 3-4). Figure 3-1 is used iteratively as described in Section 3.3.5.

Natural Channels

For natural stream channels, Manning's n value may be estimated using Cowan's Equation (Cowan, 1956), presented below:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \qquad (3\text{-}1)$$

where:

n = Manning's roughness coefficient for a natural or excavated channel

n_0 = Coefficient associated with channel lining material

n_1 = Coefficient associated with the degree of channel irregularity

n_2 = Coefficient associated with variations of the channel cross section

n_3 = Coefficient associated with the relative effect of channel obstructions

n_4 = Coefficient associated with channel vegetation

m_5 = Coefficient associated with the degree of channel meandering

Coefficients for Equation 3-1 can be determined using information in Table 3-5. Additional information is presented in FHWA-TS-84-204 (USDOT, FHWA, 1984), including procedures for determining Manning's n values for flood plains.

3.2.6 Channel Stability Considerations

A stream channel is a dynamic feature that assumes a particular shape or size in response to the flow forces acting on it and the constraints imposed by geology, vegetation, and other factors, and tends to fluctuate around this equilibrium shape. Channel response to changes may be slow or rapid and may result from natural occurrences (e.g., natural erosion, a fallen tree) or human activity (e.g., a bridge abutment). When a channel is modified locally, the change frequently causes a response and modification of the channel characteristics both up and downstream. Additional detail about channel response to urbanization and river geomorphology is provided in FHWA-HI-90-016 (USDOT, FHWA, 1990) including discussion of channel sinuosity.

Quantification of the effects of human activity on a channel can be difficult. However, a qualitative assessment of possible impacts and appropriate preventive measures is required for all channel modifications and can be aided through use of the following proportionality:

$$QS \; \% \; Q_S \, d_{50} \qquad (3\text{-}2)$$

where:

Q = Typical or dominant discharge, in cfs

S = Energy slope, in feet/foot

Q_s = Sediment transport, in pounds/second

d_{50}= Sediment mean particle diameter, in mm

The two sides of this proportionality will tend to adjust to maintain equilibrium. For example, if a bend in a stream is straightened or a rough channel is paved, the energy slope is steepened. If S increases, either Q_s or d_{50} or both must also increase. The result is generally advancing scour upstream, higher velocities through the reach with associated channel deterioration, scour just below the reach, and eventual deposition downstream. If the channel is paved, excess sediment transport capacity is transferred just downstream, causing greater scour. Volume 3 contains further discussion on this subject.

In general, when the impacts of proposed modifications on channel stability are being assessed, the following considerations and actions apply:

1. A study should be made of the stream to be modified and should include historical information, evidence of other instability (i.e., bank caving, channel movement), results of other developments, and aerial photos showing alignment changes, either natural or manmade. The composition and erodibility of bed and bank material should be determined.

2. Backwater calculations (see Section 3.4) should be performed through the reach for a range of flows, including bankfull flow. Existing worst-case velocities and slopes should be calculated and related to the existing channel configuration to determine maximum velocities and shear stress values for actual conditions.

3. A channel modification scheme that minimizes interference with the channel is preferred. Equation 3-2 can be used to evaluate the likely channel response to proposed changes.

4. Channel modifications should be sized to match existing sizes and shape. A narrow channel will deteriorate and a wide channel may collect silt. Floodways or high flow

channels should be used to carry extreme events rather than over-sizing a channel. Backwater should be recalculated through the modified reach.

5. Protection should be provided where needed, from the downstream through the upstream extent of modification effects (i.e., effects of modification are often felt beyond the project limits). For flow with significant overbank components, a central section velocity must be used instead of the mean flow velocity. Protection should be sized for the design event and design smooth transitions. If velocities are too high, grade control structures or check dams should be considered.

3.3 Uniform Flow Calculations

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (3\text{-}3)$$

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad (3\text{-}4)$$

$$S = \left(\frac{Qn}{1.49 A R^{2/3}} \right)^2 \qquad (3\text{-}5)$$

where:

v = Average channel velocity, in feet/second

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 3.2.5)

A = Cross-sectional area, in square feet

R = Hydraulic radius A/P, in feet

P = Wetted perimeter, in feet

S = Slope of the energy grade line, in feet/foot

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line and channel bottom can be assumed to be the same.

3.3.1 Geometric Relationships

Mathematical expressions for calculating the area, wetted perimeter, hydraulic radius, and channel top width for selected open channel cross sections are presented in Figure 3-2. These cross sections include trapezoidal, rectangular, triangular, parabolic, and circular shapes. Geometric properties of trapezoidal channels also can be evaluated using the chart presented in Figure 3-3.

Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter (USGS, 1976a,b).

3.3.2 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from Equation 3-4. The slope can be calculated using Equation 3-5 when the discharge, roughness coefficient, area, and hydraulic radius are known. Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 3-4 and 3-5. Figure 3-4 provides a general solution for the velocity form of Manning's Equation, while Figure 3-5 provides a solution of Manning's Equation for trapezoidal channels.

<u>General</u>

The following steps are used for the general solution nomograph in Figure 3-4:

1. Determine open channel data, including slope in feet/foot, hydraulic radius in feet, and Manning's n value.

2. Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

3. Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

4. Extend the line from Step 3 to the velocity scale to obtain the velocity in feet/second.

Trapezoidal

The trapezoidal channel nomograph solution to Manning's Equation in Figure 3-5 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

1. Determine input data, including slope in feet/foot, Manning's n value, bottom width in feet, and side slope in feet/foot.

2. a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.

   a. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.

   b. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.

   c. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.

3. a. Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.

   b. Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.

   c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Qn scale.

   d. Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.

3.3.3 Trial and Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process. Manning's Equation can be arranged as:

$$AR^{2/3} = \frac{Qn}{1.49S^{1/2}} \qquad (3\text{-}6)$$

where:

A = Cross-sectional area, in square feet

R = Hydraulic radius, in feet

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 3.2.5)

S = Slope of the energy grade line, in feet/foot

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of Equation 3-6 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 3-6 for trapezoidal channels, which is described below.

1. Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

2. Calculate the trapezoidal conveyance factor using the equation:

$$K_T = \frac{Qn}{b^{8/3} S^{1/2}} \qquad (3\text{-}7)$$

where:

K_T = Trapezoidal open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 3.2.5)

b = Bottom width, in feet

S = Slope of the energy grade line, in feet/foot

3. Enter the x-axis of Figure 3-6 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

4. From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

5. Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

3.3.4 Critical Flow Calculations

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad (3\text{-}8)$$

where:

Q = Discharge rate for design conditions, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

A = Cross-sectional area, in square feet

T = Top width of water surface, in feet

A trial and error procedure is needed to solve Equation 3-8. Semi-empirical equations (as presented in Table 3-6) or section factors (as presented in Figure 3-2) can be used to simplify trial and error critical depth calculations. The following equation from Chow (1959) is used to determine critical depth with the critical flow section factor, Z:

$$Z = Q / \sqrt{g} \qquad (3\text{-}9)$$

where:

Z = Critical flow section factor

Q = Discharge rate for design conditions, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

The following guidelines are presented for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within about 10 percent of critical depth is unstable and should be avoided in design, if possible.

2. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.

3. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.

4. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

5. If an unstable critical depth cannot be avoided in design, the least favorable type of flow should be assumed for the design.

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = v/ (gA/T)^{0.5} \qquad (3\text{-}10)$$

where:

Fr = Froude number, dimensionless

v = Velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

A = Cross-sectional area of flow, in square feet

T = Top width of flow, in feet

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

3.3.5 Vegetative Design

A two-part procedure, adapted from Chow (1959) and presented below, is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 3-4. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 3-4. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

Temple et al. (1987) present an alternative procedure for designing grass-lined channels that is acceptable but not duplicated in the manual.

If the channel slope exceeds 10 percent, or a combination of channel linings will be used, additional procedures not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

Design Stability

1. Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

2. Use Table 3-2 to assign a maximum velocity, v_m, based on vegetation type and slope range.

3. Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 3-1. Use retardance Class D for permanent vegetation and E for temporary construction.

4. Calculate the hydraulic radius using the equation:

$$R = \frac{(vR)}{V_m} \tag{3-11}$$

where:

R = Hydraulic radius of flow, in feet

*v*R = Value obtained from Figure 3-1 in Step 2

V_m = Maximum velocity from Step 2

5. Use the following form of Manning's Equation to calculate the value of vR:

$$vR = \frac{1.49 R^{5/3} S^{1/2}}{n} \tag{3-12}$$

where:

vR = Calculated value of vR product

R = Hydraulic radius value from Step 4, in feet

S = Channel bottom slope, in feet/foot

n = Manning's n value assumed in Step 3

6. Compare the vR product value obtained in Step 5 to the value obtained from Figure 3-1 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

7. For trapezoidal channels, find the flow depth using Figures 3-5 or 3-6, as described in Section 3.3.3. The depth of flow for other channel shapes can be evaluated using the trial and error procedure in Section 3.3.3.

8. If bends are considered, calculate the length of downstream protection, L_p, for the bend using Figure 3-7. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

Design Capacity

1. Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 3-2 for equations).

2. Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

3. Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

4. Use Figure 3-1 to find a Manning's n value for retardance Class C based on the vR value from Step 3.

5. Use Manning's Equation (Equation 3-3) or Figure 3-4 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

6. Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

7. Add an appropriate freeboard to the final depth from Step 6. Generally, 20 percent is adequate.

8. If bends are considered, calculate superelevation of the water surface profile at the bend using the equation:

$$\Delta d = \frac{v^2 T}{g R_c} \qquad (3\text{-}13)$$

where:

Δd = Superelevation of the water surface profile due to the bend, in feet

v = Average velocity from Step 6, in feet/second

T = Top width of flow, in feet

g = Acceleration due to gravity, 32.2 feet/second^2

R_c = Mean radius of the bend, in feet

Add freeboard consistent with the calculated Δd.

3.3.6 Riprap Design

Riprap should only be utilized when "green", "soft", geotextiles, and soil bioengineering techniques have been explored and thoroughly considered. Riprap may be used provided calculations are presented to MWS that illustrate that soil bioengineering or other techniques are not cost effective for the site or are not feasible.

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

1. Minimum riprap thickness equal to d_{100}.
2. The value of d_{85}/d_{15} less than 4.6.
3. Froude number less than 1.2.
4. Side slopes up to 2:1.
5. A safety factor of 1.2.
6. Maximum velocity less than 18 feet per second.

If significant turbulence is caused by boundary irregularities, such as installations near obstructions or structures, this procedure is not applicable.

1. Determine the average velocity in the main channel for the design condition. Use the higher value of velocity calculated both with and without riprap in place (this may require

iteration using procedures in Section 3.3.3). Manning's n values for riprap can be calculated from the equation:

$$n = 0.0395\ (d_{50})^{1/6} \qquad (3\text{-}14)$$

where:

n = Manning's roughness coefficient for stone riprap

d_{50} = Diameter of stone for which 50 percent, by weight, of the gradation is finer, in feet

2. If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 3-8 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.

3. If the specific weight of the stone varies from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 3-9.

4. Determine the required minimum d_{30} value from Figure 3-10, which is based on the equation:

$$d_{30}/D = 0.193\ Fr^{2.5} \qquad (3\text{-}15)$$

where:

d_{30} = Diameter of stone for which 30 percent, by weight, of the gradation is finer, in feet

D = Depth of flow above stone, in feet

Fr = Froude number (see Equation 3-10), dimensionless

v = Mean velocity above the stone, in feet/second

g = Acceleration of gravity, 32.2 feet/second^2

5. Determine available riprap gradations. A well graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$W = 0.5236\ \gamma_s\ d^3 \qquad (3\text{-}16)$$

where:

W = Stone weight, in pounds

d = Selected stone diameter, in feet

γ_s = Specific weight of stone, in pounds/cubic foot

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50 percent for underwater placement.

6. If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 3-11 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

7. Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

3.3.7 Approximate Flood Limits

For small streams and tributaries not included in the basin studies, analysis may be required to identify the 100-year flood elevation and to evaluate flood plain encroachment as required by Volume 1. For such cases, when the design engineer can demonstrate that a complete backwater analysis is unwarranted, approximate methods may be used.

A generally accepted method for approximating the 100-year flood elevation is outlined as follows:

1. Divide the stream or tributary into reaches that may be approximated using average slopes, cross sections, and roughness coefficients for each reach.

2. Estimate the 100-year peak discharge for each reach using an appropriate hydrologic method from Chapter 2.

3. Compute normal depth for uniform flow in each reach using Manning's Equation for the reach characteristics from Step 1 and peak discharge from Step 2.

4. Use the normal depths computed in Step 3 to approximate the 100-year flood elevation in each reach. The 100-year flood elevation is then used to delineate the flood plain.

This approximate method is based on several assumptions, including, but not limited to, the following:

1. A channel reach is accurately approximated by average characteristics throughout its length.

2. The cross-sectional geometry, including area, wetted perimeter, and hydraulic radius, of a reach may be approximated using typical geometric properties that can be used in Manning's Equation to solve for normal depth.

3. Uniform flow can be established and backwater effects are negligible between reaches.

4. Expansion and contraction effects are negligible.

As indicated, the approximate method is based on a number of restrictive assumptions that may limit the accuracy of the approximation and applicability of the method. The engineer is responsible for appropriate application of this method.

After the 100-year flood elevation and flood plain are established, floodway setback limits may be approximated by requiring conveyance of the encroached section, including any allowable flood elevation increases specified in Volume 1, to equal the conveyance of the non-encroached section. From Manning's Equation, the conveyance is given as follows:

$$K = \frac{1.49}{n} AR^{2/3} \tag{3-17}$$

$$Q = CS^{1/2} \tag{3-18}$$

where:

K = Channel conveyance

A = Cross-sectional area, in square feet

R = Hydraulic radius A/P, in feet

P = Wetted perimeter, in feet

Q = Discharge, in cfs

S = Slope of the energy grade line, in feet/foot

The following procedure may be used to approximate setback limits for a stream:

1. Divide the stream cross section into segments for which the geometric properties may be easily solved and estimate an n value for each segment.

2. Compute the area, hydraulic radius, and conveyance (Equation 3-17) of each segment for both the encroached and non-encroached segment. Include the allowable flood elevation increase from Volume 1 in the computations for the encroached segments.

3. Sum the conveyance for each cross-sectional segment to obtain the total conveyance for both the encroached and non-encroached conditions.

4. Set the total conveyance of the encroached cross section equal to the total conveyance of the non-encroached section and solve for the allowable encroachment by trial and error.

This method for approximating the allowable encroachment is based on the assumptions that the 100-year flood elevation has been established or can be approximated and that the energy grade line of the encroached and non-encroached sections remains unchanged. The accuracy of results obtained using this method may be highly subject to the accuracy of the flood elevation used. In addition, since the method assumes no change in the energy grade line, the method should not be used near bridges or similar contraction-expansion areas.

For typical natural channel cross sections, the procedure may result in an equality that is very difficult to solve for the allowable encroachment dimensions. Morris (1984) provides a series of dimensionless graphs that are solved for the allowable encroachment as a percentage of the non-encroached overbank width. These graphs are based on an allowable flood elevation increase of 1 foot and assume a symmetrical cross section with triangular overbanks and equal encroachment on both overbanks. The limitations listed above for the general procedure also apply.

Because of the simplifying assumptions required, this approximate method will have limited applicability. Generally, only very small streams will satisfy the assumptions and the engineer should use extreme caution to avoid misapplication.

3.3.8 Example Problems

Example 3-1. Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 foot, an n value of 0.020, and slope of 0.003 foot/foot.

Solve using Figure 3-4:

1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.

2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 feet and read the velocity of 2.9 feet/second from the velocity scale.

Example 3-2. Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 foot/foot. Find the channel dimensions required to comply with design stability criteria (retardance Class D) for a grass mixture.

1. From Table 3-2, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5 percent is 4 feet/second.

2. Assume an n value of 0.035 and find the value of vR from Figure 3-1.

   vR = 5.4

3. Use Equation 3-11 to calculate the value of R:

$$R = \frac{5.4}{4} = 1.35\ feet$$

4. Use Equation 3-12 to calculate the value of vR:

5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a

$$vR = \frac{1.49(1.35)^{5/3}(0.015)^{1/2}}{(0.035)} = 8.60$$

   higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed n Value	vR (Figure 3-1)	R (Equation 3-11)	vR (Equation 3-12)
0.035	5.4	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

6. Use Figure 3-5 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

   Qn = (50) (0.040) = 2.0

   S = 0.015

   For b = 10 feet, d = (10) (0.098) = 0.98 feet
   b = 8 feet, d = (8) (0.14) = 1.12 feet

   Select: b = 10 feet, such that R is approximately 0.80 feet
   z = 3
   d = 1 foot
   v = 3.9 feet/second (Equation 3-3)
   Fr = 0.76 (Equation 3-10)
   Flow is subcritical

   Design capacity calculations for this channel are presented in Example 3-3.

Example 3-3. Grassed Channel Design Capacity

Use a 10-foot bottom width for the trapezoidal channel sized in Example 3-2 and find the depth of flow for retardance Class C.

1. Assume a depth of 1.0 foot and calculate the following (see Figure 3-2):

   A = (b + zd) d

   A = [10+ (3) (1)] (1)

   A = 13.0 square feet

$$R = \frac{[b + zd]d}{b + 2d\sqrt{1 + z^2}}$$

R = 0.796 feet

$$R = \frac{[10 + (3)(1)](1)}{10 + (2)(1)\sqrt{1 + 3^2}}$$

2. Find the velocity.

v = 50/13.0

v = 3.85 feet/second

3. Find the value of vR.

vR = (3.85) (0.796) = 3.06

4. Using the vR product from Step 3, find Manning's n from Figure 3-1 for retardance Class C.

n = 0.047

5. Use Figure 3-4 or Equation 3-3 to find the velocity for S = 0.015, R = 0.796, and n = 0.047.

v = 3.34 feet/second

6. Since 3.34 feet/second is less than 3.85 feet/second, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

Assumed Depth (ft)	Area $(ft)^2$	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 3-1)	Velocity (Eq. 3-3)
1.0	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.0475	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

7. Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 feet for freeboard to give a design depth of 1.3 feet. Design data for the trapezoidal channel are summarized as follows:

Vegetation lining = grass mixture, v_m = 4 feet/second

Q = 50 cfs

b = 10 feet, d = 1.3 feet, z = 3, S = 0.015

Top width = (10) + (2) (3) (1.3) = 17.8 feet

n (stability) = 0.040, d = 1.0 foot, v = 3.9 feet/second, Froude number = 0.76 (Equation 3-10)

n (capacity) = 0.048, d = 1.1 feet, v = 3.45 feet/second, Froude number = 0.64 (Equation 3-10)

Example 3-4. Riprap Design

A natural channel has an average bankfull channel velocity of 8 feet per second with a top width of 20 feet and a bend radius of 50 feet. The depth over the toe of the outer bank is 5 feet. Available stone weight is 170 pounds per cubic foot. Stone placement is on a side slope of 2:1 (horizontal: vertical).

1. Use 8 feet per second as the design velocity, because the reach is short and the bend is not protected.

2. Determine the bend correction coefficient for the ratio of R_b/T = 50/20 = 2.5. From Figure 3-8, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 feet/second.

3. Determine the correction coefficient for the specific weight of 170 pounds from Figure 3-9 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 feet/second.

4. Determine minimum d_{30} from Figure 3-10 or Equation 3-15 as about 10 inches.

5. An available gradation has a minimum d_{30} size of 12 inches and is acceptable. It has enough fines that a filter course will not be required.

6. (Optional) Another gradation is available with a d_{30} of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From Figure 3-11, this gradation would be acceptable if the blanket thickness were increased from the original d_{100} thickness by 35 percent (a ratio of 1.35 on the horizontal axis).

7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 3-7.

3.4 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used program, HEC-RAS, was developed by the U.S. Army Corps of Engineers (1998) and SWMM-Extran block was developed by the U.S. Environmental Protection Agency (Huber et al 1992, Roesner et al 1994) are recommended for floodwater profile computations. This program can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method, as presented by Chow (1959). For an irregular nonuniform channel, the standard step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS or SWMM Extran are recommended as an alternative for manual standard step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the stream and flood plain. In general, a cross section should be obtained at each location where there are significant changes in stream width, shape, or vegetal patterns. Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for flood plains, unless the channel is very regular. Additional details regarding survey requirements are provided in Chapter 11.

3.4.1 Direct Step Method

The direct step method is limited to prismatic channels. A form for recording the calculations described below is presented in **Table 3-7** (Chow, 1959).

1. Record the following parameters across the top of Table 3-7:

   Q = Design flow, in cfs

   n = Manning's n value (see Section 3.2.5)

   S_o = Channel bottom slope, in feet/foot

   α = Energy coefficient

y_c = Critical depth, in feet

y_n = Normal depth, in feet

2. Using the desired range of flow depths, y, recorded in column 1, compute the cross-sectional area, A, the hydraulic radius, R, and average velocity, v, and record results in columns 2, 3, and 4, respectively.

3. Compute the velocity head, $\alpha v^2/2g$, in feet, and record the result in column 5.

4. Compute specific energy, E, in feet, by summing the velocity head in column 5 and the depth of flow in column 1. Record the result in column 6.

5. Compute the change in specific energy, ΔE, between the current and previous flow depths and record the result in column 7 (not applicable for row 1).

6. Compute the friction slope using the equation:

$$S_f = \frac{n^2 v^2}{2.22 R^{4/3}} \qquad (3\text{-}19)$$

where:

S_f = Friction slope, in feet/foot

n = Manning's n value

v = Average velocity in feet/second

R = Hydraulic radius, in feet

Record the result in column 8.

7. Determine the average, of the friction slope between this depth and the previous depth (not applicable for row 1). Record the result in column 9.

8. Determine the difference between the bottom slope, S_o, and the average friction slope, $\overline{S}_f$, from column 9 (not applicable for row 1). Record the result in column 10.

9. Compute the length of channel between consecutive rows or depths of flow using the equation:

$$\Delta x = \frac{\Delta E}{S_o - S_f} = \frac{column\ 7}{column\ 10}$$

(3-20)

where:

Δx = Length of channel between consecutive depths of flow, in feet

ΔE = Change in specific energy, in feet

S_o = Bottom slope, in feet/foot

S_f = Friction slope, in feet/foot

Record the result in column 11.

10. Sum the distances from the starting point to give cumulative distances, x, for each depth in column 1 and record the result in column 12.

3.4.2 Standard Step Method

The standard step method is a trial and error procedure applicable to both natural and prismatic channels. The step computations are arranged in tabular form, as shown in Table 3-8 and described below (Chow, 1959):

1. Record the following parameters across the top of Table 3-8:

   Q = Design flow, in cfs

   n = Manning's n value (see Section 3.2.5)

   S_o = Channel bottom slope, in feet/foot

   α = Energy coefficient

   k_e = Eddy head loss coefficient, in feet

   y_c = Critical depth, in feet

   y_n = Normal depth, in feet

2. Record the location of measured channel cross sections and the trial water surface elevation, z, for each section in columns 1 and 2. The trial elevation will be verified or rejected based on computations of the step method.

3. Determine the depth of flow, y, based on trial elevation and channel section data. Record the result in column 3.

4. Using the depth from Step 3 and section data, compute the cross-sectional area. A, in feet, and hydraulic radius, R, in feet. Record the results in columns 4 and 5.

5. Divide the design discharge by the cross-sectional area from Step 4 to compute the average velocity, v, in feet/second. Record the result in column 6.

6. Compute the velocity head, $\alpha\, v^2/2g$, in feet, and record the result in column 7.

7. Compute the total head, H, in feet, by summing the water surface elevation, z, in column 2 and the velocity head in column 7. Record the result in column 8.

8. Compute the friction slope, S_f, using Equation 3-19 and record the result in column 9.

9. Determine the average friction slope, $\overline{S}_f$, between the sections in each step (not applicable for row 1). Record the result in column 10.

10. Determine the distance between sections, Δx, and record the result in column 11.

11. Multiply the average friction slope, $\overline{S}_f$ (column 10), by the reach length, Δx (column 11), to give the friction loss in the reach, h_f. Record the result in column 12.

12. Compute the eddy loss using the equation:

$$h_e = k_e \frac{v^2}{2g} \qquad (3\text{-}21)$$

where:

h_e = Eddy head loss, in feet

k_e = Eddy head loss coefficient, in feet (for prismatic and regular channels, $k_e = 0$; for gradually converging and diverging channels, $k_e = 0$ to 0.1 or 0.2; for abrupt expansions and contractions, $k_e = 0.5$)

v = Average velocity, in feet/second (column 6)

g = Acceleration due to gravity, 32.2 feet/second^2

12. Compute the elevation of the total head, H, by adding the values of h_f and h_e (columns 12 and 13) to the elevation at the lower end of the reach, which is found in column 14 of the previous reach or row. Record the result in column 14.

13. If the value of H computed above does not agree closely with that entered in column 8, a new trial value of the water surface elevation is used in column 2 and calculations are repeated until agreement is obtained. The computation may then proceed to the next step or section reported in column 1.

3.4.3 Example Problems

Example 3-5. Direct Step Method

Use the direct step method (Section 3.4.1) to compute a water surface profile for a trapezoidal channel using the following data:

$Q = 400$ cfs

$B = 20$ feet

$z = 2$

$S = 0.0016$ foot/foot

$n = 0.025$

$\alpha = 1.10$

A dam backs up water to a depth of 5 feet immediately behind the dam. The upstream end of the profile is assumed to have a depth 1 percent greater than normal depth.

Results of calculations, as obtained from Chow (1959), are reported in Table 3-9. Values in each column of the table are briefly explained below.

1. Depth of flow, in feet, arbitrarily assigned values ranging from 5 to 3.4 feet.

2. Water area in square feet, corresponding to the depth, y, in column 1.

3. Hydraulic radius, in feet, corresponding to y in column 1.

4. Mean velocity, in feet/second, obtained by dividing 400 cfs by the water area in column 2.

5. Velocity head, in feet, calculated using the mean velocity from column 4 and an α value of 1.1.

6. Specific energy, E, in feet, obtained by adding the velocity head in column 5 to the depth of flow in column 1.

7. Change of specific energy, ΔE, in feet, equal to the difference between the E value in column 6 and that of the previous step.

8. Friction slope, S_f, computed by Equation 3-19, with n = 0.025, v as given in column 4, and R as given in column 3.

9. Average friction slope between the steps, $\overline{S}_f$, equal to the arithmetic mean of the friction slope computed in column 8 and that of the previous step.

10. Difference between the bottom slope, S_o, 0.0016 and the average friction slope, $\overline{S}_f$, in column 9.

11. Length of the reach. Δx, in feet, between the consecutive steps, computed by Equation 3-20 or by dividing the value of ΔE in column 7 by the value of S_o - S_f in column 10.

12. Distance from the section under consideration to the dam site. This is equal to the cumulative sum of the values in column 11 computed for previous steps.

Example 3-6. Standard Step Method

Use the standard step method (see Section 3.4.2) to compute a water surface profile for the channel data and stations considered in Example 3-5. Assume the elevation at the dam site is 600 feet.

Results of the calculations, as obtained from Chow (1959), are reported in Table 3-10. Values in each column of the table are briefly explained below:

1. Section identified by station number such as "station 1 + 55." The locations of the stations are fixed at the distances determined in Example 3-5 to compare the procedure with that of the direct step method.

2. Water surface elevation, z, at the station. A trial value is first entered in this column; this will be verified or rejected on the basis of the computations made in the remaining columns of the table. For the first step, this elevation must be given or assumed. Since the elevation of the dam site is 600 feet and the height of the dam is 5 feet, the first entry is 605.00 feet. When the trial value in the second step has been verified, it becomes the basis for the verification of the trial value in the next step, and the process continues.

3. Depth of flow, y, in feet, corresponding to the water surface elevation in column 2. For instance, the depth of flow at station 1+55 is equal to the water surface elevation minus the elevation at the dam site minus the distance from the dam site times bed slope.

   605.048 - 600.00 - (155) (0.0016) = 4.80 feet

4. Water area, A, in square feet, corresponding to y in column 3.

5. Hydraulic radius, R, in feet, corresponding to y in column 3.

6. Mean velocity, v, equal to the given discharge 400 cfs divided by the water area in column 4.

7. Velocity head, in feet, corresponding to the velocity in column 6 and an % value of 1.1.

8. Total head, H, equal to the sum of z in column 2 and the velocity head in column 7.

9. Friction slope, S_f, computed by Equation 3-19, with n = 0.025, v from column 6, and R from column 5.

10. Average friction slope through the reach, S_f, between the sections in each step, approximately equal to the arithmetic mean of the friction slope just computed in column 9 and that of the previous step.

11. Length of the reach between the sections, Δx, equal to the difference in station numbers between the stations.

12. Friction loss in the reach, h_f, equal to the product of the values in columns 10 and 11.

13. Eddy loss in the reach, h_e equal to zero.

14. Elevation of the total head, H, in feet, computed by adding the values of h_f and h_e in columns 12 and 13 to the elevation at the lower end of the reach, which is found in column 14 of the previous reach. If the value obtained does not agree closely with that entered in column 8, a new trial value of the water surface elevation is assumed until agreement is obtained. The value that leads to agreement is the correct water surface elevation. The computation may then proceed to the next step.

3.5 Rapidly Varied Flow

Rapidly varied flow common to storm drainage systems occurs at flow control structures, hydraulic jumps, and bridges. Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions, and equations for broad-crested weirs, v-notch weirs, and orifices are presented in Chapter 8. Bridges are detailed in Chapter 9. The hydraulic jump is presented below.

3.6 Hydraulic Jump

A hydraulic jump can occur when flow passes rapidly from supercritical to subcritical depth. The evaluation of a hydraulic jump should consider the high energy loss and erosive forces that are associated with the jump. For rigid-lined facilities such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity. For grass-lined channels, unless the erosive forces are controlled, serious damage can result. Control of jump location is usually obtained by check dams or grade control structures that confine the erosive forces to a protected area. Flexible material such as riprap, rock, or rubble usually affords the most effective protection.

The analysis of the hydraulic jump inside storm sewers must be approximate, because of the lack of data for circular, elliptical, or arch sections. The jump can be approximately located by intersecting the energy grade line of the super-critical and subcritical flow reaches. The primary concerns are whether the pipe can withstand the forces, which may separate the joint or damage the pipe wall, and whether the jump will affect the hydraulic characteristics. The effect on pipe capacity can be determined by evaluating the energy grade line, taking into account the energy lost by the jump. In general, for Froude numbers less than 2.0, the loss of energy is less than 10 percent. French (1985) provides semi-empirical procedures to evaluate the hydraulic jump in circular and other non-rectangular channel sections.

For long box culverts with a concrete bottom, the concerns about jump are the same as for storm sewers. However, the jump can be adequately defined for box culverts/drains and for spillways using the jump characteristics of rectangular sections.

The relationship between variables for a hydraulic jump in rectangular sections can be expressed as:

$$d_2 = -\frac{d_1}{2} + \left(\frac{d_1^{\ 2}}{4} + \frac{2 v_1^{\ 2} d_1}{g} \right)^{1/2} \qquad (3\text{-}22)$$

where:

d_2 = Depth below jump, in feet

d_1 = Depth above jump, in feet

v_1 = Velocity above jump, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

A nomograph for solving Equation 3-22 is presented in Figure 3-12. Additional details on evaluating hydraulic jumps can be found in publications by USDOT, FHWA (HEC-14, 1983), Chow (1959), Peterska (1978), and French (1985).

3.7 Construction and Maintenance Considerations

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities.

Open channels rapidly lose hydraulic capacity without adequate maintenance. Maintenance may include repairing erosion damage, mowing grass, cutting brush, and removing sediment or debris. Brush, sediment, or debris can reduce design capacity and can harm or kill vegetative linings, thus creating the potential for erosion damage during large storm events. Maintenance of vegetation should include the repeated application of fertilizer, irrigation during dry periods, and reseeding or resodding to restore the viability of damaged areas. Additional information is available in Volume 4: TCP-05, 10, 19, 20, 21; PESC-01,02, 04 and 08.

Table 3-1
Maximum Velocities for Comparing Lining Materials

Material	Maximum Velocity [a] (feet/second)
Bare soil	
Silt or fine sand	1.50
Sandy loam	1.75
Silt loam	2.00
Stiff clay	3.75
Sod	4.0
Lapped sod	5.5
Vegetation	Use Table 3-2
Rigid [a]	10

[a] Higher velocities may be acceptable for rigid linings if appropriate protection is provided (see Chapters 9 and 10).

Table 3-2
Maximum Velocities For Vegetative Channel Linings

Vegetation Type	Slope Range (%)	Maximum Velocity [a] (feet per second)
Bermudagrass	0-5	6
	5-10	5
Kentucky bluegrass	0-5	5
Buffalo grass	5-10	4
Grass Mixture	0-5	4
	5-10	3
Lespedeza Sericea Kudzu, alfalfa	0-5	2.5
Annuals	0-5	2.5

[a] Based on erosive soils.

Reference: USDA, TP-61 (1947).

Table 3-3
Recommended Manning's n Values For Artificial Channels

Lining Category [a]	Lining Type	n Value Depth Ranges 0 – 0.5 ft.	0.5 – 2.0 ft.	>2.0 ft.
Rigid	Concrete (Broom or Float Finish)	0.015	0.013	0.013
	Gunite	0.022	0.020	0.020
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch (2.5-cm) d_{50}	0.044	0.033	0.030
	2-inch (5-cm) d_{50}	0.066	0.041	0.034
Rock Riprap [b]	N/A	$n = 0.0395\,(d_{50})^{1/6}$ d_{50} = Diameter of stone for which 50 percent, by weight, of the gradation is finer, in feet		

[a] n values for vegetative linings should be determined using Figure 3-1.

[b] See Section 3.3.6.

Reference: USDOT, FHWA, HEC-15 (1986).

Table 3-4
Classification of Vegetative Covers as to Degree of Retardance

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 30”) (76 cm)
	Yellow bluestem	
	Ischaemum	Excellent stand, tall (average 36”) (91 cm)
B	Kudzu	Very dense growth, uncut
	Bermudagrass	Good stand, tall (average 12”) (30 cm)
	Native grass mixture	
	(little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24”) (61 cm)
	Lespedeza sericea	Good stand, not woody, tall (average 19”) (48 cm)
	Alfalfa	Good stand, uncut (average 11”) (28 cm)
	Weeping lovegrass	Good stand, unmowed (average 13”) (33 cm)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13”) (33 cm)
C	Crabgrass	Fair stand, uncut (10 to 48”) (25 to 122 cm)
	Bermudagrass	Good stand, mowed (average 6”) (15 cm)
	Common lespedeza	Good stand, uncut (average 11”) (28 cm)
	Grass-legume mixture--	
	Summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 to 8 inches) (15 to 20 cm)
	Centipedegrass	Very dense cover (average 6 inches) (15 cm)
	Kentucky bluegrass	Good stand, headed (6 to 12 inches) (15 to 30 cm)
D	Bermudagrass	Good stand, cut to 2.5-inch height (6 cm)
	Common lespedeza	Excellent stand, uncut (average 4.5”) (11 cm)
	Buffalograss	Good stand, uncut (3 to 6 inches) (8 to 15 cm)
	Grass-legume mixture--	
	fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5 inches) (10 to 13 cm)
	Lespedeza sericea	After cutting to 2-inch height (5 cm) Very good stand before cutting
E	Bermudagrass	Good stand, cut to 1.5-inch height (4 cm)
	Bermudagrass	Burned stubble

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Reference: USDA, TP-61 (1947).

Table 3-5
Coefficients For Computing Manning's n Values
For Natural Or Excavated Channels Using Cowan's Equation [a]

Channel Conditions			Values [b]
Material Involved	Earth	n_0	0.020
	Rock Cut		0.025
	Fine Gravel		0.024
	Coarse Gravel		0.028
Degree of Irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of Channel Cross Section	Gradual	n_2	0.000
	Alternating Occasionally		0.005
	Alternating Frequently		0.010-0.015
Relative Effect of Obstructions	Negligible	n_3	0.000
	Minor		0.010-0.015
	Appreciable		0.020-0.030
	Severe		0.040-0.060
Vegetation	Low	n_4	0.005-0.010
	Medium		0.010-0.025
	High		0.025-0.050
	Very High		0.050-0.100
Degree of Meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

[a] Cowan's Equation is presented as Equation 3-1.

[b] From Chow (1959), Table 5-5, page 109.

Table 3-6
Critical Depth Equations For Uniform Flow In Selected Channel Cross Sections

Channel Type [a]	Semi-Empirical Equation [b] For Estimating Critical Depth	Range of Applicability
1. Rectangular [c]	$d_c = \left(\frac{Q^2}{gb^2} \right)^{1/3}$	**N/A**
2. Trapezoidal [c]	$d_c = 0.81 \left(\frac{Q^2}{gz^{0.75} b^{1.25}} \right)^{0.27} - \frac{b}{30z}$	$0.1 < 0.5522 \frac{Q}{b^{2.5}} < 0.4$ For $0.5522 \frac{Q}{b^{2.5}} < 0.1$, use rectangular channel equation
3. Triangular [c]	$d_c = \left(\frac{2Q^2}{gz^2} \right)^{1/5}$	**N/A**
4. Circular [d]	$d_c = 0.325 \left(\frac{Q}{D} \right)^{2/3} + 0.083D$	$0.3 < \frac{d_c}{D} < 0.9$
5. General [e]	$\frac{A^3}{T} = \frac{Q^2}{g}$	N/A

Where:
- d_c = Critical depth, in feet
- Q = Design discharge, in cfs
- g = Acceleration due to gravity, 32.2 feet/second ^{2}
- b = Bottom width of channel, in feet
- z = Side slopes of a channel (horizontal to vertical)
- D = Diameter of circular conduit, in feet
- A = Cross-sectional area of flow, in square feet
- T = Top width of water surface, in feet

[a] See Figure 3-2 for channel sketches.
[b] Assumes uniform flow with the kinetic energy coefficient equal to 1.0.
[c] Reference: French (1985).
[d] Reference: USDOT, FHWA, HDS-4 (1965).
[e] Reference: Brater and King (1976).

TABLE 3-7
Water Surface Profile Computation Form for the Direct Step Method

Location ______________________________

Q = ________	n = ________	S_o = ________	α = ________	y_c = ________	y_n = ________

y (1)	A (2)	R (3)	v (4)	$\alpha v^2/2g$ (5)	E (6)	ΔE (7)	S_f (8)	$\bar{S}_f$ (9)	$S_o - \bar{S}_f$ (10)	Δx (11)	x (12)
1.											
2.											
3.											
4.											
5.											
6.											
7.											
8.											
9.											
10.											
11.											
12.											
13.											
14.											
15.											
16.											
17.											
18.											
19.											
20.											

(8) $S_f = \dfrac{n^2 v^2}{2.22 R^{4/3}}$ (11) $\Delta x = \dfrac{\Delta E}{S_o - \bar{S}_f}$

TABLE 3-8
Water Surface Profile Computation Form for the Standard Step Method

Location ____________													
Q = ____		n = ____		S_o = ____		α = ____		k_e = ____		y_c = ____		y_n = ____	
Station (1)	z (2)	y (3)	A (4)	R (5)	v (6)	$\alpha v^2/2g$ (7)	H (8)	$\overline{S}_f$ (9)	$\overline{S}_f$ (10)	Δx (11)	h_f (12)	h_e (13)	H (14)
1.													
2.													
3.													
4.													
5.													
6.													
7.													
8.													
9.													
10.													
11.													
12.													
13.													
14.													
15.													
16.													
17.													
18.													
19.													
20.													

(9) $S_f = \dfrac{n^2v^2}{2.22R^{4/3}}$ (12) $h_f = \Delta x\ \overline{S}_f$ (13) $h_e = k_e \dfrac{v^2}{2g}$.

Table 3-9
DIRECT STEP METHOD RESULTS FOR EXAMPLE 3-5

y (1)	A (2)	R (3)	v (4)	$\alpha v^2/2g$ (5)	E (6)	ΔE (7)	S_f (8)	$\overline{S_f}$ (9)	$S_o - \overline{S_f}$ (10)	Δx (11)	x (12)
5.00	150.00	3.54	2.667	0.1217	5.1217	--	0.000370	--	--	--	--
4.80	142.08	3.43	2.819	0.1356	4.9356	0.1861	0.000433	0.000402	0.001198	155	155
4.60	134.32	3.31	2.979	0.1517	4.7517	0.1839	0.000507	0.000470	0.001130	163	318
4.40	126.72	3.19	3.156	0.1706	4.5706	0.1811	0.000598	0.000553	0.001047	173	491
4.20	119.28	3.08	3.354	0.1925	4.3925	0.1781	0.000705	0.000652	0.000948	188	679
4.00	112.00	2.96	3.572	0.2184	4.2184	0.1741	0.000850	0.000778	0.000822	212	891
3.80	104.88	2.84	3.814	0.2490	4.0490	0.1694	0.001020	0.000935	0.000665	255	1,146
3.70	101.38	2.77	3.948	0.2664	3.9664	0.0826	0.001132	0.001076	0.000524	158	1,304
3.60	97.92	2.71	4.085	0.2856	3.8856	0.0808	0.001244	0.001188	0.000412	196	1,500
3.55	96.21	2.68	4.158	0.2958	3.8458	0.0398	0.001310	0.001277	0.000323	123	1,623
3.50	94.50	2.65	4.233	0.3067	3.8067	0.0391	0.001382	0.001346	0.000254	154	1,777
3.47	93.48	2.63	4.278	0.3131	3.7831	0.0236	0.001427	0.001405	0.000195	121	1,898
3.44	92.45	2.61	4.326	0.3202	3.7602	0.0229	0.001471	0.001449	0.000151	152	2,050
3.42	91.80	2.60	4.357	0.3246	3.7446	0.0156	0.001500	0.001486	0.000114	137	2,187
3.40	91.12	2.59	4.388	0.3292	3.7292	0.0154	0.001535	0.001518	0.000082	188	2,375

Note: Q = 400 cfs n = 0.025 $S_o = 0.0016$ $\alpha = 1.10$ $y_c = 2.22$ ft. $y_n = 3.36$ ft.

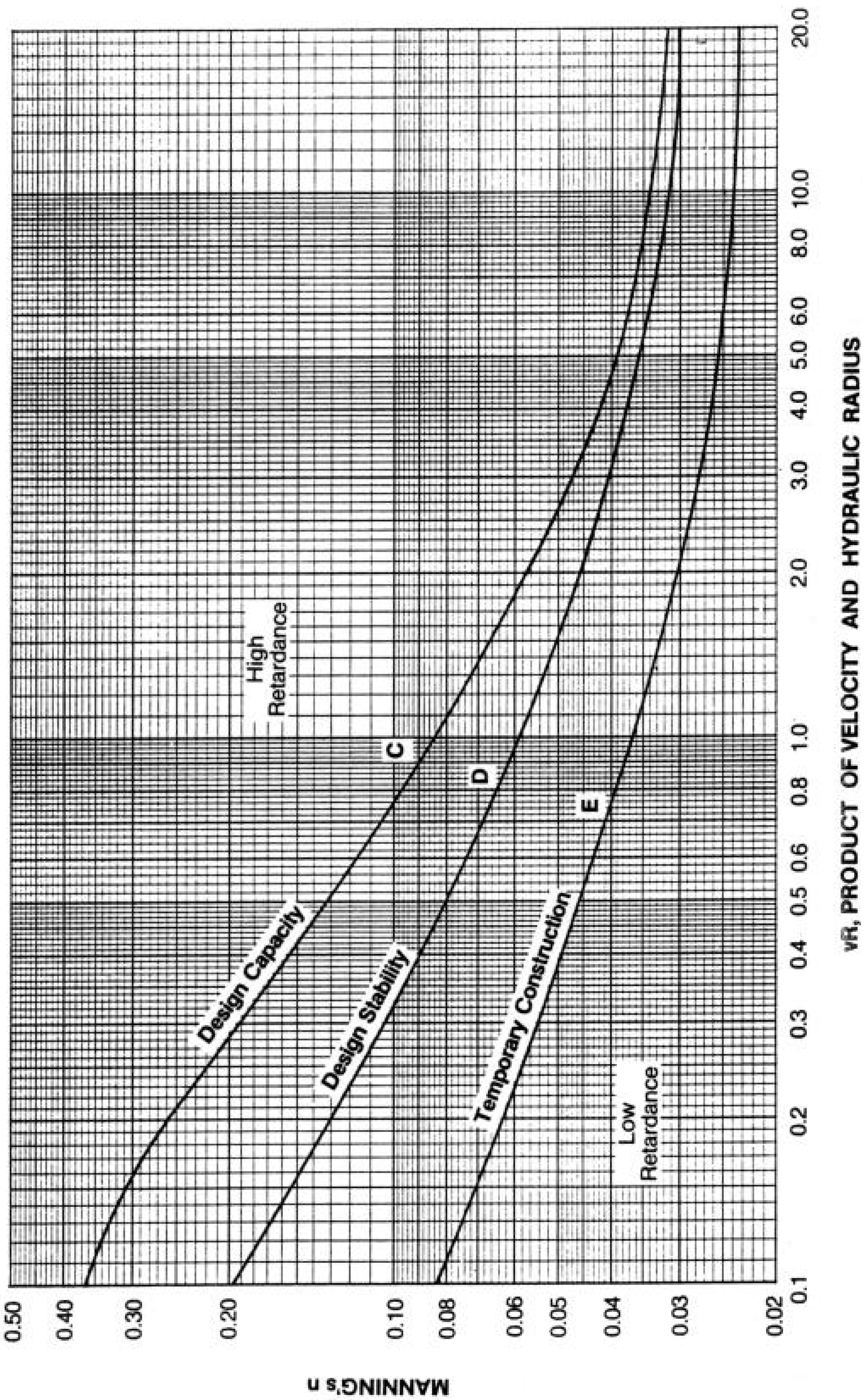
Reference: Chow (1959).

Table 3-10
STANDARD STEP METHOD RESULTS FOR EXAMPLE 3-6

Station (1)	z (2)	y (3)	A (4)	R (5)	v (6)	$\alpha v^2/2g$ (7)	H (8)	S_f (9)	$\overline{S}_f$ (10)	Δx (11)	h_f (12)	h_e (13)	H (14)
0 + 00	605.000	5.00	150.00	3.54	2.667	0.1217	605.122	0.000370	--	--	--	--	605.122
1 + 55	605.048	4.80	142.08	3.43	2.819	0.1356	605.184	0.000433	0.000402	155	0.062	0	605.184
3 + 18	605.109	4.60	134.32	3.31	2.979	0.1517	605.261	0.000507	0.000470	163	0.077	0	605.261
4 + 91	605.186	4.40	126.72	3.19	3.156	0.1706	605.357	0.000598	0.000553	173	0.096	0	605.357
6 + 79	605.286	4.20	119.28	3.08	3.354	0.1925	605.479	0.000705	0.000652	188	0.122	0	605.479
8 + 91	605.426	4.00	112.00	2.96	3.572	0.2184	605.644	0.000850	0.000778	212	0.165	0	605.644
11 + 46	605.633	3.80	104.88	2.84	3.814	0.2490	605.882	0.001020	0.000935	255	0.238	0	605.882
13 + 04	605.786	3.70	101.38	2.77	3.948	0.2664	606.052	0.001132	0.001076	158	0.170	0	606.052
15 + 00	605.999	3.60	97.92	2.71	4.085	0.2856	606.285	0.001244	0.001188	196	0.233	0	606.285
16 + 23	606.146	3.55	96.21	2.68	4.158	0.2958	606.442	0.001310	0.001277	123	0.157	0	606.442
17 + 77	606.343	3.50	94.50	2.65	4.233	0.3067	606.650	0.001382	0.001346	154	0.208	0	606.650
18 + 98	606.507	3.47	93.48	2.63	4.278	0.3131	606.820	0.001427	0.001405	121	0.170	0	606.820
20 + 50	606.720	3.44	92.45	2.61	4.326	0.3202	607.040	0.001471	0.001449	152	0.220	0	607.040
21 + 87	606.919	3.42	91.80	2.60	4.357	0.3246	607.244	0.001500	0.001486	137	0.204	0	607.244
23 + 75	607.201	3.40	91.12	2.59	4.388	0.3292	607.530	0.001535	0.001518	188	0.286	0	607.530

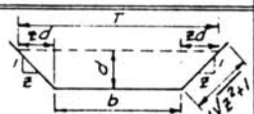
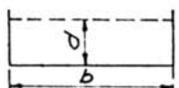
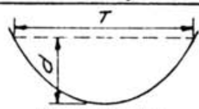
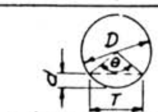
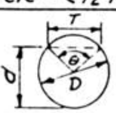
Note: Q = 400 cfs   n = 0.025 $S_o = 0.0016$ $\alpha = 1.10$ $h_e = 0$ $y_c = 2.22$ ft. $y_n = 3.36$ ft.

Reference: Chow (1959).



Reference: USDA, TP-61 (1947).

Figure 3-1
Manning's n Value for Vegetated Channels

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
 Circle - < ½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
 Circle - > ½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

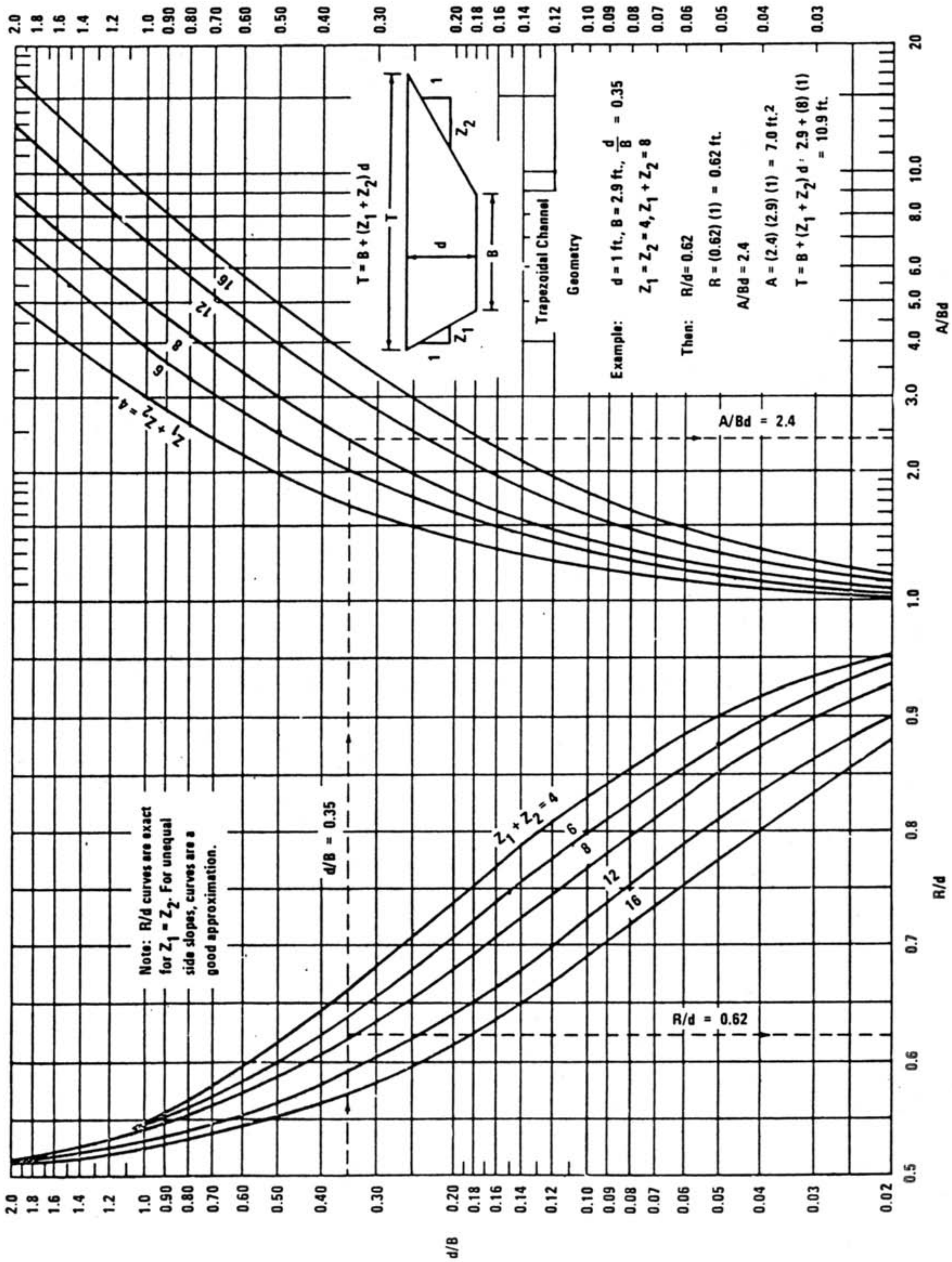
[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $P = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

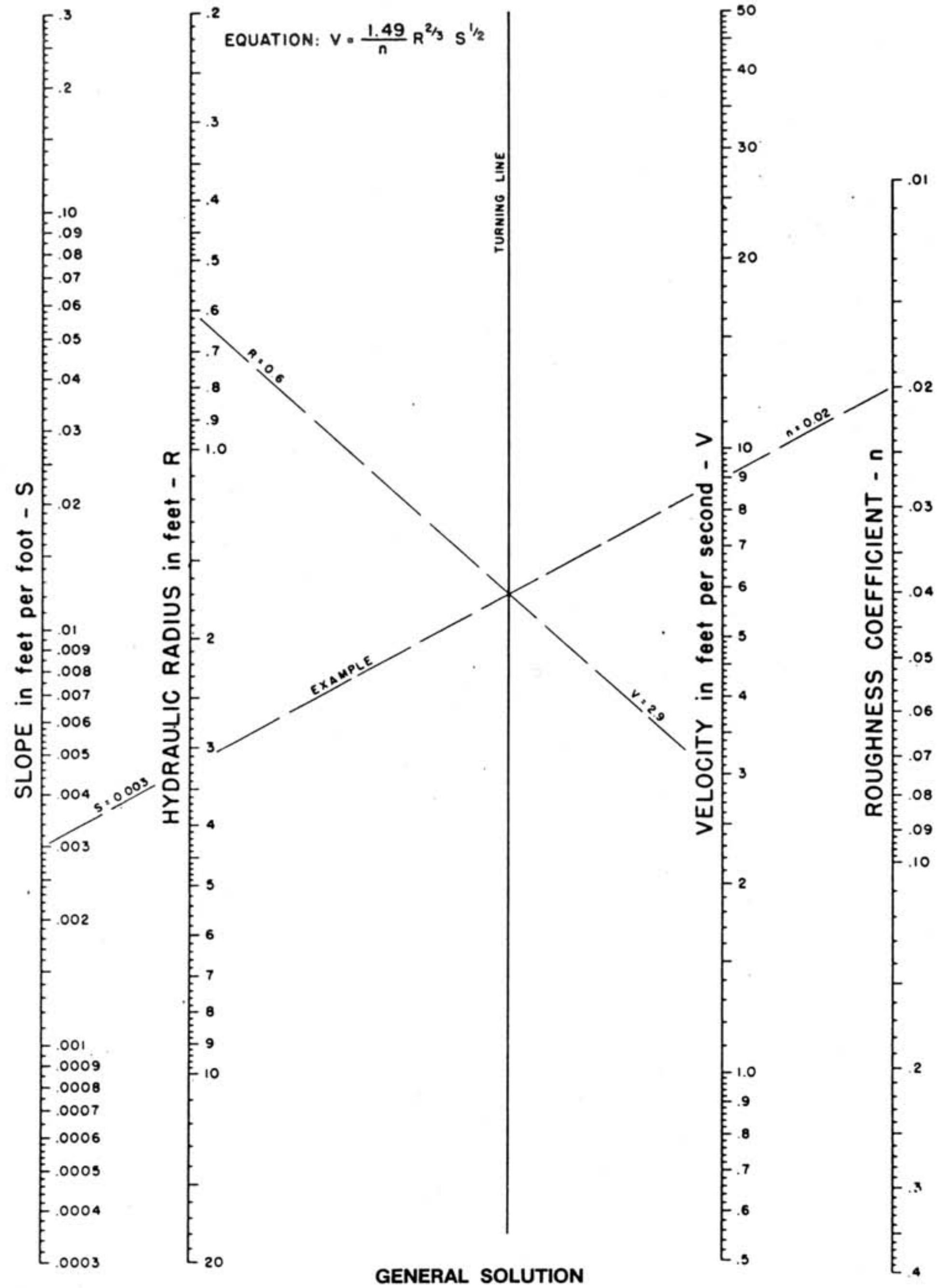
Reference: USDA, SCS, NEH-5 (1956).

Figure 3-2
Open Channel Geometric Relationships for Various Cross Sections



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3-3
Trapezoidal Channel Geometry



Reference: USDOT, FHWA, HDS-3 (1961).

Figure 3-4
Nomograph for the Solution of Manning's Equation

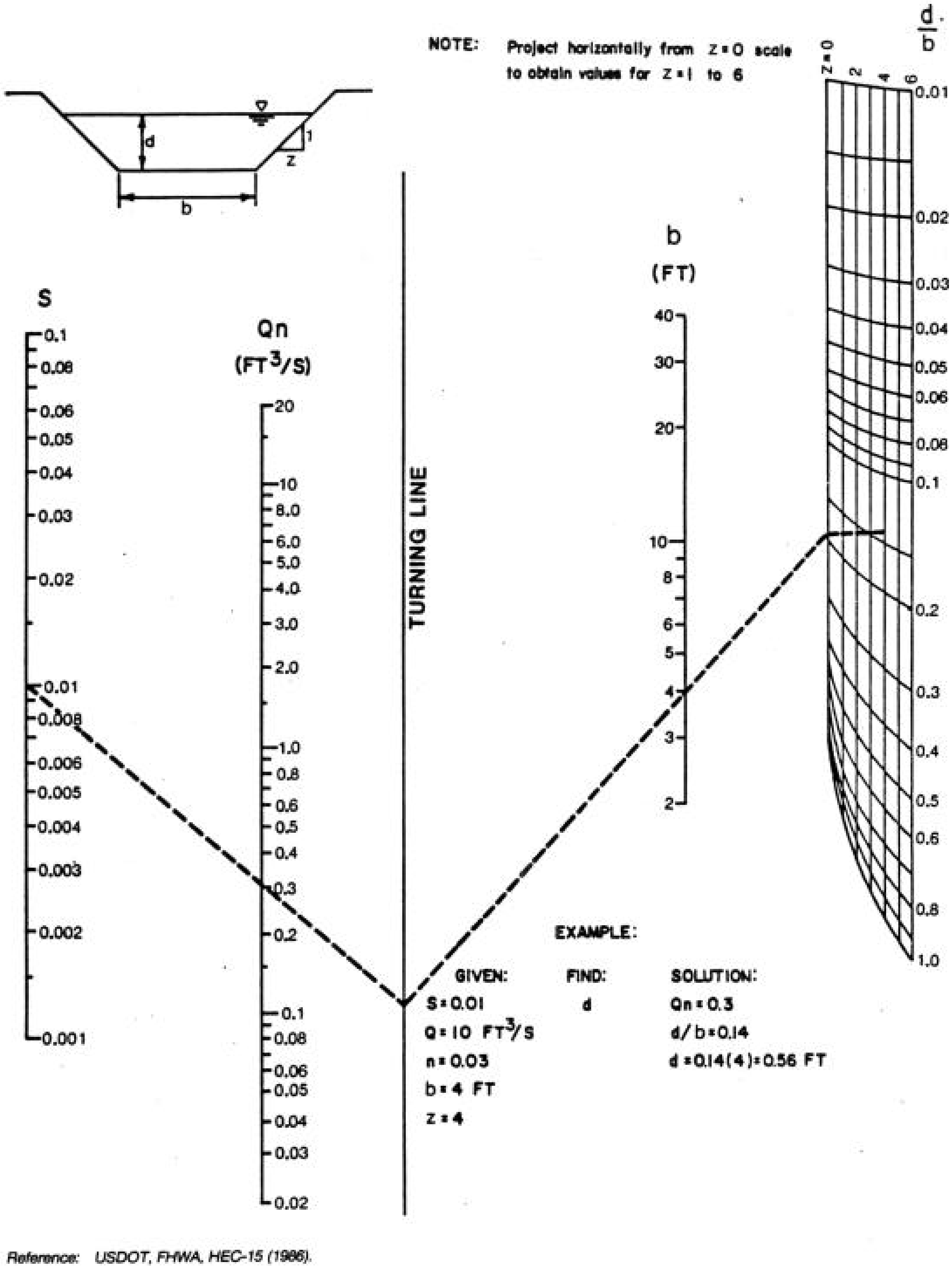


Figure 3-5
Solution of Manning's Equation for Trapezoidal Channels

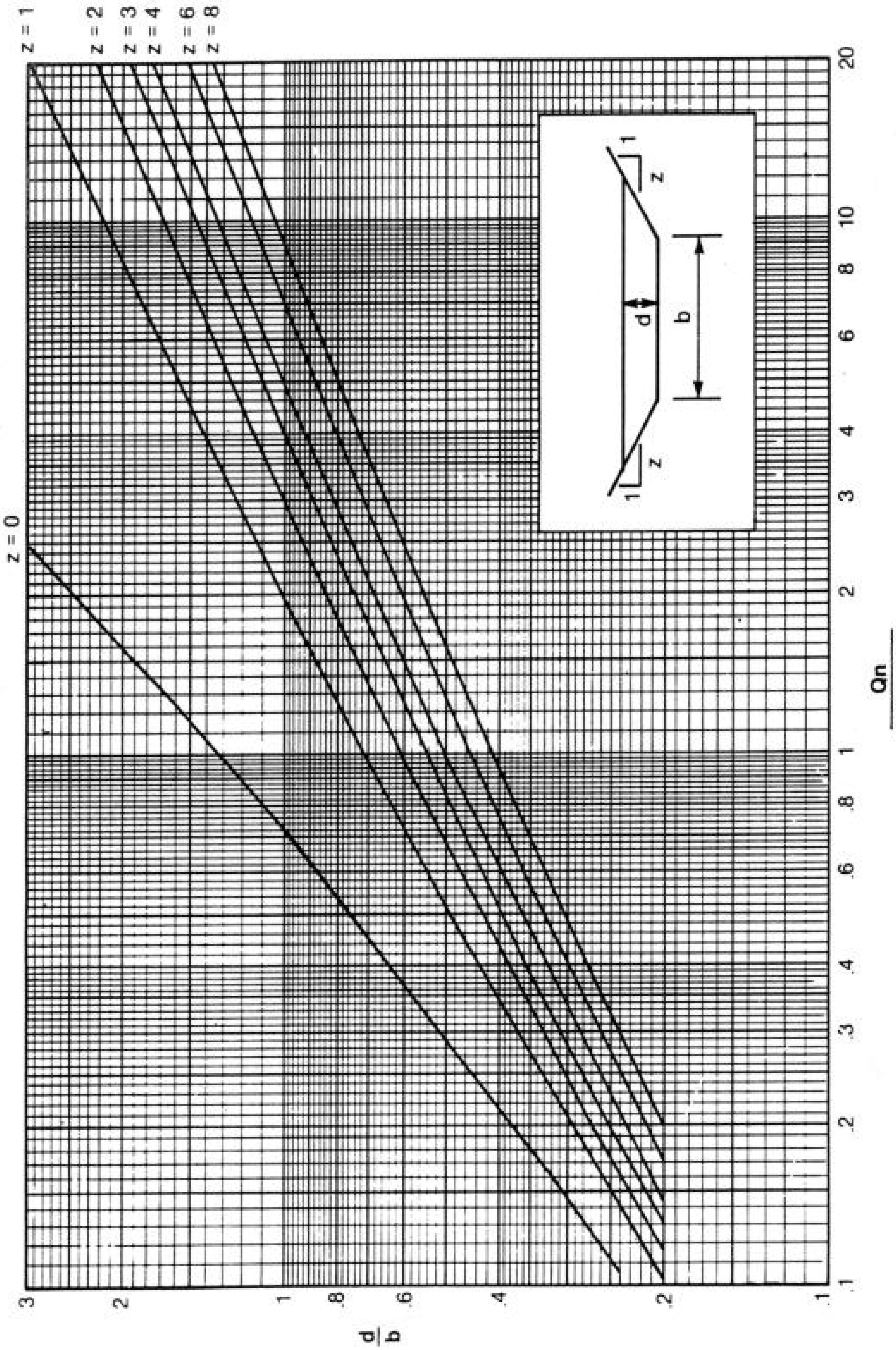
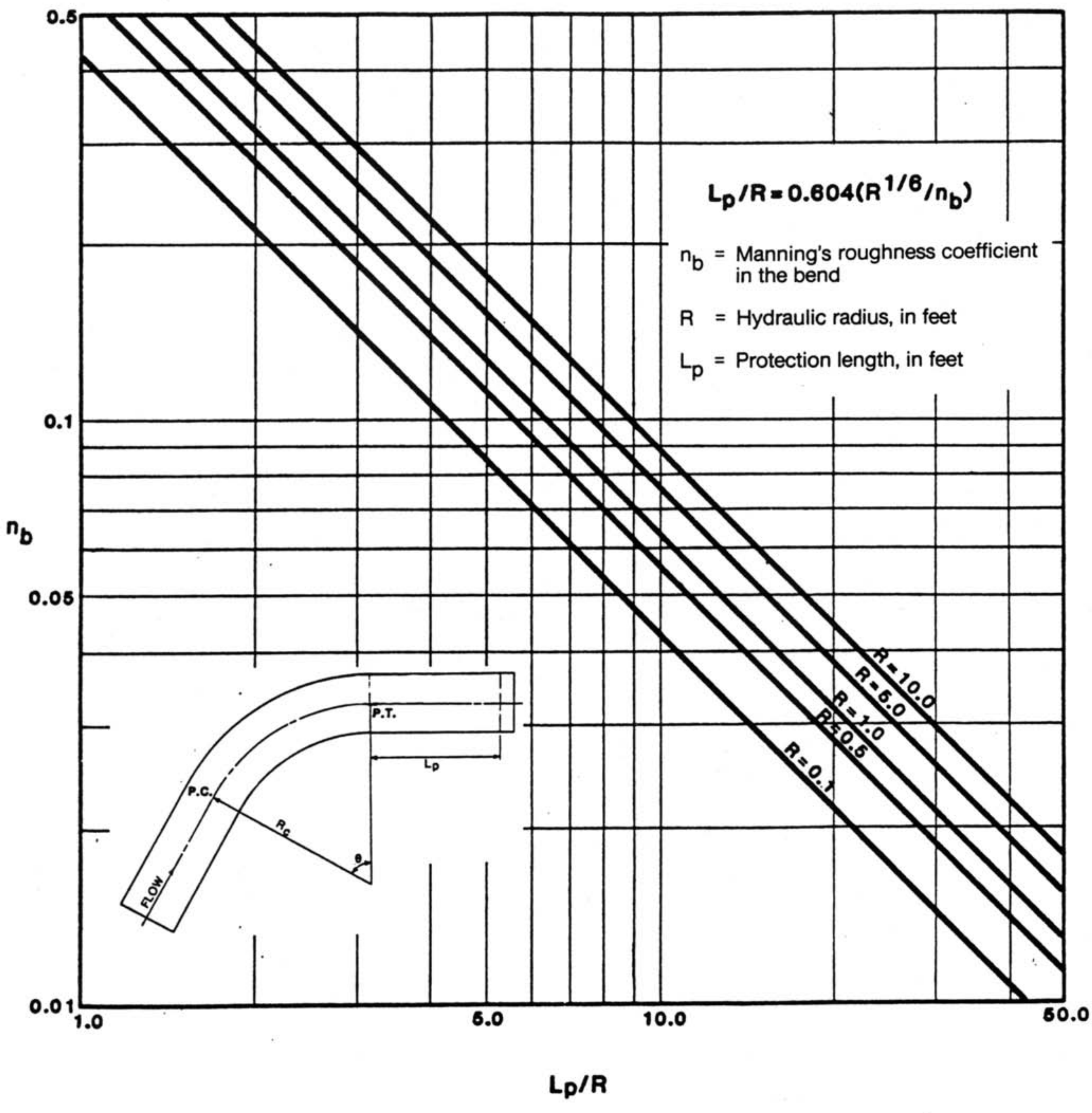
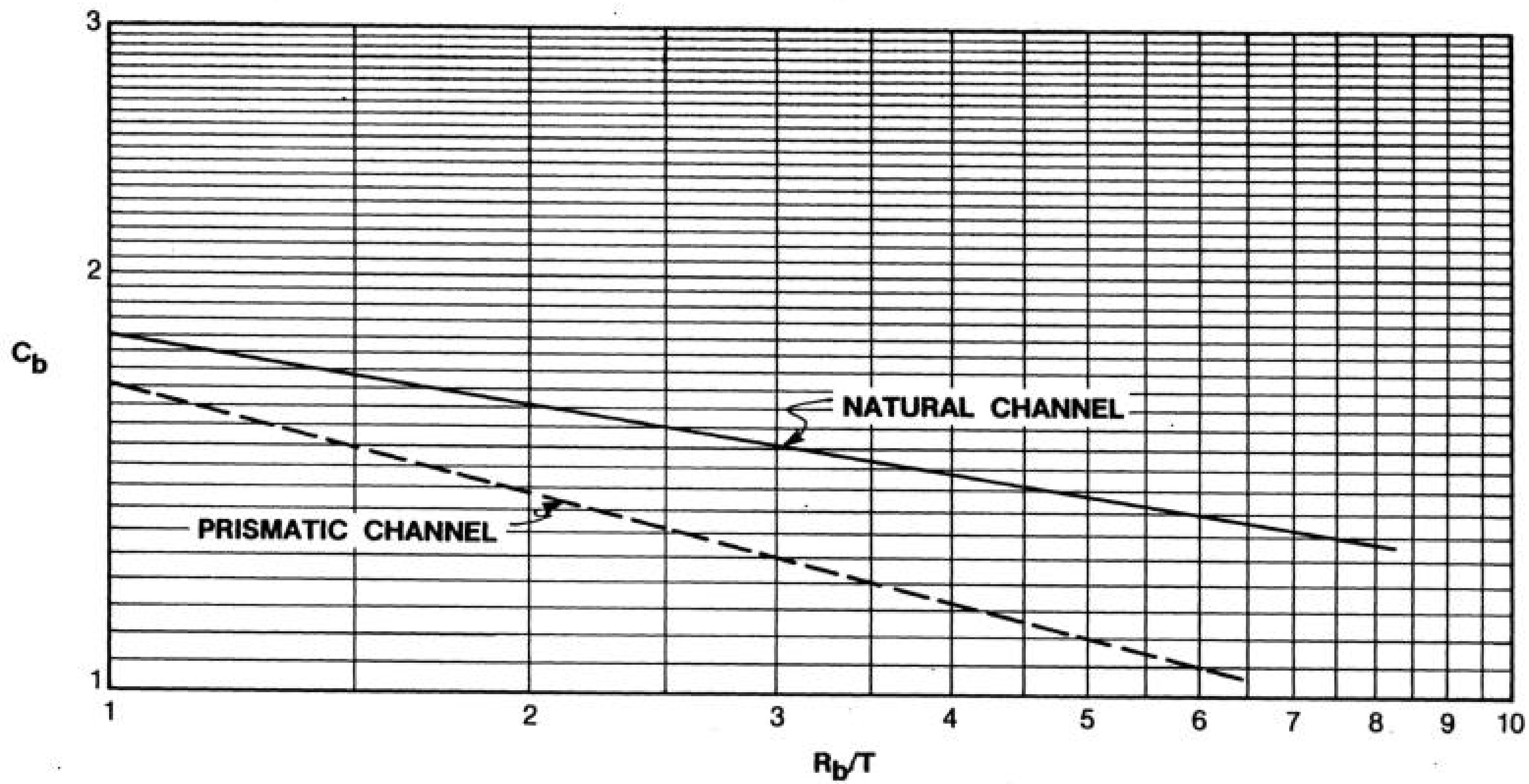


Figure 3-6
Trapezoidal Channel Capacity Chart



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3-7
Protection Length, L_p, Downstream of Channel Bend

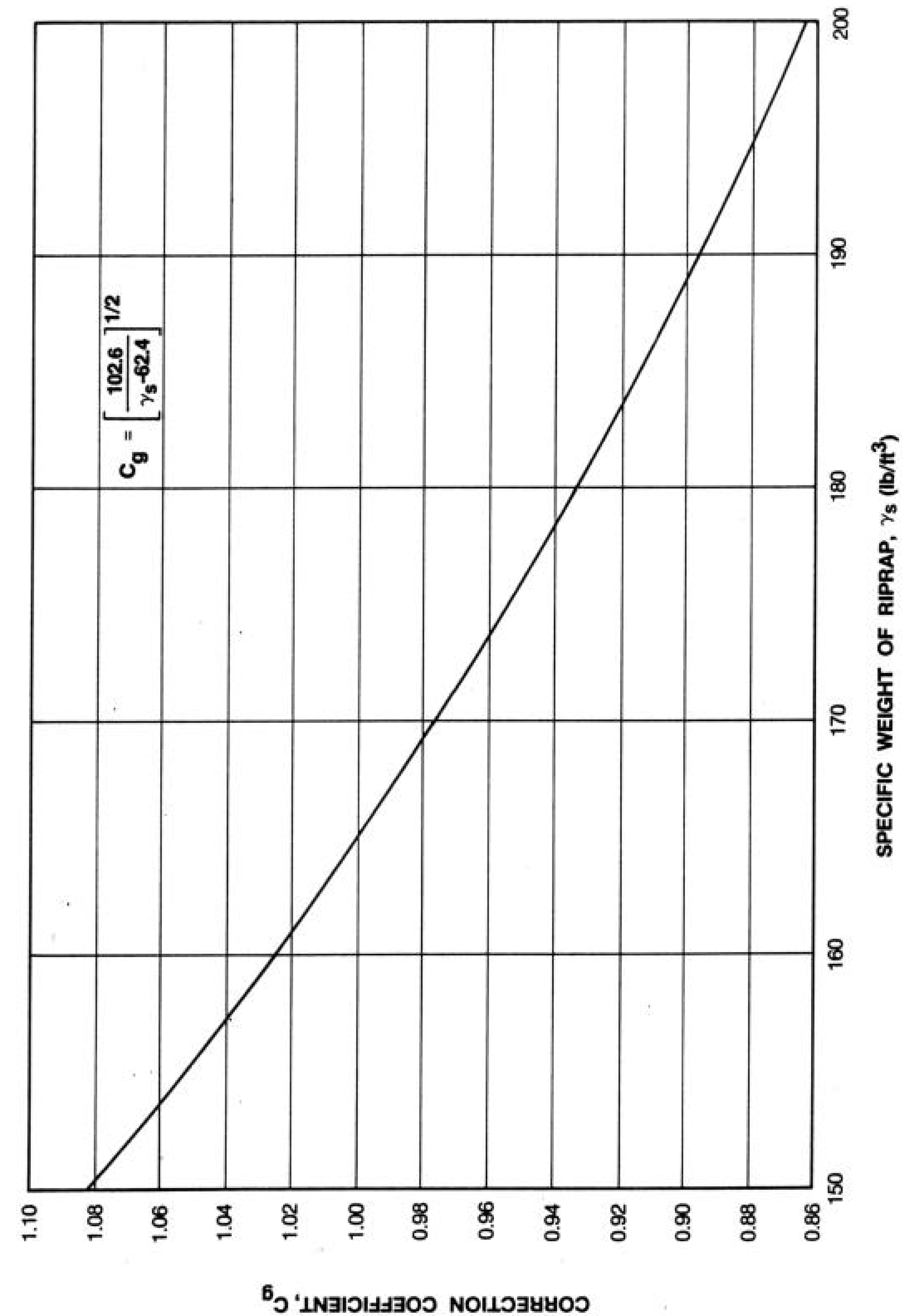


To obtain effective velocity, multiply known velocity by C_b.

T = Channel Top Width
R_b = Centerline Bend Radius
C_b = Correction Coefficient

Reference: Maynord (1987).

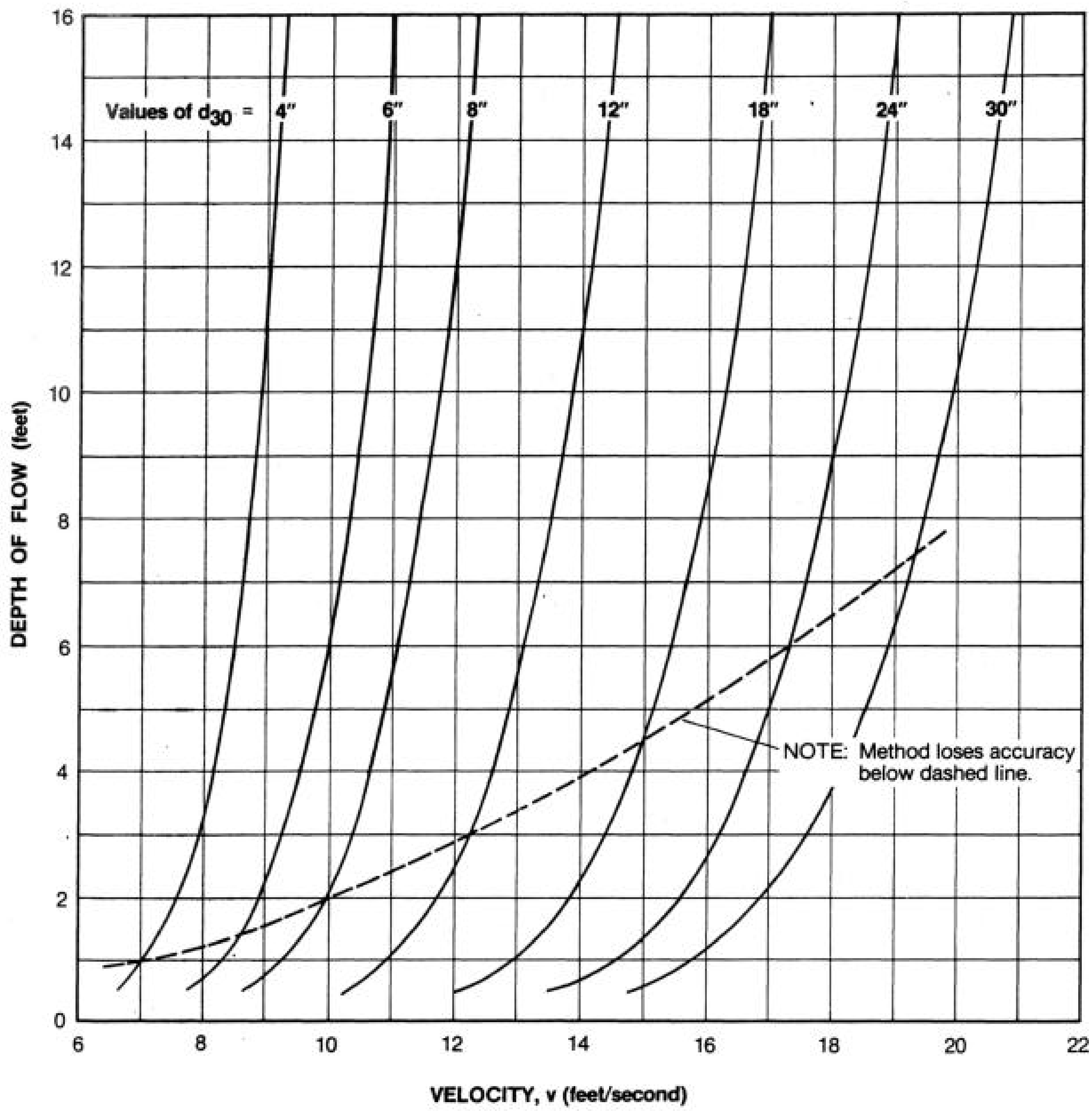
Figure 3-8
Riprap Lining Bend Correction Coefficient



C_g = Correction Coefficient

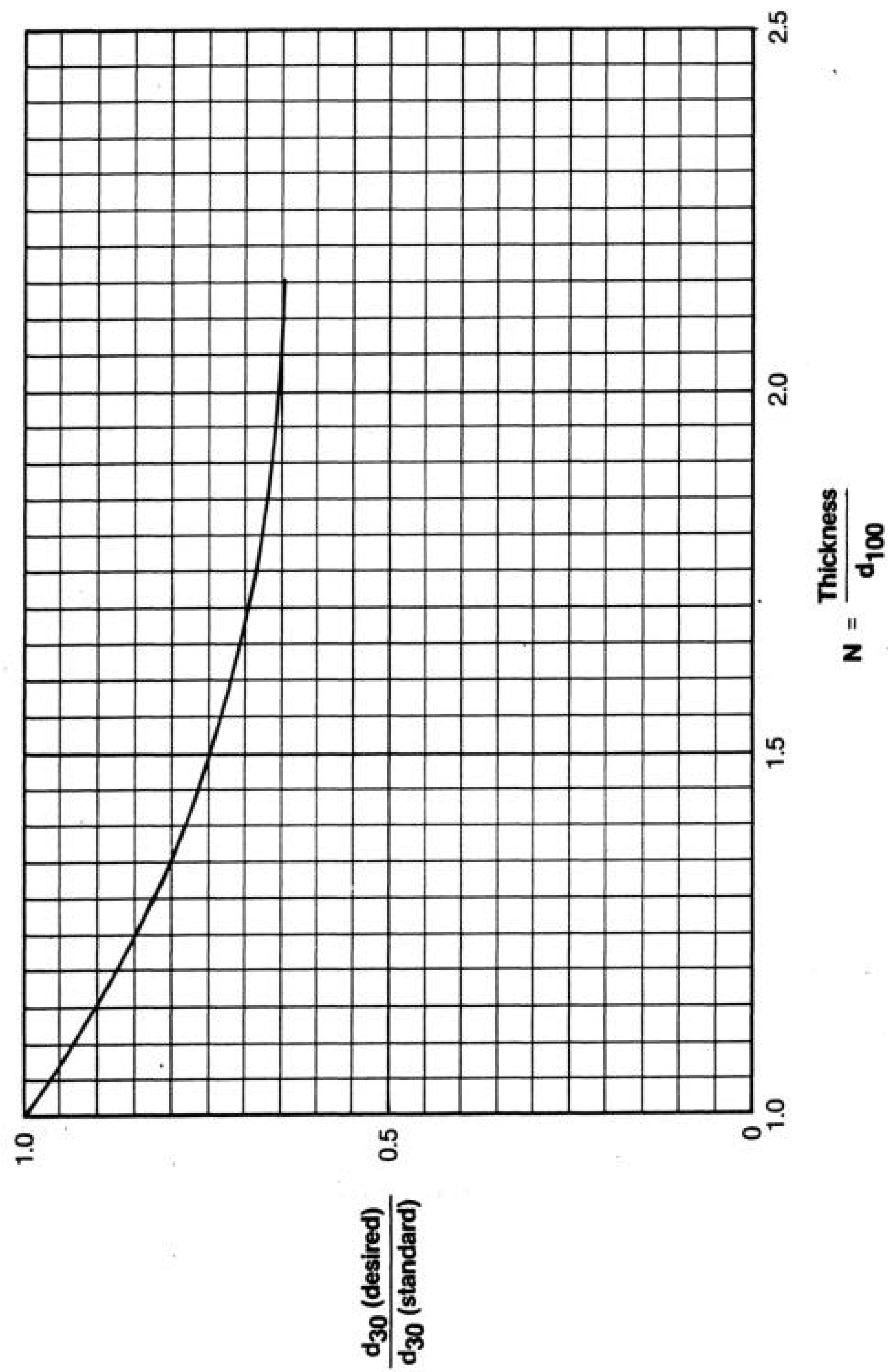
To obtain effective velocity, multiply known velocity by C_g.

Figure 3-9
Riprap Lining Specific Weight Correction Coefficient



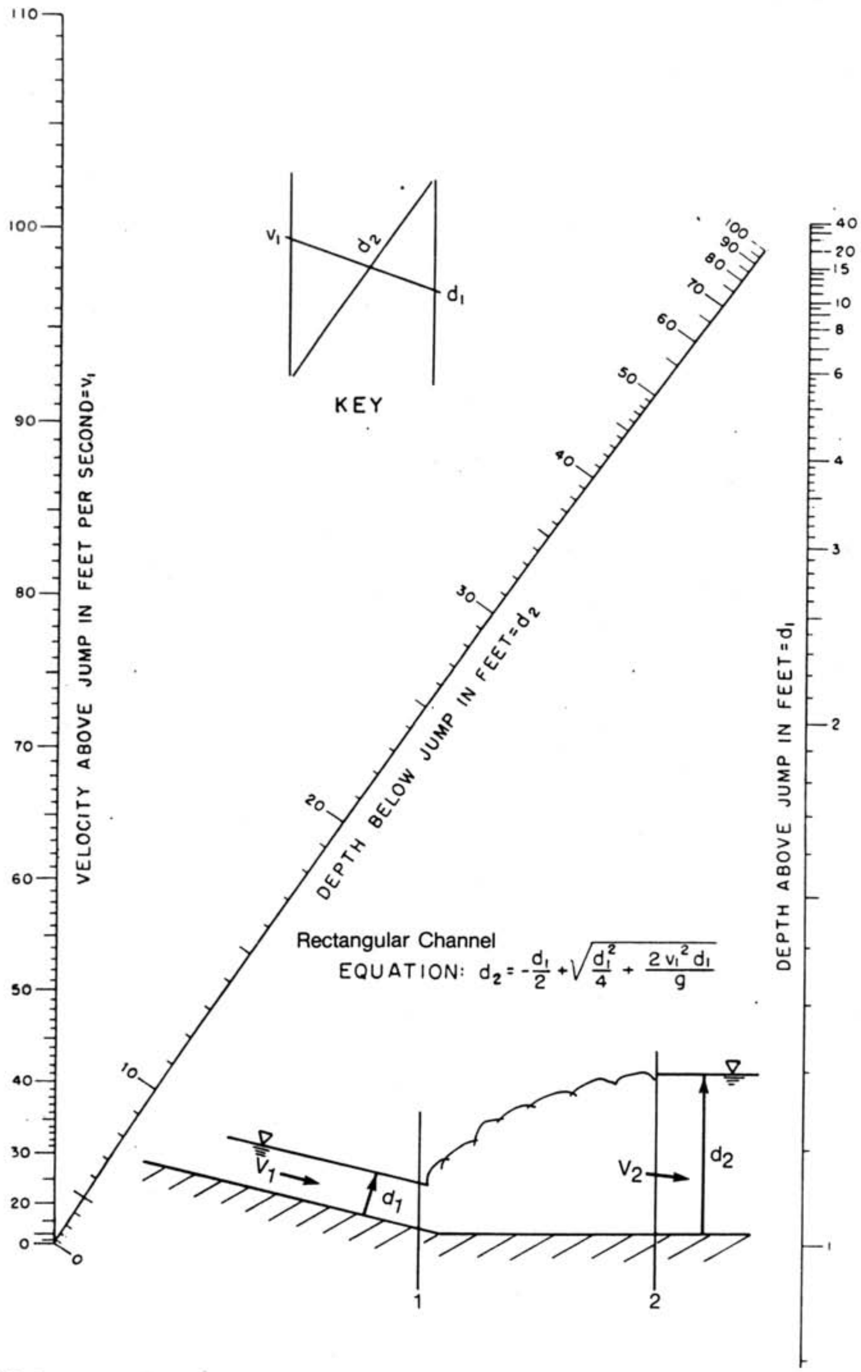
Reference: Reese (1988).

Figure 3-10
Riprap Lining d_{30} Stone Size as a Function
of Mean Velocity and Depth



Reference: Maynord (1987).

Figure 3-11
Riprap Lining Thickness Adjustment for $d_{85}/d_{15} = 2.0$ to 2.3



Reference: U.S. Department of the Interior (1973).

Figure 3-12
Nomograph for Solving the Rectangular Channel Hydraulic Jump Equation

CHAPTER 4
GUTTER AND INLET HYDRAULICS

Chapter 4
GUTTER AND INLET HYDRAULICS

Synopsis

The level of service of facilities that provide drainage of roadway surfaces should be consistent with the level of service of the roadway. Guidelines are given for evaluating roadway features and design criteria as they relate to gutter and inlet hydraulics. Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grated and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Nashville specific design information is presented for typical details contained in the standard drawings of the Metropolitan Nashville and Davidson County Subdivision Specifications for Streets and Roads.

4.1 Design Criteria

The following design criteria are typically important for gutter and inlet capacity calculations:

1. Return period
2. Spread
3. Inlet types and spacing
4. Manning's n values
5. Grade
6. Cross slope
7. Curb and gutter sections
8. Roadside and median ditches
9. Bridge decks

4.1.1 Return Period

The design storm return period for pavement drainage should be consistent with the frequency selected for other components of the drainage system.

4.1.2 Spread

For multi-laned curb and gutter or guttered roadways with no parking, it is not practical to avoid travel lane flooding when grades are flat (1.0 percent). However, flooding should never exceed the lane adjacent to the gutter (or shoulder) for design conditions. Standard practice in Nashville is to limit maximum stormwater spread to 8 feet, measured from the face of the curb. Municipal bridges with curb and gutter should also use this criterion. For single-lane roadways, at least 8 feet of roadway should remain unflooded for design conditions.

4.1.3 Inlet Types and Spacing

Inlet types shall be selected from Standard Drawings E-110, E-111, and E-112 of the Metropolitan Nashville and Davidson County Subdivision Specifications for Streets and Roads. Inlets shall be located or spaced in such a manner that the design curb flow does not exceed the spread criterion of 8 feet.

No flow will be allowed to cross intersecting streets unless approved by MWS. In addition, curb and gutter inlets should not be built in curb returns.

4.1.4 Manning's n Values

Manning's n values for various pavement surfaces are presented in **Table 4-1**. Section 4.2.1 provides hydraulic capacity data for three Nashville standard pavement sections using a Manning's n value of 0.014.

4.1.5 Grade

Curb and gutter grades that are equal to pavement slopes shall not exceed 13 percent or fall below 1 percent without approval from MWS. A minimum longitudinal gradient is more important for curbed pavements, which are susceptible to stormwater spread. Flat gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.

4.1.6 Cross Slope

The design of pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. In most Nashville design situations, cross slopes will be defined by the standard pavement sections given on Standard Drawings E-300, E-301, and E-302 of the Metropolitan Nashville and Davidson County Subdivision Specifications for Streets and Roads. The standard Nashville cross slope is a 4-inch crown over the edge of the pavement. The most common cross slopes are 0.0222 and 0.0175 foot/foot, for 30-and 38-inch pavement widths, respectively.

When three or more lanes are inclined in the same direction on multi-lane pavements, it is desirable for each successive pair of lanes, or the portion thereof outward from the first two lanes from the crown line, to have an increased slope. The two lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs, or portions thereof outward, should be increased by about 0.5 to 1.0 percent. Where three or more lanes are provided in each direction, the maximum pavement cross slope should be limited to 4 percent.

4.1.7 Curb and Gutter Sections

Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities. Curb and curb and gutter details are presented in Standard Drawing E-133 of the

Metropolitan Nashville and Davidson County Subdivision Specifications for Streets and Roads. Standard gutter width is 18 inches. Gutters are on the same cross slope as the pavement on the high side and depressed with a steeper cross slope on the low side, usually 1 inch per foot. Typical practice is to place curbs at the outside edge of shoulders or parking lanes on low speed facilities. Standard Drawing E-133 also shows a typical cross section for a curb without the gutter section.

Metropolitan Nashville and Davidson County encourages the use of curb cuts allowing runoff from roads or parking lanes to be routed to biofilter swales or biofilter strips. If this approach is to be applied to parking lot islands, it is preferable to have inlets raised 6 to 12-inches. Biofilters and this approach are discussed and illustrated in Volume 4 PTP-05.

4.1.8 Roadside and Median Ditches

Roadside ditches are commonly used with uncurbed roadway sections to convey pavement runoff and upgradient area runoff that drains toward the pavement. Right-of-way limitations prevent use of roadside ditches in densely developed urban areas. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Procedures for sizing roadside ditches are provided in Chapter 3.

Curbed highway sections are relatively inefficient at conveying water, and the area tributary to the gutter section should be kept to a minimum to reduce the hazard from water on the pavement. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by ditches as appropriate.

It is preferable to slope median areas and inside shoulders to a center swale, to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.

4.1.9 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets on scuppers have a higher potential for clogging by debris. Bridge deck constructibility usually requires a constant cross slope, so the guidelines in Section 4.1.6 do not apply. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable grade for bridge deck drainage should be 1.0 percent. When bridges are placed at a vertical curve and the grade is less than 1.0 percent, the gutter spread should be checked to ensure a safe, reasonable design.

Scuppers are the recommended method of deck drainage, because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. They also have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope pavement, slope protection, navigation channels, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains, although sod flumes may be used for extremely minor flows in some areas.

4.2 Gutter Flow Calculations

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = \frac{0.56}{n} S_x^{5/3} S^{1/2} T^{8/3} \qquad (4\text{-}1)$$

where:

Q =Gutter flow rate, in cfs

n = Manning's roughness coefficient

S_x = Pavement cross slope, in feet/foot

S = Grade, in feet/foot

T = Width of flow or spread, in feet

A nomograph for solving Equation 4-1 is presented in **Figure 4-1**. Composite cross slope situations require the use of **Figure 4-2** along with Figure 4-1. Manning's n values for various pavement surfaces are presented in Table 4-1.

4.2.1 Nashville Standard Pavement Sections

Capacity equations have been developed for the following three typical pavement cross sections used in Nashville:

Section Designation	Roadway Classification	Pavement Width (ft)	Gutter Width (inches)	Pavement Cross Slope (ft/ft)
A	Residential	30	None	0.0222
B	Residential	30	18	0.0222
C	Residential Collector Or Commercial and Industrial	38	18	0.0175

The hydraulic capacity equations given below are based on an 8-foot pavement spread and a Manning's n value of 0.014.

For design section A:

$$Q = 17.85\ S^{1/2} \qquad (4\text{-}2)$$

For design section B:

$$Q = 21.98\ S^{1/2} \qquad (4\text{-}3)$$

For design section C:

$$Q = 15.93\ S^{1/2} \qquad (4\text{-}4)$$

where:

Q = Total pavement section flow, in cfs

S = Grade, in feet/foot

Graphical solutions to the equations are presented in **Figures 4-3, 4-4, and 4-5**. To use the figures, enter the x-axis with the grade of the gutter and find the flow rate on the y-axis using the appropriate total pavement section flow curve. Grate inlet intercept curves, also shown on these figures, are discussed in Section 4.4.1.

4.2.2 Uniform Cross Slopes

The nomograph in Figure 4-1 is used with the following procedures to find gutter capacity for uniform cross slopes.

<u>Condition 1: Find spread, given gutter flow.</u>

1. Determine input parameters, including grade, S, cross slope, S_x, gutter flow, Q,and Manning's n.

2. Draw a line between the S and S_x scales and note where it intersects the turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

4. Read the value of the spread, T, at the intersection of the line from Step 3 and the spread scale.

<u>Condition 2: Find gutter flow, given spread.</u>

1. Determine input parameters, including grade, S, cross slope, S_x, spread, T, and Manning's n.

2. Draw a line between the S and S_x scales and note where it intersects the turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

4. For Manning's n values of 0.016, the gutter capacity, Q, from Step 3 is selected. For other Manning's n values, the gutter capacity times n, Qn, is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

4.2.3 Composite Gutter Sections

Figure 4-2, in combination with Figure 4-1, can be used to find the flow in a gutter with width, W, less than the total spread, T. Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets. The procedures below are used to evaluate composite gutter sections.

<u>Condition 1: Find spread, given gutter flow.</u>

1. Determine input parameters, including grade, S, cross slope, S_x, depressed section slope, S_w, depressed section width, W, Manning's n, gutter flow, Q, and a trial value of the gutter capacity above the depressed section, Q_s.

2. Calculate the gutter flow in W, Q_w, using the equation:

$Q_w = Q - Q_s$ (4-5)

3. Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 4-2 to find an appropriate value of W/T.

4. Calculate the spread, T, by dividing the depressed section width, W, by the value of W/T from Step 3.

5. Find the spread above the depressed section, T_s, by subtracting W from the value of T obtained in Step 4.

6. Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 4-1 (see Section 4.2.2, Condition 2).

7. Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

<u>Condition 2: Find gutter flow, given spread.</u>

1. Determine input parameters, including spread, T, spread above the depressed section, T_s, cross slope, S_x, grade, S, depressed section slope, S_w, depressed section width, W, Manning's n, and depth of gutter flow, d.

2. Use Figure 4-1 to determine the capacity of the gutter section above the depressed section, Q_s. Use the procedure in Section 4.2.2, Condition 2, substituting T_s for T.

3. Calculate the ratios W/T and S_w/S_x , and, from Figure 4-2, find the appropriate value of E_o (the ratio of Q_w/Q).

4. Calculate the total gutter flow using the equation:

$Q = Q_s \div (1 - E_o)$ (4-6)

where:

Q = Gutter flow rate, in cfs

Q_s = Flow capacity of the gutter section above the depressed section, in cfs

E_o = Ratio of frontal flow to total gutter flow (Q_w /Q)

5. Calculate the gutter flow in width, W, using Equation 4-5.

4.2.4 Example Problems

Example 4-1. Gutter Flow for Standard Pavement Sections

Find the allowable gutter flow for standard pavement sections A, B, and C (as defined in Section 4.2.1) using a grade of 1 percent and 3 percent.

Gutter flows for the conditions can be found using Equations 4-2, 4-3, and 4-4 or found directly using Figures 4-3, 4-4, and 4-5.

Pavement Section	Grade (%)	Allowable Gutter Flow (cfs)
A	1	1.8
A	3	3.1
B	1	2.2
B	3	3.8
C	1	1.6
C	3	2.8

Example 4-2. Gutter Flow for Composite Sections

Find the gutter flow for the following pavement conditions:

$T = 6$ feet

$T_s = 6 - 1.5 = 4.5$ feet

$S_x = 0.03$ foot/foot

$S = 0.040$ foot/foot

$S_w = 0.0833$ foot/foot

$W = 1.5$ feet

$n = 0.014$

$d = 0.0999 + 0.125 = 0.225$ foot

1. Use Figure 4-1 to find the gutter section capacity above the depressed section.

$Qn = 0.038$ cfs

$Q_s = (0.038) / (0.014) = 2.7$ cfs

2. Calculate $W/T = 1.5/6 = 0.25$ and

$S_w/S_x = 0.0833/0.03 = 2.78$

Use Figure 4-2 to find $E_o = 0.64$

3. Calculate the gutter flow using Equation 4-6:

$Q = (2.7) / (1 - 0.64)$

$Q = 7.5$ cfs

4. Calculate the gutter flow in width, W, using Equation 4-5:

$Q_w = 7.5 - 2.7$

$Q_w = 4.8$ cfs

4.3 Combination Inlets

4.3.1 Continuous Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone.

4.3.2 Sump Conditions

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity.

4.4 Grate Inlets

Grates are efficient for intercepting pavement drainage if clogging by debris is properly controlled. Grate inlets will intercept all of the gutter flow passing over the front of the grate if the grate is sufficiently long and the gutter flow does not splash over the grate. The portion of side flow intercepted will depend on the cross slope of the pavement, length of grate, and flow velocity.

Procedures to determine the capacity of grate inlets placed on continuous grade and at sump locations are presented below.

4.4.1 Continuous Grade (Nashville Standards)

Inlet intercept charts have been developed for Nashville standard grate inlets located on a continuous grade. Inlet details are given on Standard Drawings E-110, E-111, and E-112. Because inlet intercept is a function of both approach flow and inlet characteristics, the charts apply to full gutter flow conditions.

Figure 4-3 is used for pavement design section A (i.e., 30-foot pavement with curb and S_x = 0.0222 foot/foot), with a Manning's roughness coefficient of 0.014 and a stormwater spread of 8 feet. The four curves shown in Figure 4-3 are the total pavement section flow and the intercepts for a single, double, and triple Nashville standard grate inlet. All flow rates are shown as a function of grade. Figures 4-4 and 4-5 present similar information for standard pavement cross sections B and C, as defined on the charts and in Section 4.2.1.

To use the figures, enter the x-axis with the grade of the gutter and find the intercepted flow rate on the y-axis using the curve for the appropriate number (1, 2, or 3) of grate inlets.

4.4.2 Continuous Grade (General)

The ratio of frontal flow intercepted to total frontal flow, R_f, is determined using **Figure 4-6**. The side flow interception efficiency, R_s, is determined using **Figure 4-7**. Steps for using the figures are given below.

1. Determine the following input parameters:

   Grate type (see Standard Drawings E-160 and E-161; grate types M and K are approximately equivalent to the P-1-1/8 grate shown in Figure 4-6)

   Frontal width of grate = W, in feet

   Gutter flow rate = Q, in cfs

   Gutter average velocity = v, in feet/second

   Grate length = L, in feet

   Cross slope = S_x, in feet/foot

Ratio of frontal flow = E_o, from Figure 4-2

2. Enter Figure 4-6 on the x-axis with the length of grate, L, and draw a vertical line upward to the appropriate grate type curve (P-1-1/8 for types M and K). From the point of intersection, draw a line horizontally to the intersection with the appropriate velocity line, v. From the point of intersection, draw a vertical line downward to find the value of R_f.

3. Enter Figure 4-7 on the y-axis with the value of S_x and draw a line horizontally to the intersection with the appropriate length line, L. From the point of intersection, draw a vertical line to the intersection with the appropriate velocity curve. From the point of intersection, draw a line horizontally to the y-axis and read a value of R_s.

4. Calculate the grate inlet interception with the equation:

$$Q_i = Q\ R_f E_o + R_s\ (1\text{-}E_o) \qquad (4\text{-}7)$$

where

Q_i = Grate inlet intercept, in cfs

Q = Total gutter flow, in cfs

R_f = Ratio of frontal flow intercepted to total frontal flow, from Step 2 (Figure 4-6)

E_o = Ratio of frontal flow to total flow, from Figure 4-2

R_s = Side flow interception efficiency, from Step 3 (Figure 4-7)

4.4.3 Sump Conditions (Nashville Standards)

The hydraulic capacity of Nashville standard grate inlets operating under sump conditions is shown in **Figure 4-8**. Depth is measured at the curb referenced to the gutter flow line. Figure 4-8 is developed for flow depths up to the top of curb (6 inches). Under these conditions, the inlet will operate under weir control.

To use the figure, enter the x-axis with the depth of flow measured at the curb and find the inlet flow on the y-axis for the appropriate number (1, 2, or 3) of inlets.

4.4.4 Sump Conditions (General)

Because grated inlets in sump conditions are subject to clogging, a curb opening or slot is required as a supplemental inlet. The capacity of grate inlets operating as weirs or orifices (i.e., in sump conditions) can be evaluated using **Figure 4-9**. Grate size, as it affects the depth at which a grate begins operating as an orifice, varies as indicated in Figure 4-9. Hydraulic capacity in the transitional zone from weir to orifice flow can be approximated by drawing a curve between the lines representing the perimeter and net area of the grate selected. Steps for using **Figure** 4-9 are presented below.

<u>Condition 1: Find depth, d, given required inlet capacity for 100 percent intercept, Q.</u>

1. Determine input parameters, including perimeter, P, clear opening area, A, allowance for clogging by debris, and required inlet capacity for 100 percent intercept, Q. The clear opening area is the grate area minus the area occupied by longitudinal and lateral bars. A reduction factor to account for clogging should then be applied. A 50 percent reduction value is suggested as a minimum. Greater values should be considered for highly critical locations. Tilt bar and curved vane grates are not recommended for sump conditions.

2. Enter the x-axis with Q and draw a vertical line to the intersection with the appropriate P value (if weir flow) or A value with allowance for clogging (if orifice flow). If Q occurs in the transitional zone, draw a curve between the lines representing the perimeter and net area of the grate selected.

3. From the point of intersection in Step 2, draw a horizontal line to find the depth of gutter flow, d. If the depth of flow is too high, return to Step 1 using a larger inlet.

<u>Condition 2: Find inlet capacity, Q, given depth, d.</u>

1. Determine input parameters, including depth, d, perimeter, P, clear opening area, A, and allowance for clogging by debris. The clear opening area is the grate area minus the area occupied by longitudinal and lateral bars. A reduction factor to account for clogging should then be subtracted. A 50 percent reduction value is suggested as a minimum. Greater values should be considered for highly critical locations. Tilt bar and curved vane grates are not recommended for sump locations.

2. Enter the y-axis with d, draw a horizontal line to the right, and locate the intersection with the appropriate P value (if weir flow) or A value with allowance for clogging (if orifice flow). If d occurs in the transitional zone, draw a curve between the lines representing the perimeter and net area of the grate selected.

3. From the point of intersection obtained in Step 2, draw a vertical line downward to obtain the grate inlet capacity. If the inlet capacity is not suitable, return to Step 1 using a new inlet size.

4.4.5 Example Problems

Example 4-3. Grate Inlet Intercept on Grade

Using data from Example 4-1, find the grate inlet intercepted flow for single, double, and triple inlets.

Pavement Section	Grade (%)	Allowable Gutter Flow (cfs)	Intercepted Flow (cfs) Single	Double	Triple
A	1	1.8	0.9	1.3	1.4
A	3	3.1	1.4	1.8	2.2
B	1	2.2	1.3	1.6	1.8
B	3	3.8	2.2	2.5	2.8
C	1	1.6	0.9	1.2	1.3
C	3	2.8	1.7	1.8	2.1

Example 4-4. Grate Inlet at Sump

Find the intercepted flow for standard single, double, and triple grate inlets located at a sump with an allowable spread of 8 feet for standard pavement section B.

For 8 feet of spread on pavement section B, the depth of flow at the curb is calculated as follows:

$d = (6.5)\ (0.0222) + (1.5)\ (0.0833)$

$d = 0.1443 + 0.125 = 0.2693$ feet

Using Figure 4-8, the following intercepted flow estimates are obtained:

Number of Inlets	Intercepted Flow (cfs)
1	1.5
2	2.0
3	2.3

Table 4-1
MANNING'S n VALUES FOR STREET AND PAVEMENT GUTTERS

Type of Gutter or Pavement	***Manning's n***
Design value for Nashville charts (Figures 4-3, 4-4, and 4-5)	0.014
Concrete gutter, troweled finish	0.012
Asphalt pavement	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement	
Smooth	0.013
Rough	0.015
Concrete pavement	
Float finish	0.014
Broom finish	0.016
For gutters where sediment may accumulate, increase values of n by	0.002

Reference: USDOT, FHWA, HDS-3 (1961).

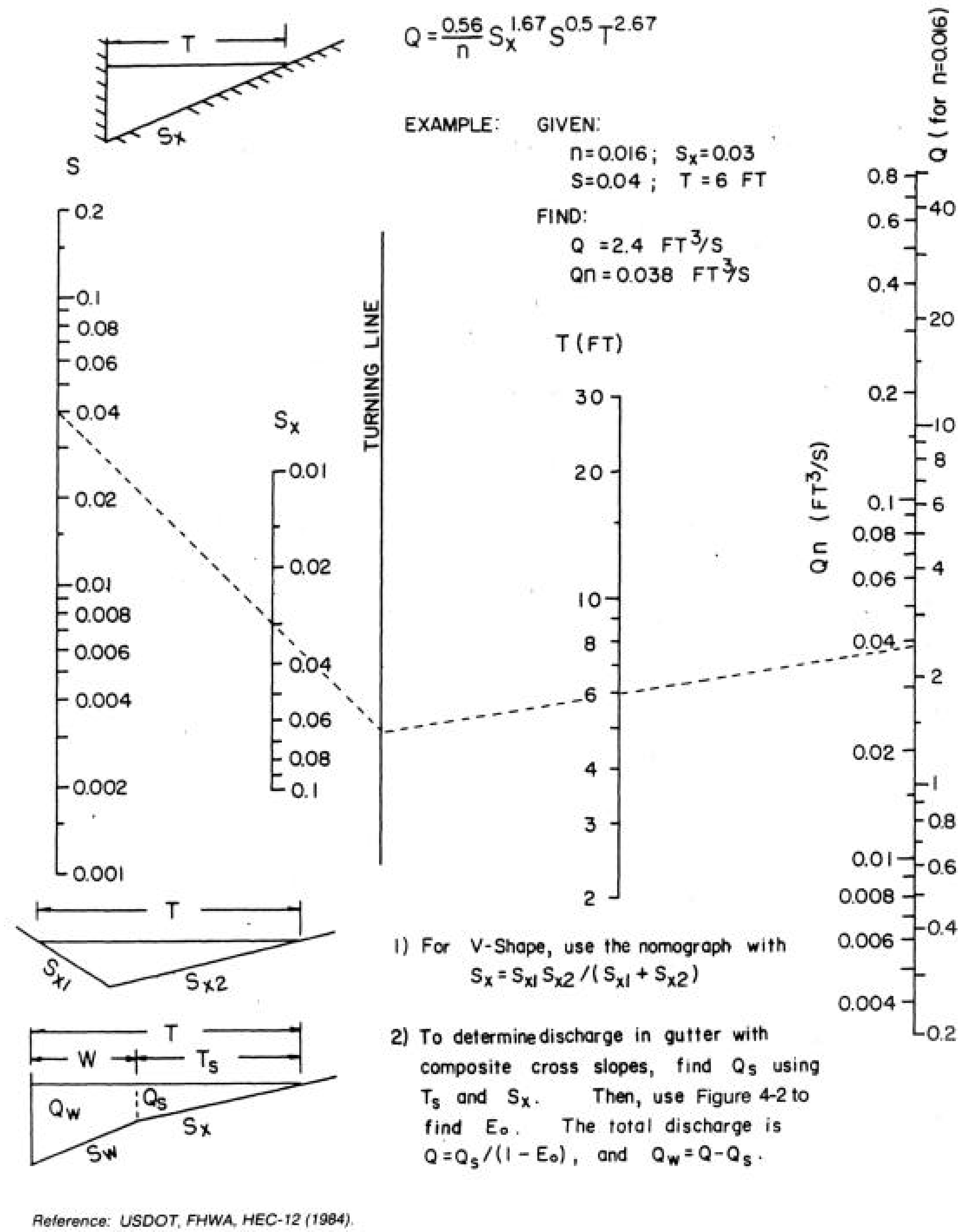
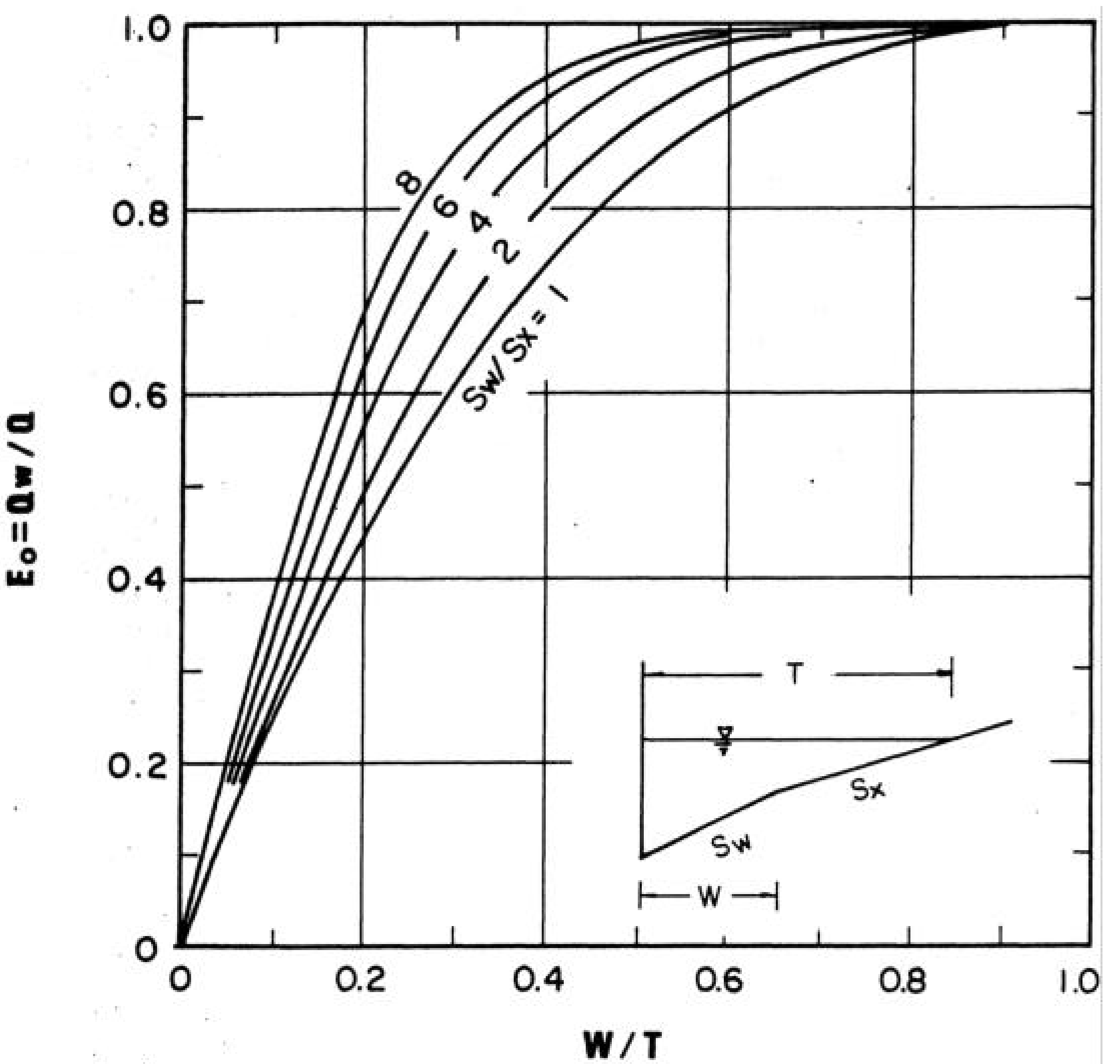


Figure 4-1
Nomograph for Flow in Triangular Gutter Sections



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 4-2
Ratio of Frontal Flow to Total Gutter Flow

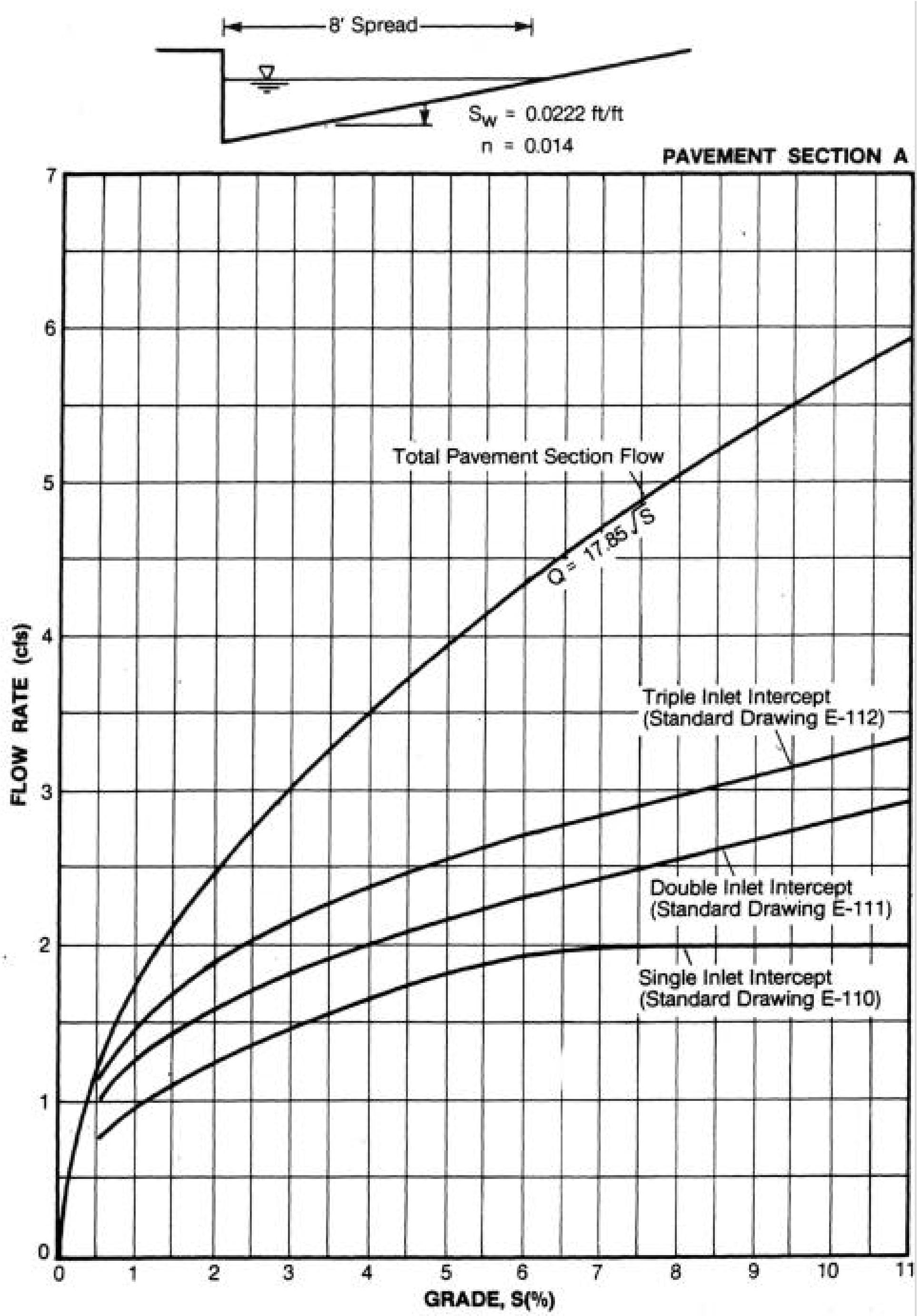


Figure 4-3
Pavement Section Flow and Intercept for 30-foot Pavement
With Curb (No Gutter) and 8-foot Spread

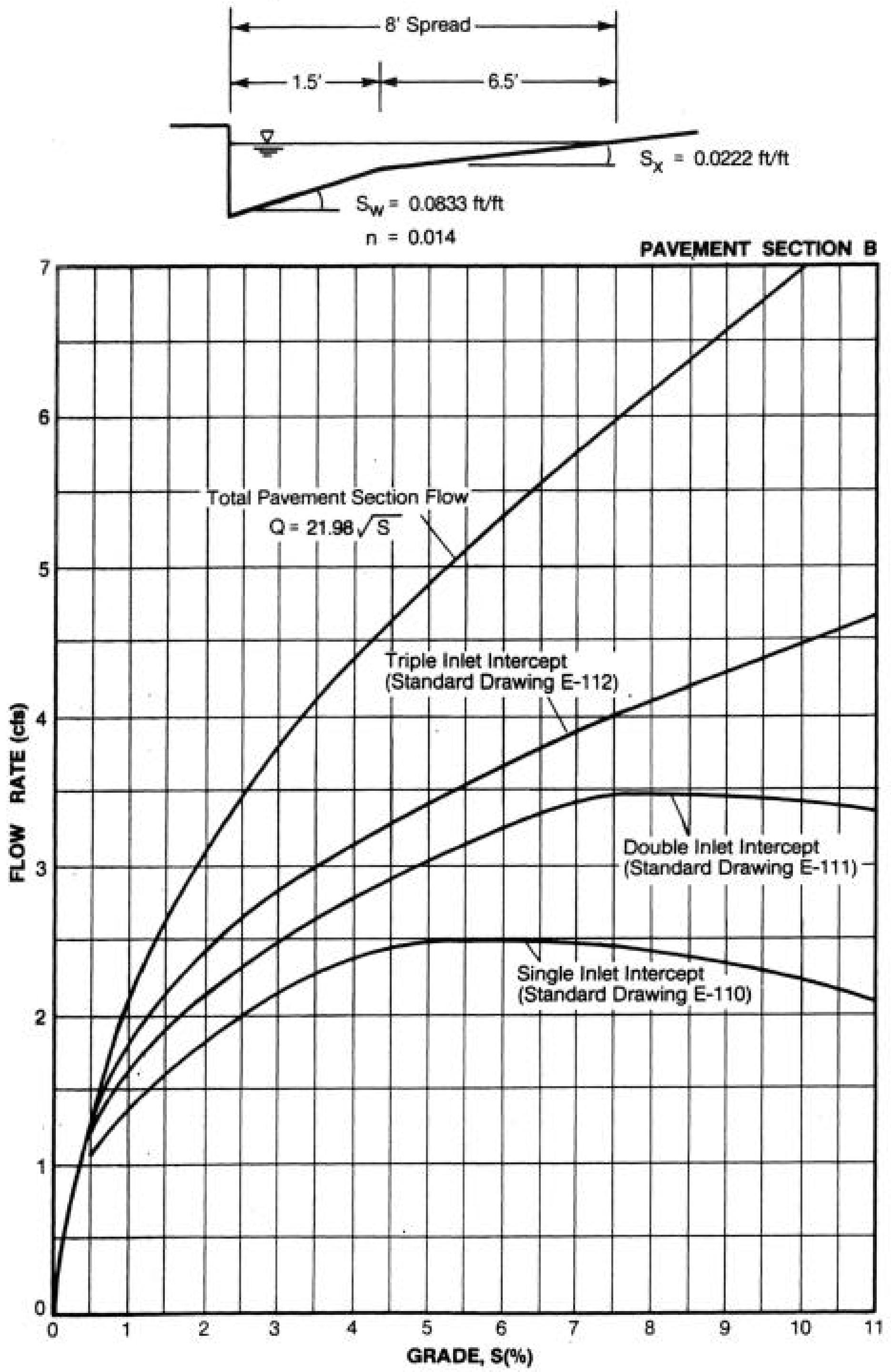


Figure 4-4
Pavement Section Flow and Intercept for 30-foot Pavement
with 24-inch Curb and Gutter (18-in Gutter) and 8-foot Spread

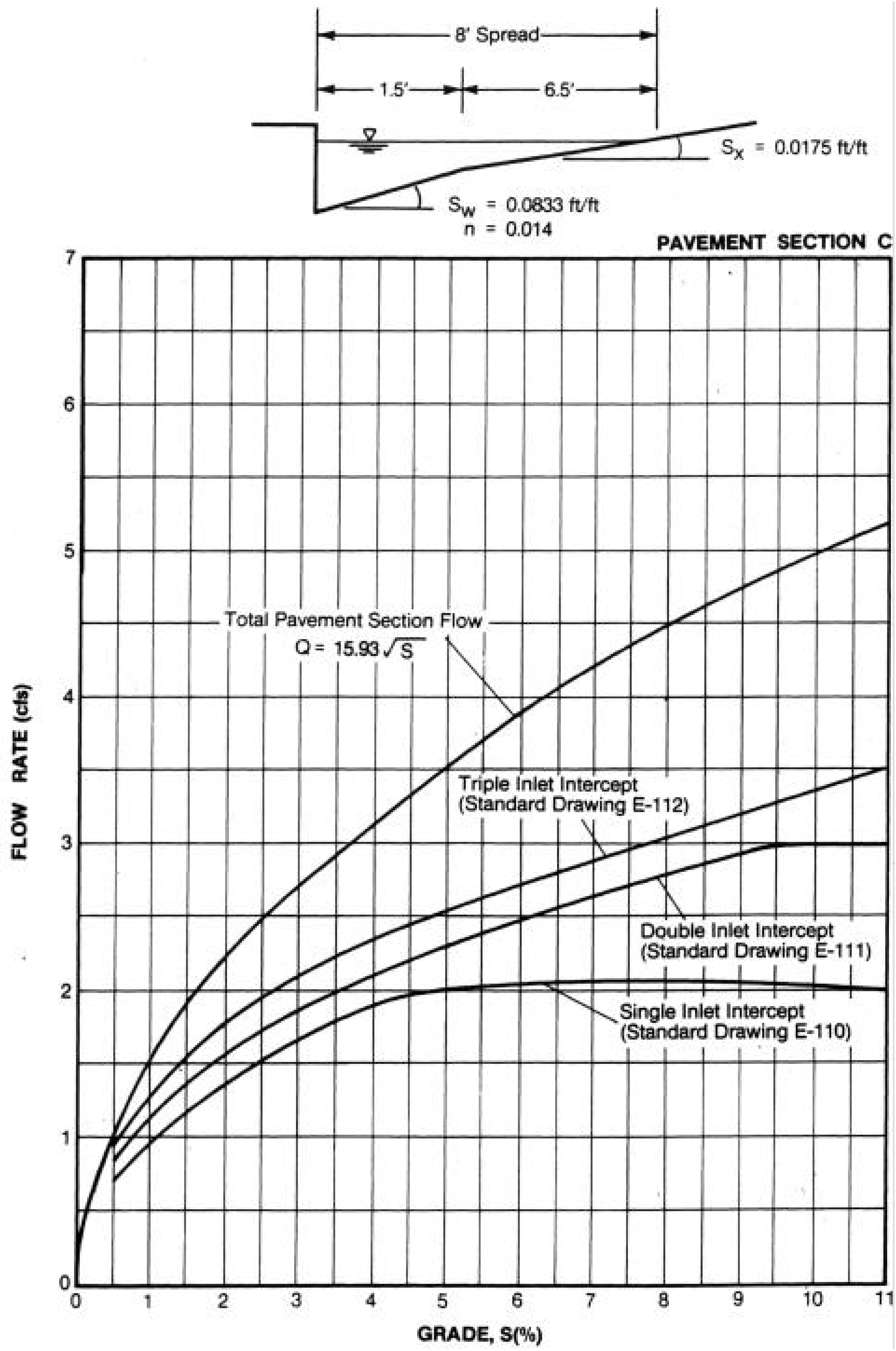
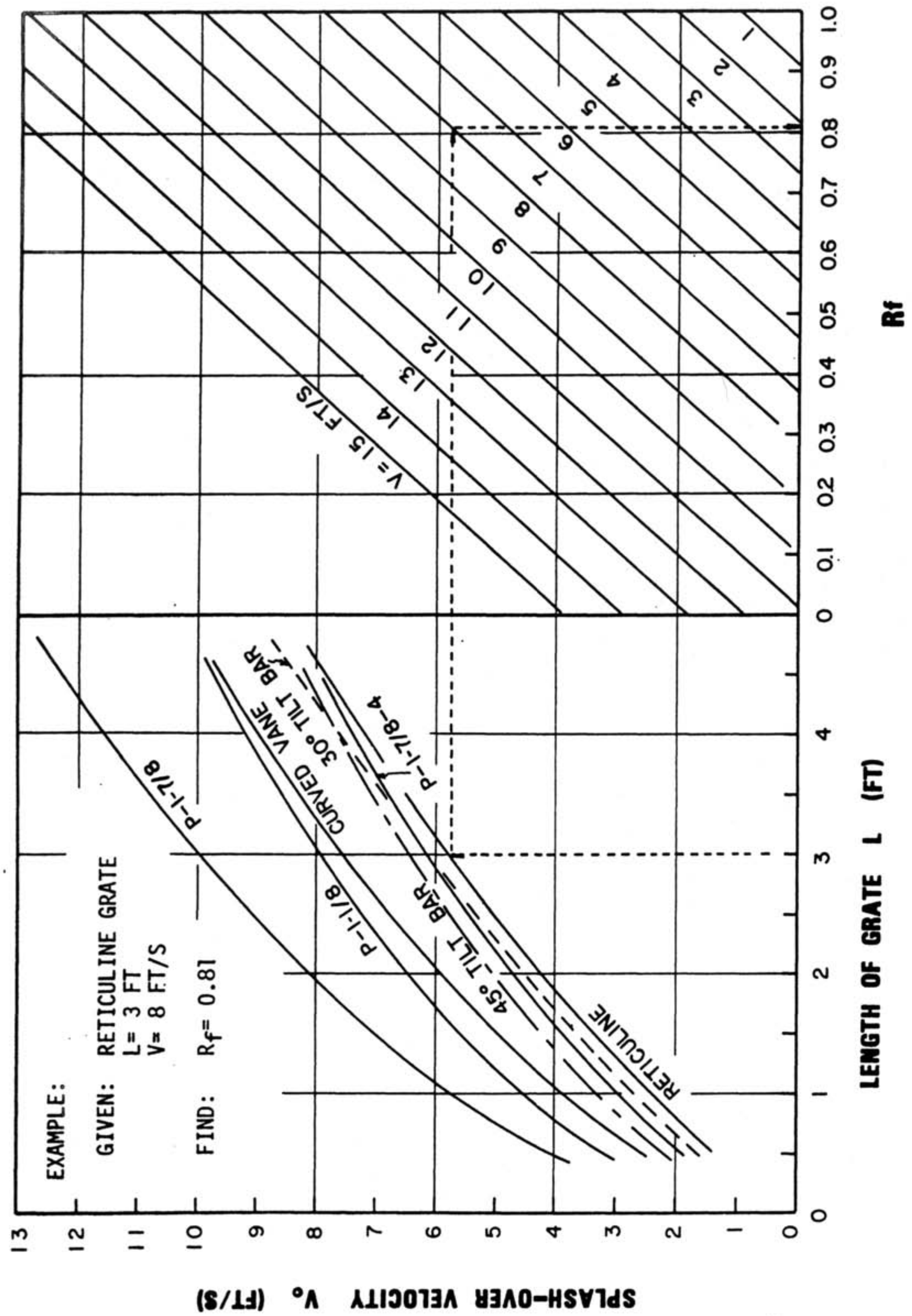
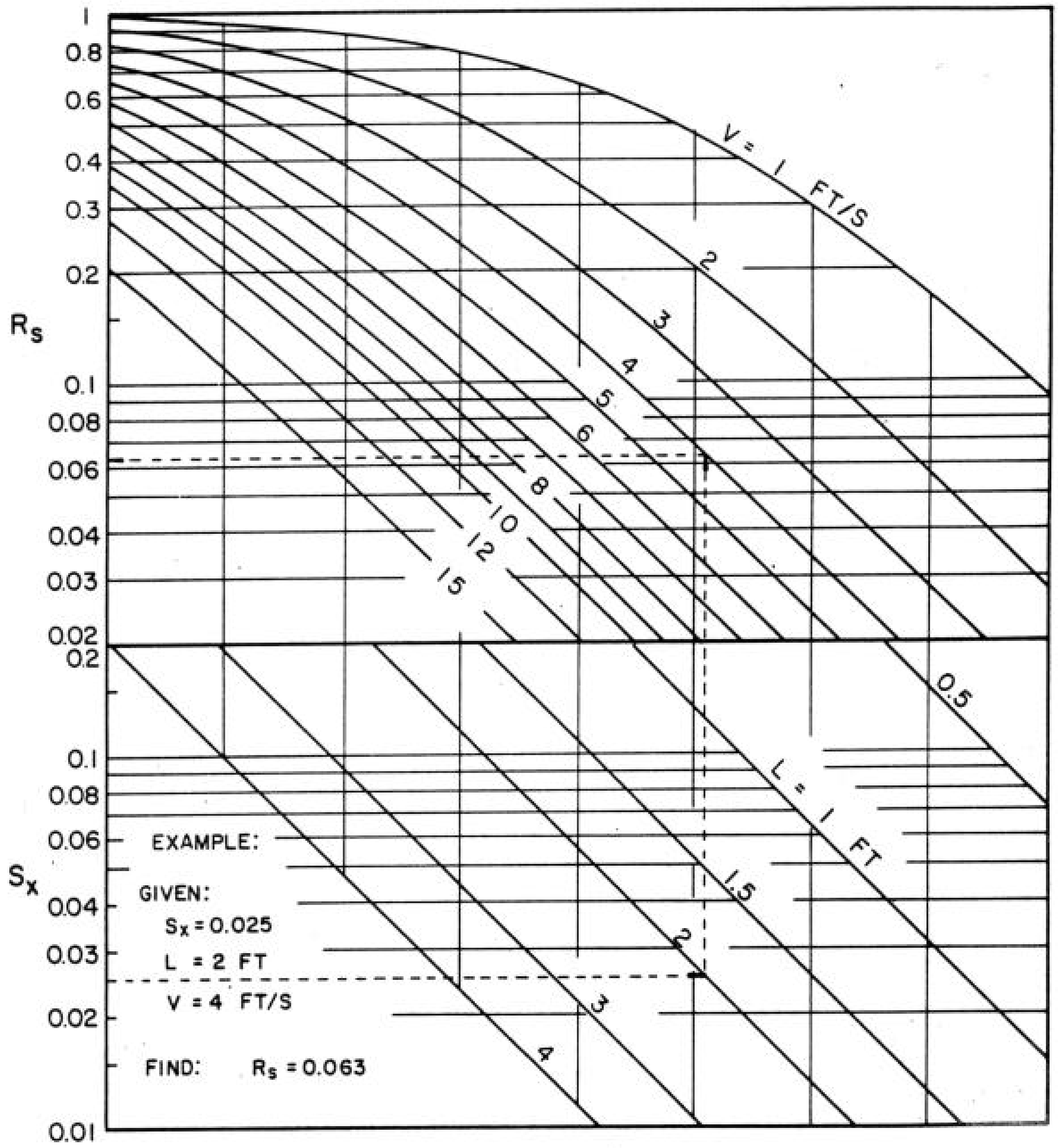


Figure 4-5
Pavement Section Flow and Intercept for 38-foot Pavement
with 24-inch Curb and Gutter (18-inch Gutter) and 8-foot Spread



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 4-6
Grate Inlet Frontal Flow Interception Efficiency



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 4-7
Grate Inlet Side Flow Interception Efficiency

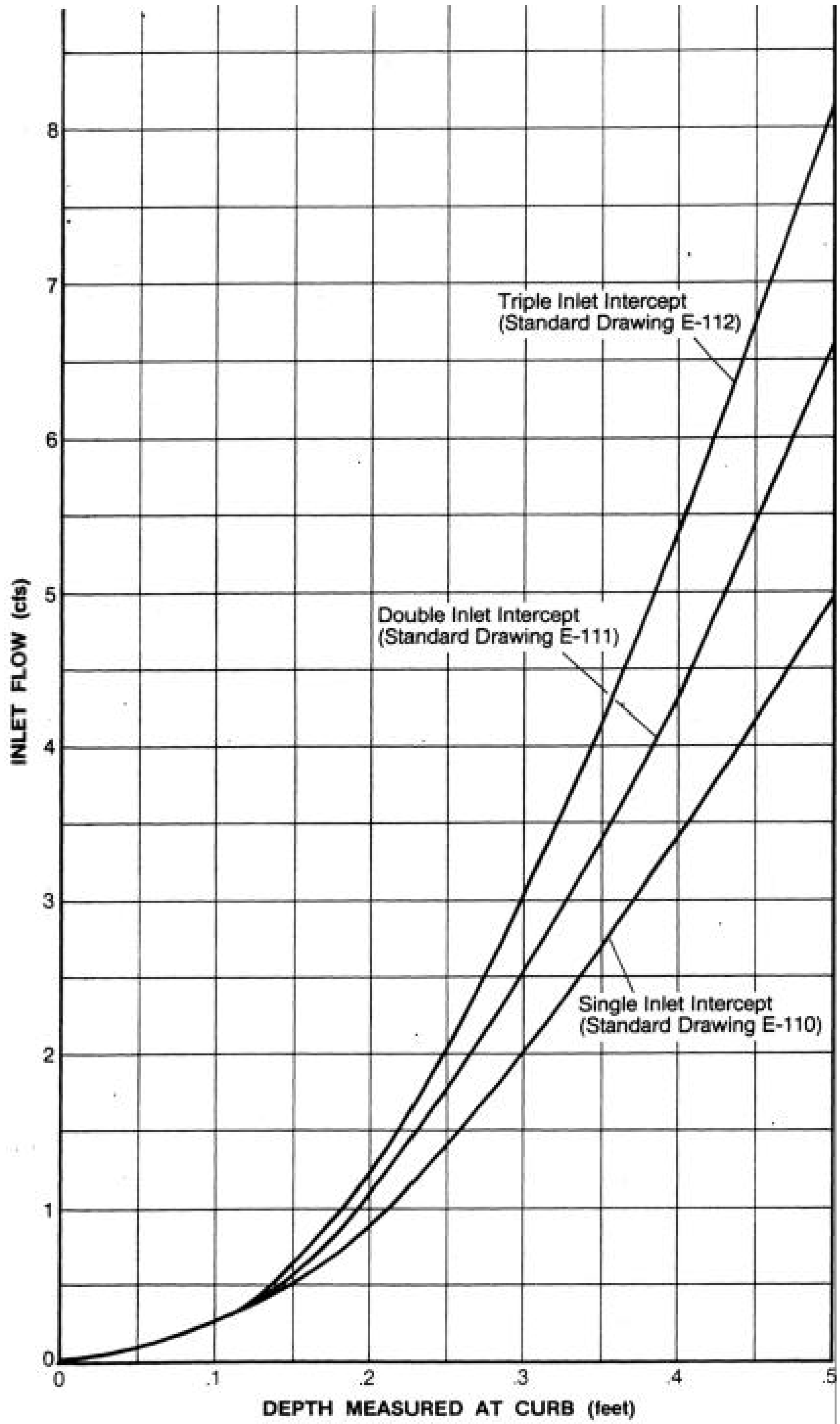
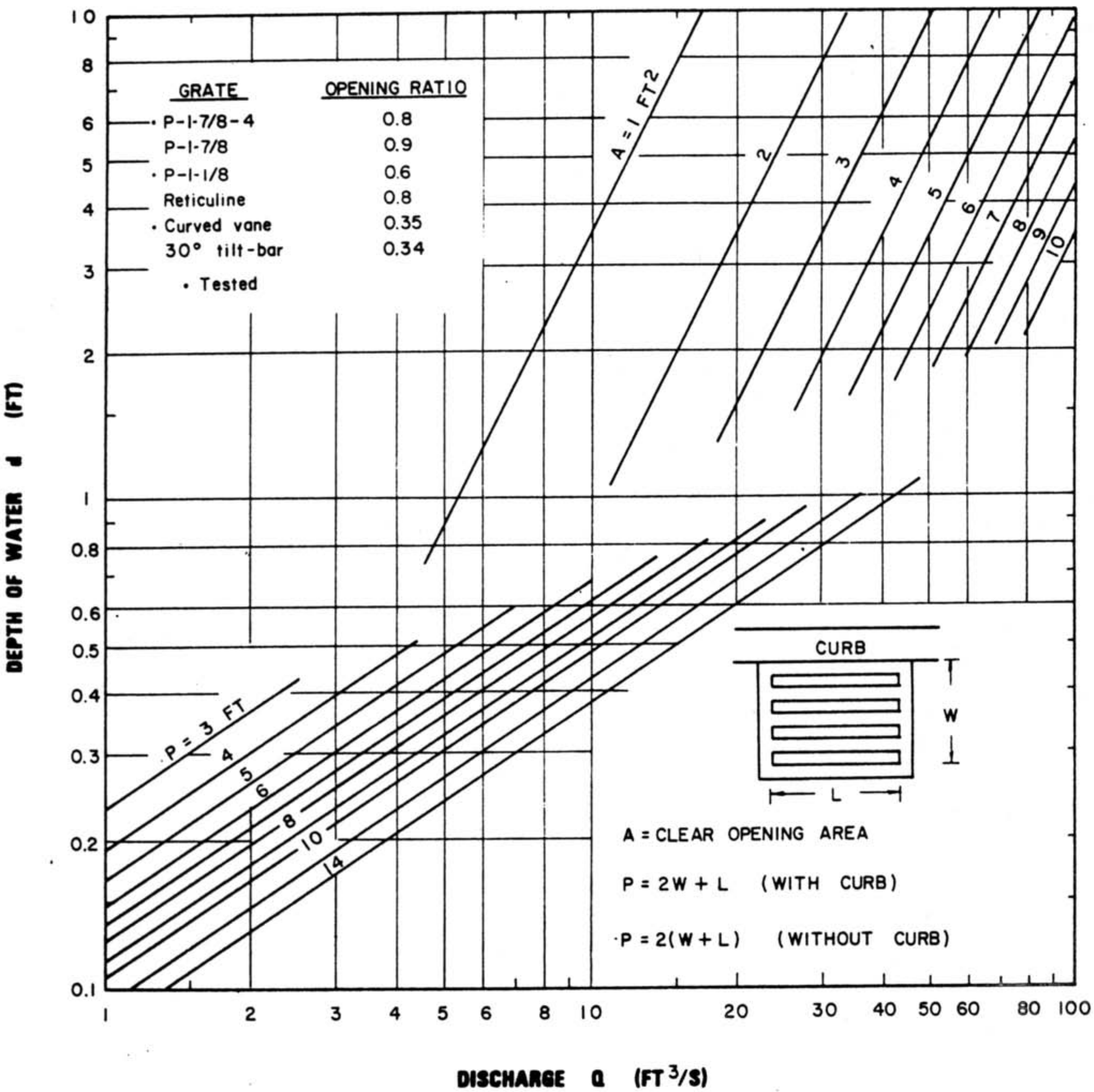


Figure 4-8
Hydraulic Capacity of Nashville Standard Grate Inlets
Under Sump Conditions



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 4-9
Grate Inlet Capacity in Sump Conditions

CHAPTER 5
CULVERT HYDRAULICS

Chapter 5
CULVERT HYDRAULICS

Synopsis

Guidelines for selecting values of design variables such as the storm return period, headwater, tailwater, length, slope, velocity limitations, and construction material are briefly discussed. Nomographs are presented for performing culvert capacity calculations. The primary reference for information presented in this chapter is HDS-5 (USDOT, FHWA, 1985). Terminology is discussed in Volume 3.

5.1 Culvert Selection

The following factors should be considered when selecting a culvert:

1. Application
2. Shape and material
3. Size
4. End treatment
5. Control section

5.1.1 Application

For purposes of consistency, culvert applications are divided into two major categories: cross drains and side drains.

<u>Cross Drain</u>

A cross drain is a culvert placed transversely under roadway sections, with endwalls or some end treatment. Because cross drain installations are normally under pavement, they should have soiltight joints as a minimum to prevent soil migration. Leaking joints can cause uneven and differential settling of road surfaces or adjacent buildings.

<u>Side Drain</u>

This culvert is generally a pipe used longitudinally in roadway ditches under driveways or graded connections. Alignment joints rather than soiltight joints are generally adequate for side drains.

5.1.2 Shape and Material

Overall costs are a major factor in the selection of a culvert shape and material; other important factors are hydraulic capacity, load carrying capacity, allowable headwater, grade controls, and aesthetic considerations.

The two materials for culverts allowed within Right of Ways (or culverts that carry public water) are concrete and corrugated metal. Concrete pipe has a lower roughness coefficient (Manning's n value), although corrugated metal with a lining is comparable. To provide lower flow resistance, a lining should be expected to last for the service life of the culvert. Corregated metal pipe (CMP) systems should be Aluminized Steel - Type 2, unless they are open-bottom culverts. All reinforced concrete pipes (RCP) with inverts less than 18 feet shall be Class III. Culverts are to be designed with upstream and downstream headwalls.

When the vertical distance from invert to roadway is limited, arch or elliptical culverts may be appropriate. When the rise of a culvert exceeds 4 feet, box culverts will generally offer cost advantages. Multiple pipe culverts are discouraged by MWS when other alternatives are available.

5.1.3 Size

Culverts should be sized using the nomographs presented in this chapter. Size reductions may be warranted after an evaluation of the effects of ponding and temporary storage. The minimum culvert pipe size shall be 15 inches.

5.1.4 End Treatment

The selection of end treatment facilities must be consistent with hydraulic requirements and give proper consideration to bank stability, safety, and costs. Entrance loss coefficients (k_e) for the standard inlet configurations are summarized in Table 5-1.

5.1.5 Control Section

The two basic types of culvert control sections are inlet and outlet control. The control section for inlet control is just inside the entrance. Critical depth occurs at or near this location and flow in the culvert is supercritical. The control section for outlet control is located at the barrel exit or downstream from the culvert. Either partially full subcritical flow or full pipe pressure flow conditions can occur. (See Volume 3 for additional information on culvert fundamentals.)

If inlet control exists, the culvert barrel is capable of carrying more flow than the inlet will accept, and a tapered inlet (see Section 5.3.4) can be used to increase capacity up to the outlet capacity. If outlet control exists, the culvert barrel would have to be increased to add capacity.

5.2 Design Criteria

The following parameters should be considered when culvert hydraulic calculations are performed:

1. Return period
2. Headwater
3. Tailwater
4. Length and slope
5. Velocity limitations

5.2.1 Return Period

The appropriate design storm return periods (from Section 6.5 of Volume 1) are summarized below:

Major facilities, including residential collector and commercial crossings	100 years
Minor facilities, including residential roads and crossings	10 years

In addition to the standard design flood, the culvert capacity should be checked for the base flood (Q_{100}), to ensure that access is consistent with the roadway classification. If ponding occurs at the entrance of the culvert, a reduction in the discharge may be appropriate. A reservoir routing procedure can be used to determine the discharge reduction attributable to storage.

5.2.2 Headwater

The allowable headwater elevation is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. The following criteria should be considered:

1. Non-damaging or permissible upstream flooding elevations (e.g., existing buildings or Flood Insurance Rate Map elevations) should be identified and headwater kept below them.

2. Headwater depth for the design discharge should not exceed a height greater than 1.5 feet below the edge of the shoulder of a road. In general, the maximum allowable headwater above the crown of a culvert should not exceed 5 feet.

3. Headwater depth for the design discharge should not cause water to rise above the top of approach channels adjacent to improved land or above the established flood plain easements.

4. Level pool backwater conditions should be evaluated upstream from the culvert to ensure that flooding of buildings does not occur for the 100-year, 24-hour design storm (see Section 6.5, Volume 1).

In general, the constraint that gives the lowest allowable headwater elevation should establish the basis for hydraulic calculations.

5.2.3 Tailwater

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for the design discharge. If the culvert outlet is operating in a free fall condition (e.g., a cantilever pipe), the critical depth and equivalent hydraulic grade line should be determined using procedures from this chapter. For culverts that discharge to an open channel, the normal depth of flow in the channel must be evaluated using procedures presented in Chapter 3.

If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert. If the culvert discharges to a lake, pond, or other major waterbody, the expected high water elevation of the particular waterbody may establish the culvert tailwater.

5.2.4 Length and Slope

The length and slope of a culvert should be based on the channel bottom of the stream waterbody being conveyed, the geometry of the roadway embankment, and the skew angle of the culvert. In general, the culvert slope should be chosen to approximate existing topography.

5.2.5 Velocity Limitations

Both minimum and maximum velocities should be considered when designing a culvert. A minimum velocity of 2.5 feet per second when the culvert is flowing full is recommended to ensure a self-cleaning condition during partial depth flow. When velocities below this minimum are anticipated, the installation of a sediment trap upstream of the culvert should be considered.

The maximum velocity should be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for channel stabilization at the culvert outlet increases. If velocities exceed permissible velocities for the outlet lining material (see Chapter 3), the installation of outlet protection may be appropriate. Channel stability information is presented in Chapter 3 and outlet protection is covered in Chapter 10.

5.3 Design Calculations

A flow chart for performing culvert design calculations is provided in Figure 5-1 (see discussion in Section 5.3.1). Worksheets for performing calculations are provided in Figures 5-2, 5-3, and 5-4 for standard culvert design, tapered inlet design, and mitered inlet design, respectively. Seven inlet control, seven outlet control, and six critical depth charts from HDS-5 (USDOT, FHWA, 1985) are provided in this chapter for performing culvert capacity calculations. The following culvert capacity charts are provided:

Culvert Type	Inlet Control Fig. No.	Outlet Control Fig. No.
Concrete Pipe	5-5	5-6
Corrugated Metal Pipe (CMP)	5-7	5-8
Structural Plate CMP	5-7	5-9
Concrete Box	5-10	5-11
Oval Concrete Pipe—Long Axis Horizontal	5-12	5-14
Oval Concrete Pipe—Long Axis Vertical	5-13	5-14
CMP Arch	5-15	5-16
Structural Plate CMP (18-inch Corner Radius)	5-15	5-17
Circular Pipe With Beveled Ring	5-18	5-6 or 5-8

Additional charts for corrugated metal box, arch, and long span culverts are available from HDS-5 (USDOT, FHWA, 1985).

5.3.1 General Procedure

The following general procedure, as represented schematically by the flow chart in Figure 5-1, is recommended to select a culvert size with the charts from HDS-5 (USDOT, FHWA, 1985):

1. Perform hydrologic calculations (see Chapter 2).

2. List the following design data (see suggested tabulation form, Figure 5-2):

   a. Design discharge, Q, in cfs, with average return period (e.g., Q_{10}). When more than one barrel is used, show Q divided by the number of barrels. Note that MWS discourages the use of multiple barrel culverts when other alternatives are available.

   b. Approximate length, L, of culvert, in feet.

   c. Slope of culvert (if grade is given in percent, convert to slope in feet/foot).

d. Allowable headwater depth, AHW, in feet; i.e., the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert.

e. Mean and maximum flood velocities in natural stream (optional), in feet/second.

f. Type of culvert, including barrel material, barrel cross-sectional shape, and inlet configuration.

3. Determine a trial culvert size by choosing one of the following options:

a. Arbitrary selection

b. An approximating equation such as:

$$A = \frac{Q}{v} \qquad (5\text{-}1)$$

where:

A = Culvert area, in square feet

Q = Design discharge, in cfs

v = Average velocity, in feet/second

c. Inlet control nomographs for the culvert type selected (see Section 5.3.2). A trial size is determined by assuming HW_i/D (e.g., $HW_i/D = 1.5$) and using the given Q.

If the trial size selected is larger than available standard culvert sizes or its use is prohibited by other physical limitations (such as limited embankment height), multiple culverts may be used by dividing the discharge equally between the number of barrels. It is also possible to consider raising the embankment height or using pipe arch and box culverts with width greater than twice the height.

4. Find inlet and outlet control headwater, HW, depths for the trial culvert size as follows:

a. For inlet control, perform the following calculations:

(1) Use an appropriate inlet control chart as discussed in Section 5.3.2 and the trial size from Step 3, to find HW_i. Tailwater, TW, conditions are

neglected in this determination. HW_i is found by multiplying HW_i/D, obtained from the nomographs, by the height of culvert D.

(2) If HW_i is greater or less than allowable (see Section 5.2.2 for design criteria), try another trial size until HW_i is acceptable for inlet control, before computing HW_o, for outlet control.

(3) The headwater elevation for inlet control conditions is calculated as follows:

$$EL_{hi} = HW_i + EL_i \qquad (5\text{-}2)$$

where:

EL_{hi} = Headwater elevation for inlet control, in feet

HW_i = Headwater depth, for inlet control, in feet

EL_i = Elevation of the culvert entrance, in feet

b. For outlet control, perform the following calculations:

(1) Calculate H, the head loss across the culvert, using the proper outlet control nomograph from Figures 5-6 to 5-17 and procedures discussed in Section 5.3.3.

(2) Approximate the depth of TW, in feet, above the invert at the outlet for the design flood condition in the outlet channel (see Section 5.2.3).

(3) If the TW depth determined in Step (2) is equal to or greater than the top of the culvert at the outlet, set design tailwater, h_o, equal to TW and find the headwater depth for outlet control, HW_o, using the following equation:

$$HW_o = H + h_o - LS_o \qquad (5\text{-}3)$$

where:

HW_o = Headwater depth for outlet control, in feet

H = Total head loss for outlet control from Step (1) above, in feet

h_o = Design tailwater, in feet

L = Barrel length, in feet

S_o = Barrel slope, in feet/foot

(4) If the TW depth determined in Step (2) is less than the top of the culvert at the outlet, find the headwater depth for outlet control, HW_o , using Equation 5-3, except that h_o is the greater of the following two parameters:

$$d_e = \frac{d_c + D}{2} \qquad (5\text{-}4)$$

or

TW

where:

d_e = Equivalent hydraulic depth at outlet, in feet

d_c = Critical depth, in feet (from Figures 5-19 through 5-24). Note: d_c cannot exceed D.

D = Height of culvert opening, in feet

TW = Downstream tailwater depth, in feet

Critical depth charts from HDS-5 (USDOT, FHWA, 1985) are presented in Figures 5-19 through 5-24 for the following culvert types;

Culvert Type	Critical Depth Fig. No.
Rectangular	5-19
Circular	5-20
Oval—Long Axis Vertical	5-21
Oval—Long Axis Horizontal	5-22
Standard CMP Arch	5-23
Structural Plate CMP Arch	5-24

Note: Headwater depth determined for this condition becomes increasingly less accurate as the headwater computed by this method falls below the value:

$$HW_o \leq D + (1 + k_e)\frac{v^2}{2g} \qquad (5\text{-}5)$$

where:

HW_o = Headwater depth for outlet control, in feet

D = Height of culvert opening, in feet

k_e = Entrance loss coefficient

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

(5) The headwater elevation for outlet control is calculated as follows:

$$EL_{ho} = EL_o + H + h_o \qquad (5\text{-}6)$$

where:

EL_{ho} = Headwater elevation for outlet control, in feet

EL_o = Elevation of the culvert outlet, in feet

H = Total head loss for outlet control (see Step (1) above), in feet

h_o = Design tailwater, in feet

5. Compare the headwater values found in Step 4a (inlet control) and Step 4b (outlet control). The higher headwater governs and indicates the type of flow control under the given conditions for the trial size and inlet configuration selected.

6. If outlet control governs and the HW_o is higher than the acceptable AHW, select a larger trial size and find HW_o as instructed under Step 4b. (Inlet control does not need to be checked, since the smaller size should be satisfactory for this control as determined under Step 4a.).

7. If desired, select an alternate culvert type or shape and determine size by repeating the above steps.

8. If the culvert operates under inlet control, a tapered inlet can be designed following the procedure in Section 5.3.4.

9. If roadway overtopping occurs, capacity calculations can be performed following the procedure in Section 5.3.5.

10. If storage routing is considered important, procedures in Chapter 8 should be followed.

11. Compute outlet velocities for the culvert size and types to be considered in selection:

    a. If outlet control governs in Step 5, calculate the outlet velocity using Equation 5-1. The depth of flow at the outlet section is selected using the following criteria:

       $TW \leq d_c$, $d_o = d_c$

       $D > TW > d_c$, $d_o = TW$

       $TW > D$, $d_o = D$

       where:

       TW = Tailwater depth, in feet (see Section 5.3.3)

       d_c = Critical depth (see Figures 5-19 through 5-24), in feet

       d_o = Depth at outlet, in feet

       D = Culvert height, in feet

    b. If inlet control governs in Step 5, the depth of flow at the outlet section is assumed to be equal to the normal depth, d_n, which should be calculated using Manning's Equation (see Chapter 3).

12. Determine whether channel protection should be considered (see Chapter 3).

13. Record final selection of culvert with size, type, required headwater, outlet velocity, channel protection, and economic justification.

5.3.2 Inlet Control

Inlet control charts from HDS-5 (USDOT, FHWA, 1985) are presented in the following figures:

Culvert Type	Inlet Control Figure No.
Circular Concrete Pipe	5-5
Circular CMP	5-7
Concrete Box	5-10
Oval Concrete Pipe—Long Axis Horizontal	5-12
Oval Concrete Pipe—Long Axis Vertical	5-13
CMP Arch	5-15
Circular Pipe with Beveled Ring	5-18

The following three types of inlet calculations can be performed using these figures:

1. To determine the headwater, HW, given Q and size for selected culvert type and inlet configuration:

    a. Use a straightedge to connect the culvert diameter or height, D, scale and the discharge, Q, scale, or Q/B for box culverts. Note the point of intersection of the straightedge on the HW_i/D scale marked (1).

    b. If the HW_i/D scale marked (1) represents the inlet configuration used, read HW_i/D on this scale. When either of the other two inlet configurations listed on the nomograph is used, extend the point of intersection obtained in Step lb horizontally to scale (2) or (3) and read HW_i/D.

    c. Compute HW_i by multiplying HW_i/D by D.

    Note: Approach velocity is assumed to be zero by this procedure. If the approach velocity is considered significant, HW_i can be decreased by subtracting the velocity head.

2. To determine Q per barrel, given HW_i and size for selected culvert type and inlet configuration:

    a. Compute HW_i/D or given conditions.

    b. Locate HW_i/D on scale for appropriate inlet configuration. If scale (2) or (3) is used, extend the HW_i/D point horizontally to scale (1).

    c. Use a straightedge to connect the point on HW_i/D scale (1) obtained above with the culvert size on the far left scale. Read Q or Q/B at the intersection of this line with the middle discharge scale.

d. If Q/B is read in Step 2c, multiply by B (span of box culvert) to find Q.

3. To determine culvert size, given Q, AHW, and type of culvert with desired inlet configuration:

a. Using a trial size, compute HW_i/D.

b. Locate HW_i/D on the scale for the appropriate inlet configuration. If scale (2) or (3) is used, extend the HW_i/D point horizontally to scale (1).

c. Use a straightedge to connect the point on HW_i/D scale (1) obtained above with the given discharge on the middle scale. Read diameter, height, or size of culvert required at the intersection of this line with the culvert size scale on the far left.

d. If D is not as originally assumed, repeat procedure with a new D.

5.3.3 Outlet Control

Outlet control charts from HDS-5 (USDOT, FHWA, 1985) are presented in the following figures:

Culvert Type	Outlet Control Figure No.
Circular Concrete Pipe	5-6
Circular CMP	5-8
Structural Plate CMP	5-9
Concrete Box	5-11
Oval Concrete Pipe—Long Axis Horizontal or Vertical	5-14
CMP Arch	5-16
Structural Plate CMP Arch with 18-inch Corner Radius	5-17

The following steps outline the use of these figures:

1. To determine H for a given culvert and Q:

a. Locate the appropriate nomograph for the type of culvert selected. Find k_e for the inlet configuration using data from Table 5-1.

b. Begin nomograph solution by locating the proper starting point on the length scale:

(1) If the n value of the nomograph corresponds to that of the culvert being used, select the proper length curve for an assigned k_e value and locate the starting point at the given culvert length. If a curve is not shown for the

selected k_e, see (2) below. If the n value for the culvert selected differs from that of the nomograph chart, see (3) below.

(2) For the n value of the nomograph and a k_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two scales in proportion to the k_e value.

(3) For a different roughness coefficient, n_1, than that of the chart n, use the length scales shown with an adjusted length, L_1, calculated as:

$$L_1 = L\left(\frac{n_1}{n}\right)^2 \qquad (5\text{-}7)$$

(See Step 2 for n values)

c. Use a straightedge to connect the point on the length scale to the size of the culvert barrel and mark the point of crossing on the turning line. See Step 3 for size considerations for a rectangular box culvert.

d. Pivot the straightedge on this point on the turning line and connect with the given discharge rate. For multiple barrels, divide Q by the number of barrels before using the nomograph. Note that MWS discourages the use of multiple barrel culverts when other alternatives are available. Read head in feet on the H scale located on the far right. For values beyond the limit of the printed scales, find H by solving the equation:

$$H = \left[1 + k_e + \frac{29n^2L}{R^{1.33}}\right]\frac{v^2}{2g} \qquad (5\text{-}8)$$

where:

H = Total head loss, or the elevation difference between HW_o and h_o, in feet

HW_o = Headwater depth for outlet control, in feet

h_o = Design tailwater, in feet (see Section 5.3.1, Step 4)

k_e = Entrance loss coefficient (see Table 5-1)

n = Manning's roughness coefficient

L = Barrel length, in feet

R = Hydraulic radius of the culvert, in feet

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

2. Values of n, which are the basis for the nomographs from HDS-5 (USDOT, FHWA, 1985), are presented on each nomograph.

3. To use the box culvert nomograph (Figure 5-11) for full flow for other than the configurations shown:

   a. Compute cross-sectional area of the rectangular box.

   b. Use a straightedge to connect the proper point (see Step 1) on the length scale to the barrel area and mark the point on the turning line. Note that the area scale on the nomograph is calculated for barrel cross sections with span B twice the height D; its close correspondence with the area of square boxes ensures that it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions, use Equation 5-8 for more accurate results.

5.3.4 Tapered Inlets

Tapered inlet nomographs from HDS-5 (USDOT, FHWA, 1985) are presented in the following figures:

Culvert Type	Tapered Inlet Figure No.
Throat Control—Side-Tapered Pipes	5-25
Face Control—Side-Tapered Pipes	5-26
Throat Control—Tapered Box	5-27
Face Control—Side-Tapered Box	5-28
Face Control—Slope-Tapered Box	5-29

Detail drawings for typical tapered inlet configurations are presented in Figures 5-30 through 5-35.

The following steps outline the design process for culverts with tapered inlets. Steps 1 and 2 are the same for all culverts, with and without tapered inlets.

1. Estimate the culvert barrel size to begin calculations (see Step 3 of Section 5.3.1).

2. Complete the culvert design form in Figure 5-2. These calculations yield the required fall at the culvert entrance. For the inlet control calculations, the appropriate inlet control nomograph is used for the tapered inlet throat. The required fall is upstream of the inlet face section for side-tapered inlets and is between the face section and throat section for slope-tapered inlets. The culvert design form should be completed for all barrels of interest. Plot outlet control performance curves for the barrels of interest and inlet control performance curves for the faces of culverts with nonenlarged inlets and for the throats of tapered inlets.

3. Use the tapered inlet design form in Figure 5-3 for selecting the type of tapered inlet to be used and determining its dimensions. If a slope-tapered inlet with mitered face is selected, use the special design form shown in Figure 5-4. A separate form is provided for the mitered inlet because of its dimensional complexity.

   To use the tapered inlet design form (Figure 5-3) or the design form for a slope-tapered inlet with mitered face (Figure 5-4), perform the following steps:

   a. Fill in the required design data on the top of the form:

      (1) Flow, Q, is the selected design flow rate from the culvert design form, Figure 5-2.

      (2) EL_{hi} is the inlet control headwater elevation.

      (3) The elevation of the throat invert, EL_t, is the inlet invert elevation, EL_i, from Figure 5-2.

      (4) The elevation of the streambed at the face, EL_{sf}, the stream slope, S_o, and the slope of the barrel, S, are from Figure 5-2. (For the slope-tapered inlet with mitered face, estimate the elevation of the streambed at the crest. This point is located upstream of the face section.)

      (5) The fall is the difference between the streambed elevation at the face and the throat invert elevation.

      (6) Select a side taper between 4:1 and 6:1. For slope-tapered inlets, select a fall slope, S_f, between 2:1 and 3:1. The side taper may be modified during the calculations.

(7) Enter the barrel shape and material, the size, and the inlet edge configuration from Figure 5-2. Note that for tapered inlets, the inlet edge configuration is designated the "tapered inlet throat."

b. Calculate the face width:

(1) Enter the flow rate, the inlet control headwater elevation, EL_{hi}, and the throat invert elevation on the design forms. (For the slope-tapered inlet with mitered face, the face section is downstream of the crest. Calculate the vertical difference between the streambed at the crest and the face invert, y. y includes part of the total inlet fall.)

(2) Perform the calculations resulting in the face width, B. Face control design nomographs are presented in Figures 5-26, 5-28, and 5-29.

c. Calculate tapered inlet dimensions. If the fall is less than D/4 (D/2 for a slope-tapered inlet with a mitered face), a side-tapered inlet must be used. Otherwise, either a side-tapered inlet with a depression upstream of the face or a slope-tapered inlet may be used.

(1) For a slope-tapered inlet with a vertical face, calculate L_2, L_3, and the taper. (For the slope-tapered inlet with a mitered face, calculate the horizontal distance between the crest and the face section invert L_4. These dimensions are shown on the small sketches in the top center of the forms.)

(2) Calculate the overall tapered inlet length, L_1.

(3) For a side-tapered inlet, check that the fall between the face section and the throat section is 1 foot or less. If not, return to Step 3b with a revised face invert elevation.

d. For a side-tapered inlet with fall upstream of the face or for a slope-tapered inlet with a mitered face, calculate the minimum crest width and check it against the proposed crest width. To obtain the necessary crest length for a depressed side-tapered inlet, it may be necessary to increase the flare angle of the wingwalls for the type of depression shown in Figure 5-31, or to increase the length of crest on the sump for the design shown in Figure 5-32. For a slope-tapered inlet with a mitered face, reduce the taper to increase crest width. Note that the taper must be greater than 4:1.

e. Using a sketch based on the derived dimensions and a sketch of the roadway section to the same scale, check that the culvert design fits the site. Adjust inlet dimensions as necessary, but do not reduce them below the minimum dimensions on the design form.

f. Using additional flow rate values and the appropriate nomographs, calculate a performance curve for the selected face section. Do not adjust inlet dimensions at this step in the design process. Plot the face control performance curve on the same sheet as the throat control and the outlet control performance curves.

g. If the design is satisfactory, enter the dimensions at the lower right of the design form. Otherwise, calculate another alternative design by returning to Step 3a.

4. The following dimensional limitations must be observed when designing tapered inlets using the design charts:

a. Side-tapered inlets:

(1) 4:1 $\leq$ taper $\leq$ 6:1.

(2) Wingwall flare angle range from 15 to 26 degrees with top edge beveled or from 26 to 90 degrees with or without bevels (Figure 5-35).

(3) If a fall is used upstream of the face, extend the barrel invert slope upstream from the face a distance of D/2 before sloping upward more steeply. The maximum vertical slope of the apron is 2 (horizontal) : 1 (vertical) .

(4) $D \leq E \leq 1.1D$.

b. Slope-tapered inlets:

(1) 4:1 $\leq$ taper $\leq$ 6:1.

Tapers greater than 6:1 may be used, but performance will be underestimated.

(2) $3:1 \geq S_f \geq 2:1$.

If $S_f > 3:1$, use side-tapered design.

(3) Minimum L_3 = O.5B.

(4) $D/4 \leq \text{fall} \leq 1.5D$.

i. For fall < D/4, use side-tapered design.

ii. For fall < D/2, do not use the slope-tapered inlet with mitered face.

iii. For fall > 1.5D, estimate friction losses between the face and the throat by using the following equation and adding the additional losses to HW:

$$H_1 = \left(\frac{29n^2L_1}{R^{1.33}}\right)\frac{Q^2}{2gA^2} \qquad (5\text{-}9)$$

where:

H_1 = Friction head loss in the tapered inlet, in feet

n = Manning's n for the tapered inlet material

L_1 = Length of the tapered inlet, in feet

R = Average hydraulic radius of the tapered inlet = $(A_f + A_t)/(P_f + P_t)$, in feet

Q = Flow rate, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

A = Average cross-sectional area of the tapered inlet = $(A_f + A_t)/2$, in square feet

(5) Wingwall flare angles range from 15 to 26 degrees with top edge beveled or from 26 to 90 degrees with or without bevels (Figure 5-35).

5. This section supplements the general design procedures described above with information specifically related to rectangular box culverts. The design charts for throat and face control for tapered box inlets are Figures 5-27, 5-28, and 5-29. There is a single throat control nomograph (Figure 5-27) for side- or slope-tapered rectangular inlets.

a. For determining the required face width, Figure 5-28 is used for side-tapered inlets and Figure 5-29 is used for slope-tapered inlets. Each nomograph has two scales, and each scale refers to a specific inlet edge condition. The edge conditions are depicted in Figure 5-35. Both the inlet edge condition and the wingwall flare angle affect the performance of the face section for box culverts.

Scale 1 on the design nomographs refers to the less favorable edge conditions, defined as follows:

(1) Wingwall flares of 15 to 26 degrees and a 1:1 top edge bevel, or

(2) Wingwall flares of 26 to 90 degrees and square edges (no bevels) . A 90-degree wingwall flare is a straight headwall.

Scale 2 applies to the more favorable edge conditions, defined as follows:

(1) Wingwall flares of 26 to 45 degrees with 1:1 top edge bevel, or

(2) Wingwall flares of 45 to 90 degrees with a 1:1 bevel on the side and top edges.

Note that undesirable design features, such as wingwall flare angles less than 15 degrees or 26 degrees without a top bevel, are not covered by the charts. Although the large 33.7-degree bevels can be used, the smaller 45-degree bevels are preferred because of structural considerations.

b. When designing side- or slope-tapered inlets for box culverts with double barrels, the required face width is the total clear width of the face. The thickness of the center wall must be added to this clear width to obtain the total face width. Design procedures for tapered inlets on box culverts with more than two barrels are not available.

5.3.5 Roadway Overtopping

The overall performance curve for roadway overtopping can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated (see Sections 5.3.2 and 5.3.3).

2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and the following equation to calculate flow rates across the roadway:

$$Q_o = C_d\ L\ HW_r^{1.5} \qquad (5\text{-}10)$$

where:

Q_o = Overtopping flow rate, in cfs

$C_d = K_t C_r$ = Discharge coefficient

K_t = Submergence factor (from Figure 5-36)

C_r = Unsubmerged discharge coefficient (from Figure 5-36)

L = Length of roadway crest, in feet

HW_r = Upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown, in feet

4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

5.3.6 Example Problems

The following example problems demonstrate the procedure required to design culverts as presented in Section 5.3.1 and shown schematically in Figure 5-1. The example problems use the design forms shown in Figures 5-2 and 5-3, as well as appropriate nomographs.

<u>Example 5-1. Circular Pipe Culvert</u>

A culvert at a new roadway crossing must be designed to pass the 10-year flood. Hydrologic analysis indicates a peak flow rate of 200 cfs. The following information is known:

Invert elevation at culvert inlet = 100 feet

Natural streambed slope = 1% = 0.01 foot/foot

Tailwater for 10-year flood = 3.5 feet

Approximate culvert length = 200 feet

Roadway shoulder elevation = 110 feet

Required freeboard at shoulder = 2 feet

No fall at inlet face

Design a circular pipe culvert for this site. Consider a corrugated metal pipe with standard 2-2/3- by 1/2-inch corrugations with beveled concrete inlet edges. Also consider a concrete pipe with a groove end and 45-degree headwalls.

1. Record the known data on a copy of the culvert design form shown in Figure 5-2. The completed design form for this example problem is shown in Figure 5-37. The design headwater elevation is 108 feet (shoulder elevation minus the required freeboard). The outlet elevation is 98 feet (inlet elevation minus the length of the culvert times the bed slope). Other terms were given above.

2. Make the first trial calculation by assuming a circular corrugated metal pipe with a diameter of 72 inches. Because there is only one culvert barrel, the design flow rate of 200 cfs is written in the columns titled "Total Flow" and "Flow per Barrel."

3. <u>Inlet Control Calculations</u>: Find HW_i, the headwater that will occur if the inlet controls flow, using the nomograph in Figure 5-18. Multiply the diameter times the HW_i /D value to find the inlet headwater depth, HW_i, which is added to the inlet elevation to obtain EL_{hi}, the headwater elevation under inlet control conditions.

4. <u>Outlet Control Calculations</u>: The tailwater value was given as 3.5 feet. The critical depth, d_c, is obtained from Figure 5-20 for circular pipes. The equivalent hydraulic depth at the outlet, d_e, is calculated using Equation 5-4. The design tailwater, h_o, is determined to be the given tailwater value, TW, or d_e, whichever is greater. The entrance loss coefficient, k_e is found in Table 5-1. The head loss across the culvert, H, is found from the nomograph shown in Figure 5-8 for CMP. The headwater elevation under conditions of outlet control is obtained by adding the head loss and estimated depth to the inlet elevation:

   $EL_{ho} = EL_o + H + h_o$.

5. Compare the headwater elevations EL_{hi} (inlet control) and EL_{ho} (outlet control) to determine which is greater and which type of control exists. For this example, the first iteration capacity is based on inlet control and the headwater elevation is 105.8 feet. Since a headwater of 108 feet is allowed, a smaller pipe can be used.

6. Determine the outlet velocity according to the procedure discussed in Section 5.3.1, Step 11. Because the flow in this first iteration is controlled at the inlet, the following steps are performed:

a. Calculate the normal depth for the culvert using Manning's Equation. For circular pipes, the solution is shown graphically in Figure 3-7. The normal depth for the 6-foot diameter culvert is 4.32 feet.

b. Calculate the area of flow at the outlet section of the culvert based on the normal depth. For this iteration, the area of flow (depth = 4.32 feet) is 21.8 square feet.

c. Determine the outlet velocity from Equation 5-1: v = Q/A = 9.2 feet per second for this iteration.

7. Results of the second iteration (see Figure 5-37) show that the outlet controls with a headwater elevation of 108.6 feet. Since this is above the design headwater, the size is unacceptable. The outlet velocity was found to be 11.8 feet per second.

   The conclusion at this point is that a 72-inch CMP will be required.

8. Assuming a diameter of 5 feet, perform the concrete pipe calculations as described above. The HW_i/D ratio for inlet control is determined from Figure 5-5, and the head loss for outlet control, H, is obtained from Figure 5-6. Results indicate that the inlet controls with a headwater elevation of 106.8 feet. The outlet velocity was found to be 11.8 feet per second.

9. Because the control headwater elevation is 1.2 feet below the design headwater, a smaller diameter pipe can be considered. A 4.5-foot diameter is assumed and the computations are repeated.

10. Results for the 4.5-foot concrete pipe show that the control headwater elevation is 108 feet for inlet control. The design headwater is not exceeded so the pipe is acceptable. The outlet velocity of 15.2 feet per second is high and protective measures will be necessary (see Chapter 10).

Example 5-2. Concrete Box Culvert

A culvert at a new roadway crossing will be designed to carry the 100-year flow rate of 300 cfs. The following information is known:

Maximum design headwater = EL_{hd} = 110.0 feet, based on adjacent structures

Roadway shoulder elevation = 113.5 feet

Elevation of streambed at culvert face = EL_{sf} = 100 feet

Natural stream slope = S_o = 0.02 foot/foot

Tailwater depth = TW = 4 feet

Approximate culvert length = L = 200 feet

Design a reinforced concrete box culvert for this installation. Consider both square edges and 45-degree beveled edges in a 90-degree headwall. The inlet is to be set at the existing channel elevation (no fall).

1. Assume a box size of 6 feet by 5 feet. The total flow rate and the flow rate per foot width per barrel (only for box culverts, see equation [1] in the Technical Footnotes) are written into the first and second columns, respectively, of the form presented in Figure 5-2. Data and results for this problem are recorded in Figure 5-38.

2. Determine the HW_i/D ratio for inlet control from Figure 5-10 for box culverts. The headwater for inlet control is 7.9 feet, giving an elevation of 107.9 feet.

3. Determine the critical depth of 4.25 feet from Figure 5-19. The entrance loss coefficient, k_e, is found in Table 5-1.

4. Determine the head loss for outlet control, H, from Figure 5-11. The resulting elevation is 103.2 feet.

5. The inlet conditions will create a higher headwater elevation and control flow through the culvert at an elevation of 107.9 feet. For this trial, the normal depth is 2.37 feet, which gives a flow area of 14.2 square feet and an outlet velocity of 21.1 feet per second.

6. Because the control headwater elevation is 2.1 feet below the maximum allowable headwater, it may be possible to use a smaller culvert size. A 5-foot by 5-foot box is assumed and Steps 1 through 5 are repeated for this new size.

7. Results reported in Figure 5-38 show that a 5-foot by 5-foot box with a headwater elevation of 109.6 feet is adequate. The normal depth is 2.8 feet and the outlet velocity is 21.2 feet per second.

8. Examine the effect of incorporating 45-degree bevels into the design. Repeat the computations as before. The result is that the inlet control headwater elevation is 108.6 feet, or 1 foot below the design without bevels. The normal depth and outlet velocity do not change. Both designs are acceptable, but the bevels provide more flow capacity.

Example 5-3. Box Culvert with Roadway Overtopping

An existing 7-foot by 7-foot concrete box culvert was designed for a 100-year flood of 600 cfs and a design headwater elevation of 114 feet. Upstream development has increased the 100-year runoff to 1,000 cfs. It is expected that the roadway will be overtopped. The gravel roadway profile can be approximated as a broad-crested weir 200 feet long, 40 feet wide, and with a mean elevation of 116 feet. Other site information is given below:

Inlet invert elevation = 110 feet

Existing culvert slope = 0.05 foot/foot

Existing culvert length = 200 feet

Entrance conditions = Square edges

For flow, in cfs, of	tailwater =
400	2.6 feet
600	3.1
800	3.8
1,000	4.1
1,200	4.5

Prepare a performance curve for this installation, including any roadway overtopping up to a total flow rate of 1,000 cfs.

1. Apply the methods presented in Section 5.3.5 along with culvert capacity methods from Examples 5-1 and 5-2. Data and results for this problem are shown in Figures 5-39 and 5-40.

2. Record the culvert description and the flow rates ranging from 400 to 1,000 cfs. The culvert flows are divided by the number of culverts (N = 1) and the width of each barrel (B = 7 feet) and recorded in the second column of the culvert form (Figure 5-2).

3. Perform the design computations as for Example 5-2. Figure 5-10 is used to obtain the HW_i/D ratios for inlet control. Figure 5-19 is used to obtain the critical depths, and Figure 5-11 is used to obtain the head loss for outlet control.

4. Record the maximum elevation of the inlet control or outlet control headwater for each flow rate in the control headwater elevation column in Figure 5-39. No outlet velocity computations are needed for the performance curve, but results are provided.

5. The performance curve shown in Figure 5-40 consists of three components: the culvert flow, the weir flow, and the combined or total flow. The culvert flow curve is defined by the flow rates written in column 1 (Q) of the design form and the resulting control headwater elevations. The weir flow is the amount of flow that will overtop the roadway

and is based on the control headwater elevations. The flow rate over the roadway is calculated from the weir equation (Equation 5-10) where, for this problem, C_d, is found from Figure 5-36 and HW_r = control headwater elevation - weir elevation (116 feet). The total flow is the sum of the culvert flows and weir flows at a specific elevation.

Example 5-4. Box Culvert with Side-Tapered Inlet

The culvert analyzed in Example 5-3 is reexamined for the possible addition of a side-tapered inlet to assure the new 100-year flood of 1,000 cfs can be passed without exceeding the design headwater elevation of 114 feet. The known information is the same as given in Example 5-3. Design a side-tapered inlet that will pass the 100-year flood without exceeding the design headwater of 114 feet. The inlet will be constructed upstream of the existing culvert. The elevation of the throat invert will be 100 feet. Begin the design by assuming a face invert elevation of 101 feet. Prepare face control, throat control, and outlet control performance curves for the new inlet.

1. The method for designing tapered inlet is described in Section 5.3.4. The procedure requires the use of the culvert design form in Figure 5-2 and the tapered inlet design form in Figure 5-3. The completed forms and performance curves for this example problem are shown in Figures 5-41 and 5-42.

2. First determine if a side-tapered inlet will pass the 100-year flood of 1,000 cfs without exceeding the design headwater elevation. Complete the culvert design form for the given culvert with a side-tapered inlet. Fill in the known elevations, pipe description, and flow rates at the appropriate section. Obtain the HW_i/D ratio from Figure 5-27. Other items in the headwater calculations are the same as in Example 5-3. The head loss across the culvert, H, is found in Figure 5-11.

3. Results for the headwater computations show that the culvert with a side-tapered inlet will pass the 100-year flood within the design headwater elevation.

4. To create the performance curves, other flow rates are used to calculate the resulting inlet (throat) and outlet control headwater elevations, as reported in Figure 5-41.

5. Design the required side-tapered inlet using the tapered inlet design form in Figure 5-3. The known information is first written into the appropriate sections. The computations for the inlet table are performed as follows:

   Q is known = 1,000 cfs

   EL_{hi} is known = 114 feet

   EL throat invert is known = 100 feet

EL face invert is assumed = 101 feet

HW_f: See equation (2) below data in Figure 5-42

HW_f/E: See equation (3) below data in Figure 5-42, E = height of inlet, E = 7 feet

Q/F_f: Obtained from Figure 5-28

Min. B_f: See equation (5) below data in Figure 5-42

Selected B_f: Rounded up to nearest whole foot

L_1: See equation (11) below data in Figure 5-42

The computations result in a side-tapered inlet with a width of 12 feet and a length of 10 feet.

6. Obtain the performance curve data for face control from the tapered inlet design form in Figure 5-3 by working the calculations from Step 5 in reverse. The procedure is as follows:

    a. Choose the flow rate, Q, and divide by the designed inlet width, B_f. Place the new value in the Q/F_f column.

    b. Determine the HW_f/E value from Figure 5-28 and calculate HW_f.

    c. Find the design elevation of the face invert from the following equation:

       EL Face Invert = $(L_1)\ (S_o)$

    d. Find the headwater elevation under face control conditions by adding HW to the elevation of the face invert.

7. Create the performance curves by graphing the following parameters:

Face Control: Flow rate, Q, vs. EL_{hi} from tapered inlet design form

Throat Control Flow rate, Q, vs. EL_{hi} from culvert design form
Outlet Control Flow rate, Q, vs. EL_{ho} from culvert design form

Table 5-1
CULVERT ENTRANCE LOSS COEFFICIENTS

Type of Structure and Design of Entrance	Entrance Coefficient, k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-section conforming to fill slope[a]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe or Pipe Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-section conforming to fill slope[a]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

[a]"End section conforming to fill slope," made of either metal or concrete, is the section commonly available from manufacturers. From limited hydraulic tests, the sections are equivalent in operation to a headwall in both inlet and outlet control. End sections that incorporate a closed taper in their design have a superior hydraulic performance.

Reference: USDOT, FHWA, HDS-5 (1985).

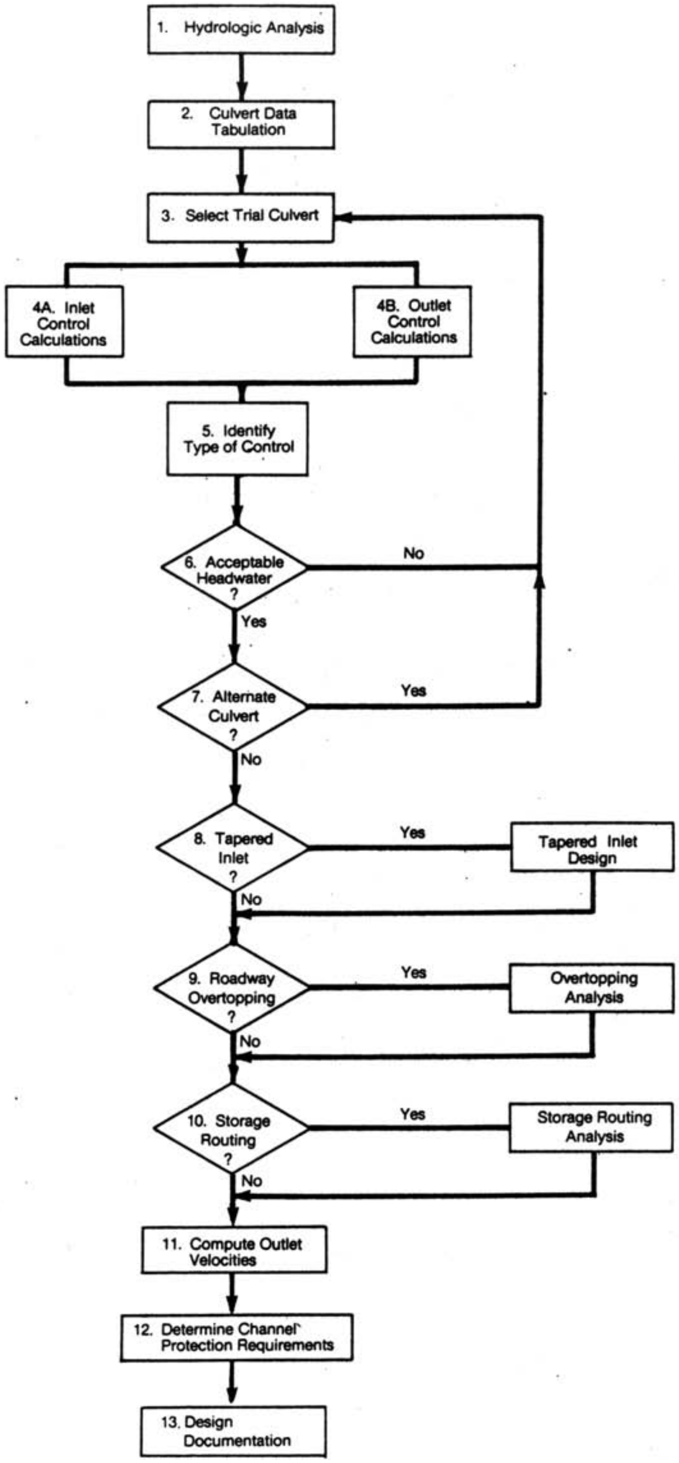


Figure 5-1
Culvert Design Flow Chart

CULVERT DESIGN FORM

PROJECT : ______________________

STATION : ______________
SHEET ______ OF ______

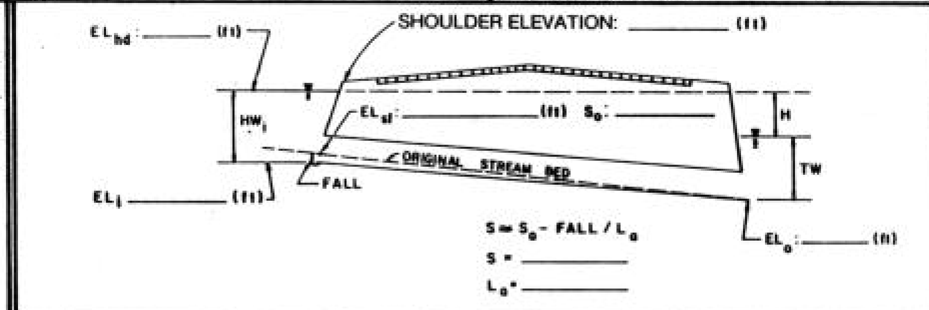
DESIGNER / DATE : ________ / ________
REVIEWER / DATE : ________ / ________

HYDROLOGICAL DATA

SEE ADD'L SHTS.
☐ METHOD : ______________
☐ DRAINAGE AREA : ________ ☐ STREAM SLOPE : ________
☐ CHANNEL SHAPE : ______________
☐ ROUTING : ________ ☐ OTHER : ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L) / R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS :

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE : ______________
SHAPE : ______________
MATERIAL : ________ n ________
ENTRANCE : ______________

Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-2
Culvert Design Form

PROJECT: ____________________

STATION: ____________________

SHEET ______ OF ______

TAPERED INLET DESIGN FORM

DESIGNER / DATE: ________ / ________

REVIEWER / DATE: ________ / ________

DESIGN DATA:

Q ______ = ______ cfs ; EL_{hi} ______ ft

EL. THROAT INVERT ______ ft

EL. STREAM BED AT FACE ______ ft

FALL ______ ft TAPER ______ : 1 (4 : 1 TO 6 : 1)

STREAM SLOPE, So = ______ ft/ft

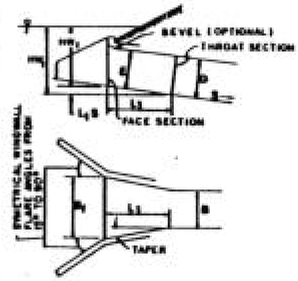
SLOPE OF BARREL, S = ______ ft/ft

S_f ______ : 1 (2:1 TO 3:1)

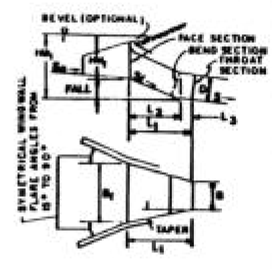
BARREL SHAPE AND MATERIAL: ______

N = ______, B = ______, D = ______

INLET EDGE DESCRIPTION ______



SIDE - TAPERED



SLOPE - TAPERED

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	$\frac{HW_f}{E}$ (3)	$\frac{Q}{B_f}$ (4)	MIN. B_f (5)	SELECTED B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)

(1) SIDE - TAPERED: EL. FACE INVERT = EL. THROAT INVERT + 1 ft (APPROX.)
SLOPE - TAPERED: EL. FACE INVERT = EL. STREAM BED AT FACE

(2) $HW_f = EL_{hi} -$ EL. FACE INVERT

(3) $1.1 D \geq E \geq D$

(4) FROM DESIGN CHARTS

(5) MIN. $B_f = Q/(Q/B_f)$

(6) MIN. $L_3 = 0.5 NB$

(7) $L_2 =$ (EL. FACE INVERT − EL. THROAT INVERT) S_f

(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_3$

(9) IF (8)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_2$

(10) IF (7)>(8), ADJ. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(11) SIDE - TAPERED: $L = \left[\frac{B_f - NB}{2}\right] \cdot TAPER$
SLOPE - TAPERED: $L_1 = L_2 + L_3$

(12) $HW_c = EL_{hi} -$ EL. CREST INVERT

(13) MIN. $W = 0.35 Q / HW_c^{1.5}$

SELECTED DESIGN

B_f ______

L_1 ______

L_2 ______

L_3 ______

BEVELS ANGLE ______°

b = ______ in ; d = ______ in

TAPER ______ : 1

S_f = ______ : 1

Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-3
Tapered Inlet Design Form

PROJECT: ____________________

STATION: __________
SHEET ______ OF ______

MITERED INLET DESIGN FORM

DESIGNER / DATE: ________ / ________
REVIEWER / DATE: ________ / ________

DESIGN DATA: N ______ ; B ______ ; D ______

Q ______ = ______ cfs ; EL_{hi} ______ ft

EL. THROAT INVERT ______ ft EL. STREAM BED AT CREST ______ ft

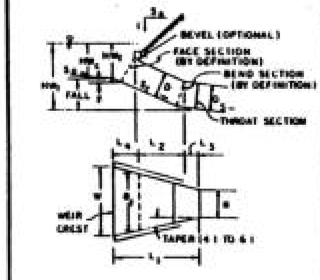
FALL ______ ft ; TAPER ______ : (4:1 TO 6:1)

STREAM SLOPE, S_o ______ ft/ft ; BARREL SLOPE, S = ______ ft/ft

SLOPE OF THE EMBANKMENT S_e = ______ : 1 ; S_f ______ : 1 (2:1 TO 3:1)

BARREL SHAPE AND MATERIAL: ____________

INLET EDGE DESCRIPTION: ____________



SLOPE-TAPERED INLET/MITERED FACE

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	y (1)	EL. FACE INVERT (2)	HW_f (3)	$\frac{HW_f}{E}$ (4)	$\frac{Q}{B_f}$ (5)	MIN. B_f (6)	SELECTED B_f	MIN. L_3 (7)	L_4 (8)	L_2 (9)	CHECK L_2 (10)	ADJ. L_3 (11)	ADJ. TAPER (12)	L_1 (13)	EL. CREST INV.	HW_c (14)	MIN. W (15)	W (16)

(1) $y = \left[\frac{(S_e \cdot S_f) - 1}{(S_e + S_f)(S_f^2 + 1)^{0.5}}\right] D$

(2) EL. FACE INVERT = EL. STREAM BED AT CREST - y

(3) $HW_f = EL_{hi}$ - EL. FACE INVERT

(4) $1.1 D \geq E \geq D$

(5) FROM DESIGN CHARTS

(6) MIN. $B_f = Q / (Q / B_f)$

(7) MIN. $L_3 = 0.5 NB$

(8) $L_4 = S_f y + D / S_f$

(9) $L_2 =$ (EL. CREST INVERT - EL. THROAT INVERT) $S_f - L_4$

*** IF L_2 IS NEGATIVE DO NOT USE THIS INLET

(10) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_3$

(11) IF (10)>(9), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(12) IF (9)>(10), ADJ. TAPER $= (L_2 + L_3) / \left[\frac{B - NB}{2}\right]$

(13) $L_1 = L_2 + L_3 + L_4$

(14) $HW_c = EL_{hi}$ - EL. CREST INVERT

(15) MIN $W = 0.35 Q / (HW_c)^{1.5}$

(16) $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

IF W<MIN. W, ADJUST TAPER

SELECTED DESIGN

B_f ______

L_1 ______

L_2 ______

L_3 ______

L_4 ______

BEVELS ANGLE ______ °

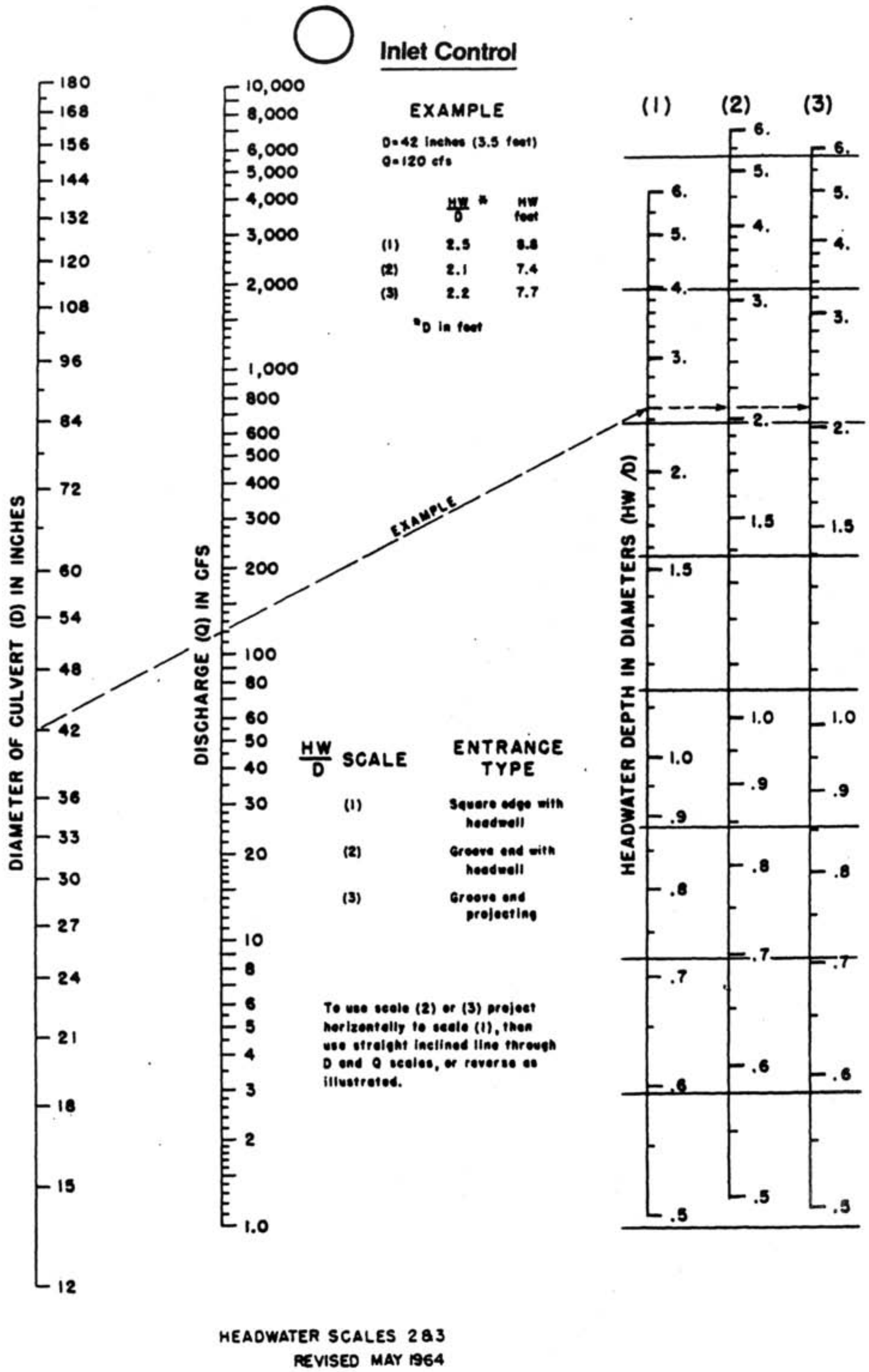
b= ______ in; d= ______ in

TAPER ______ : 1

S_f ______ : 1

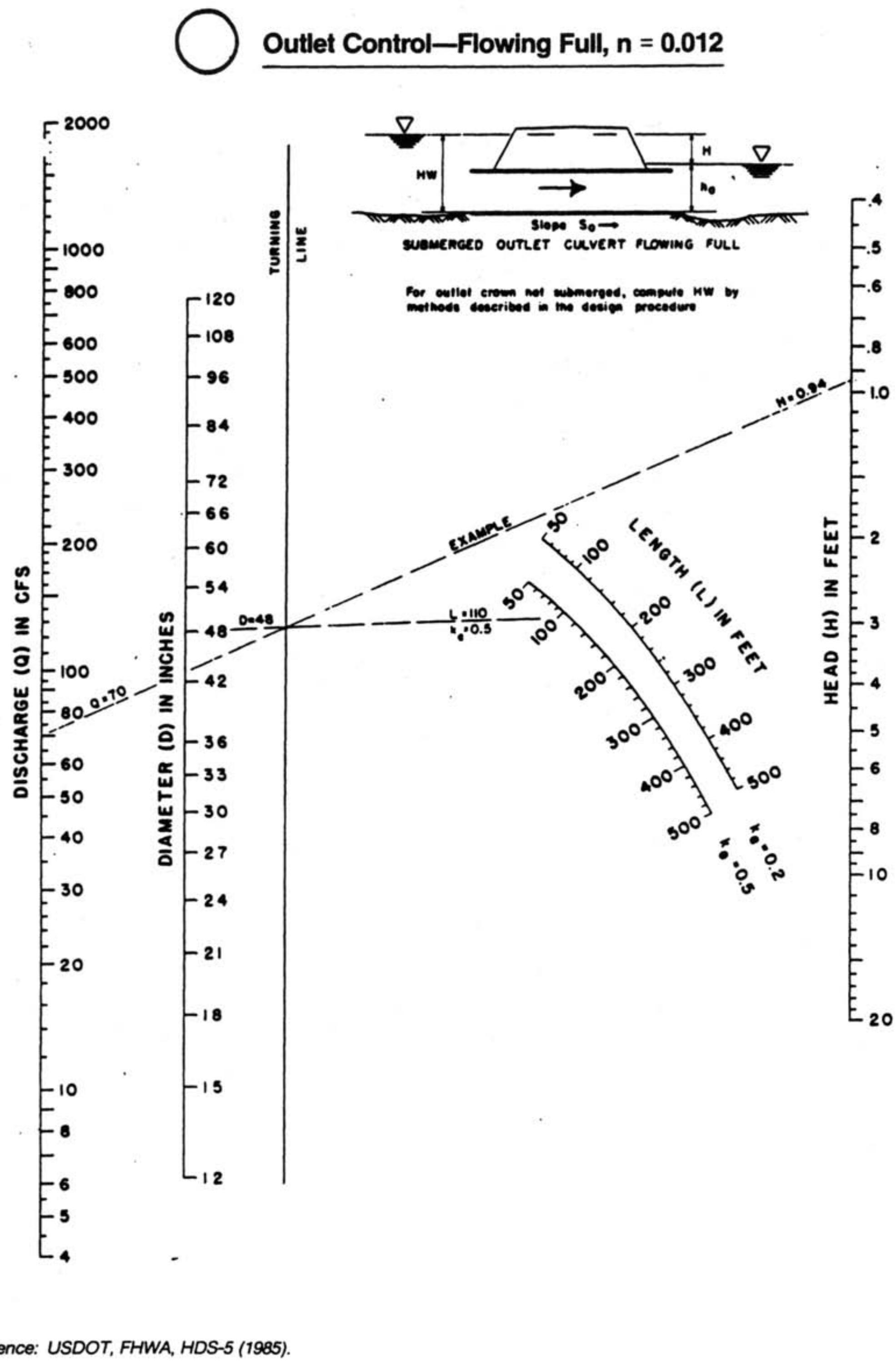
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-4
Mitered Inlet Design Form



Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-5
Inlet Control Chart for Circular Concrete Pipe Culverts



Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-6
Outlet Control Chart for Circular Concrete
Pipe Culverts Flowing Full

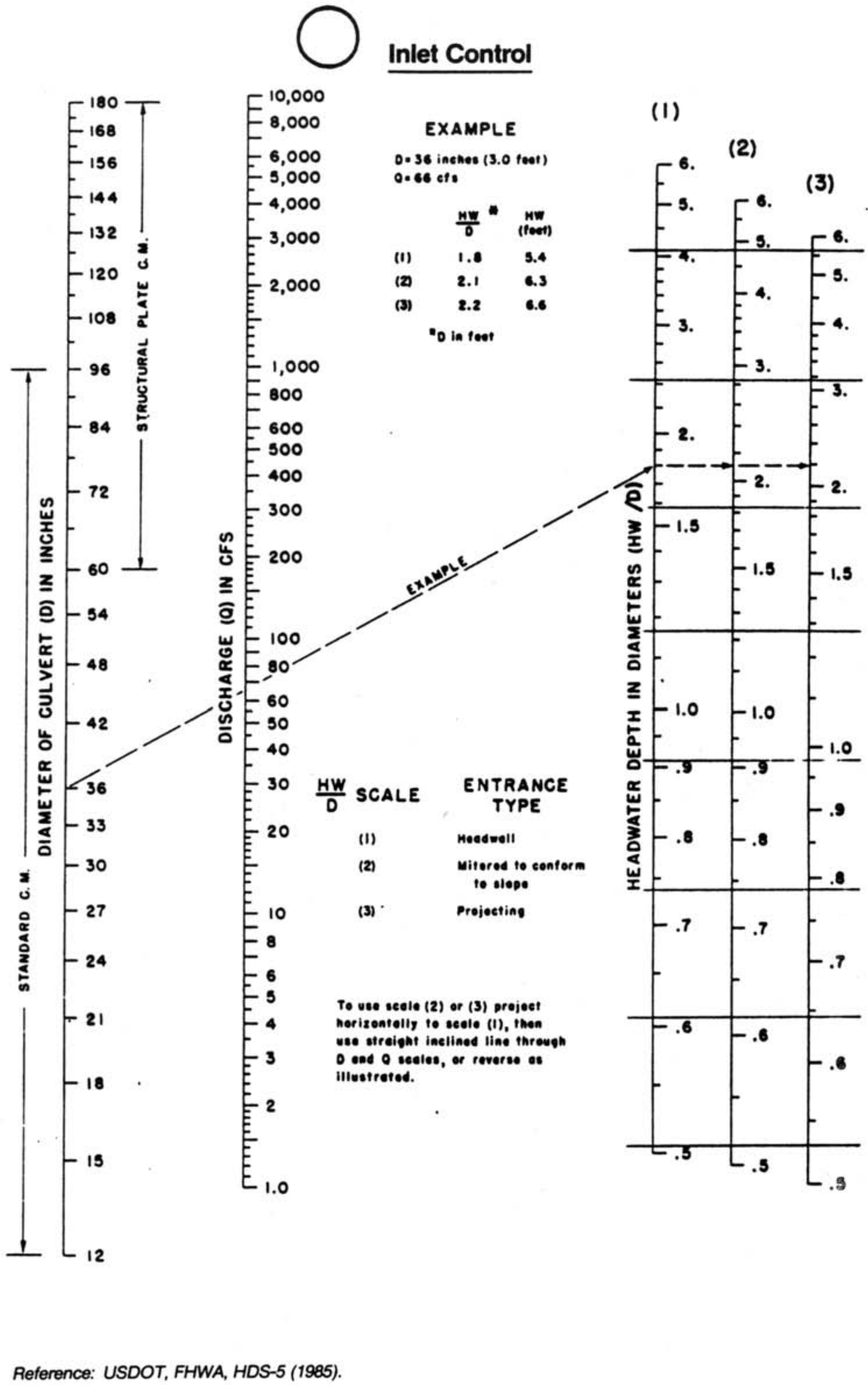
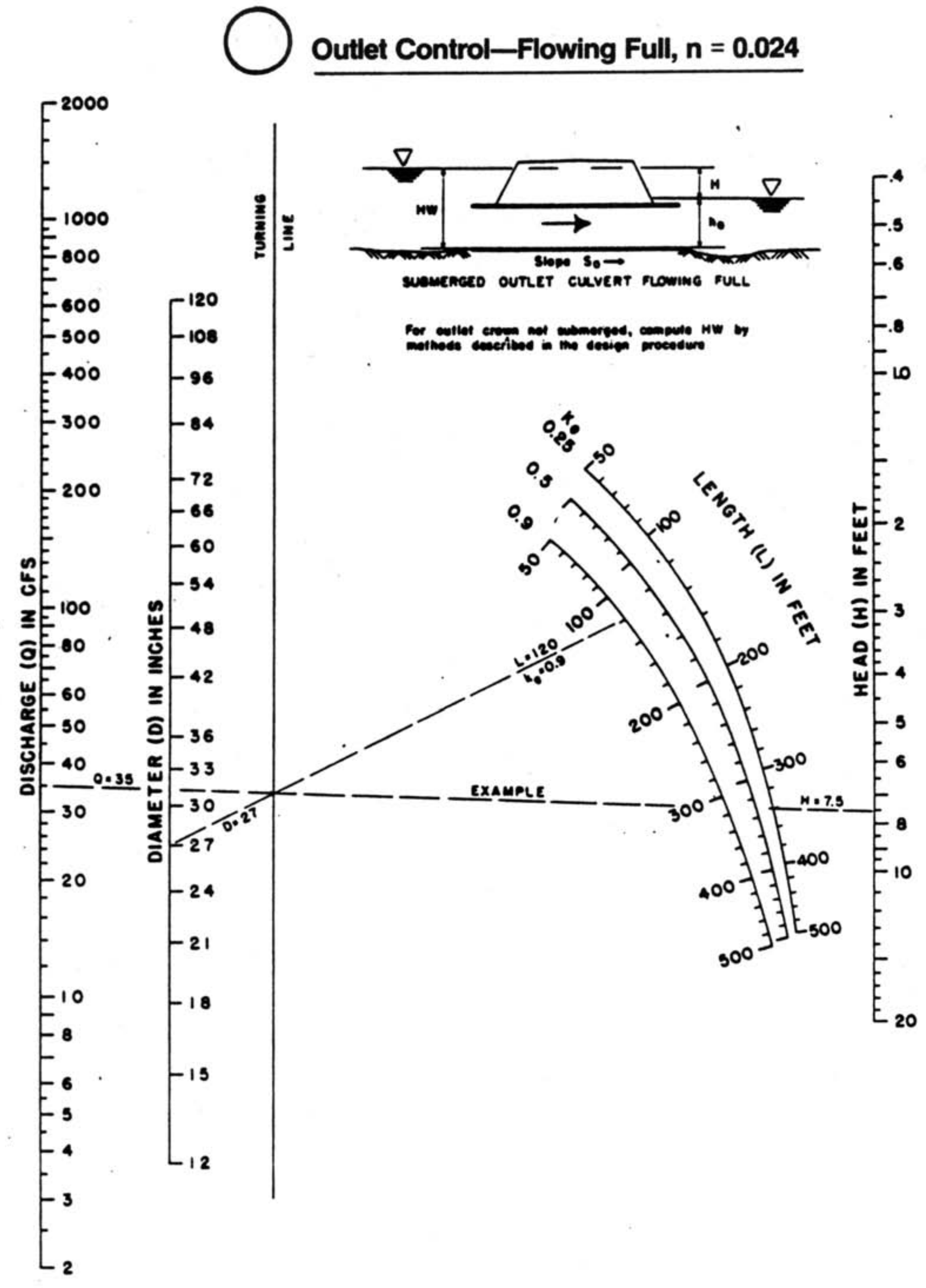
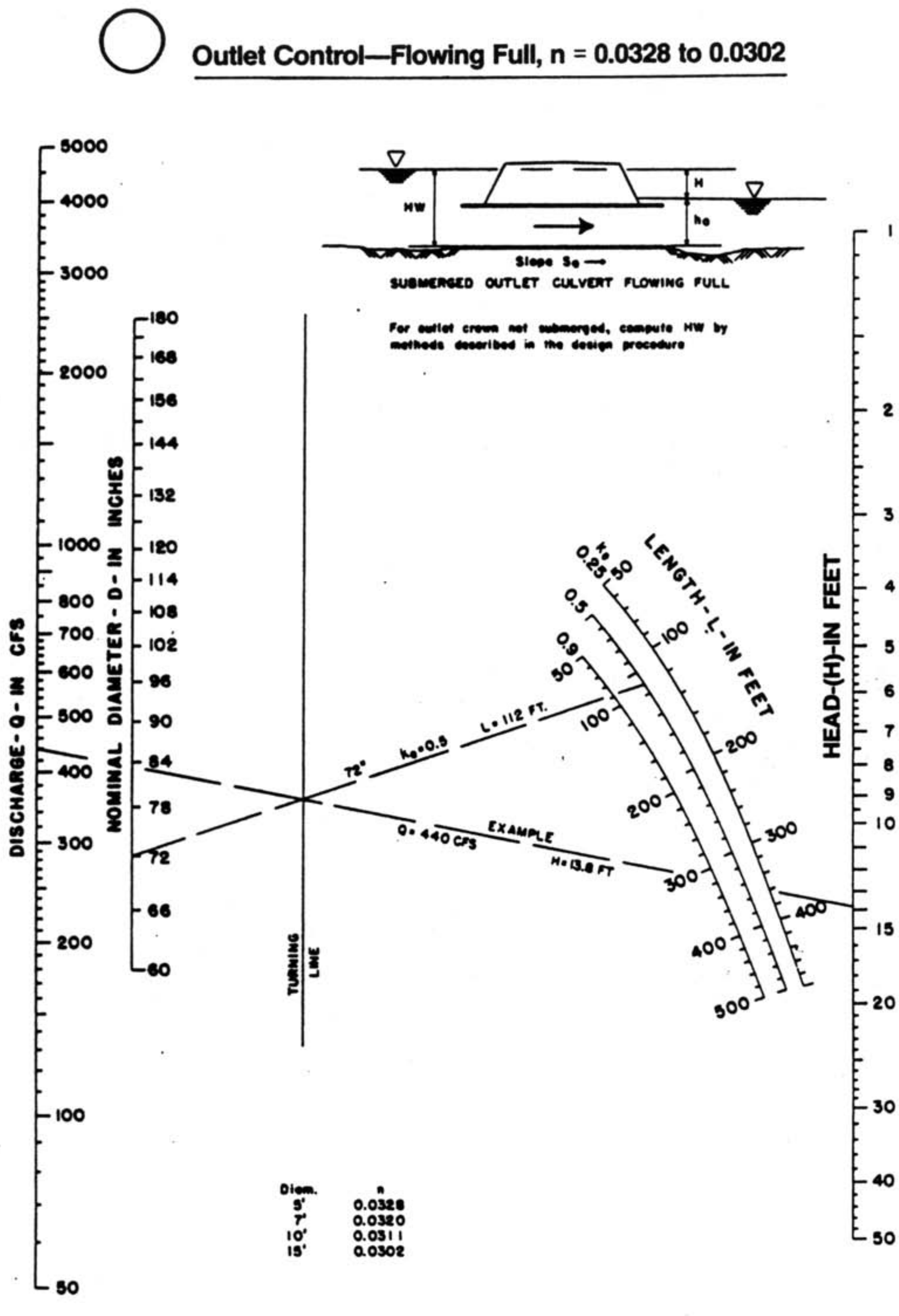


Figure 5-7
Inlet Control Chart for Circular CMP Culverts



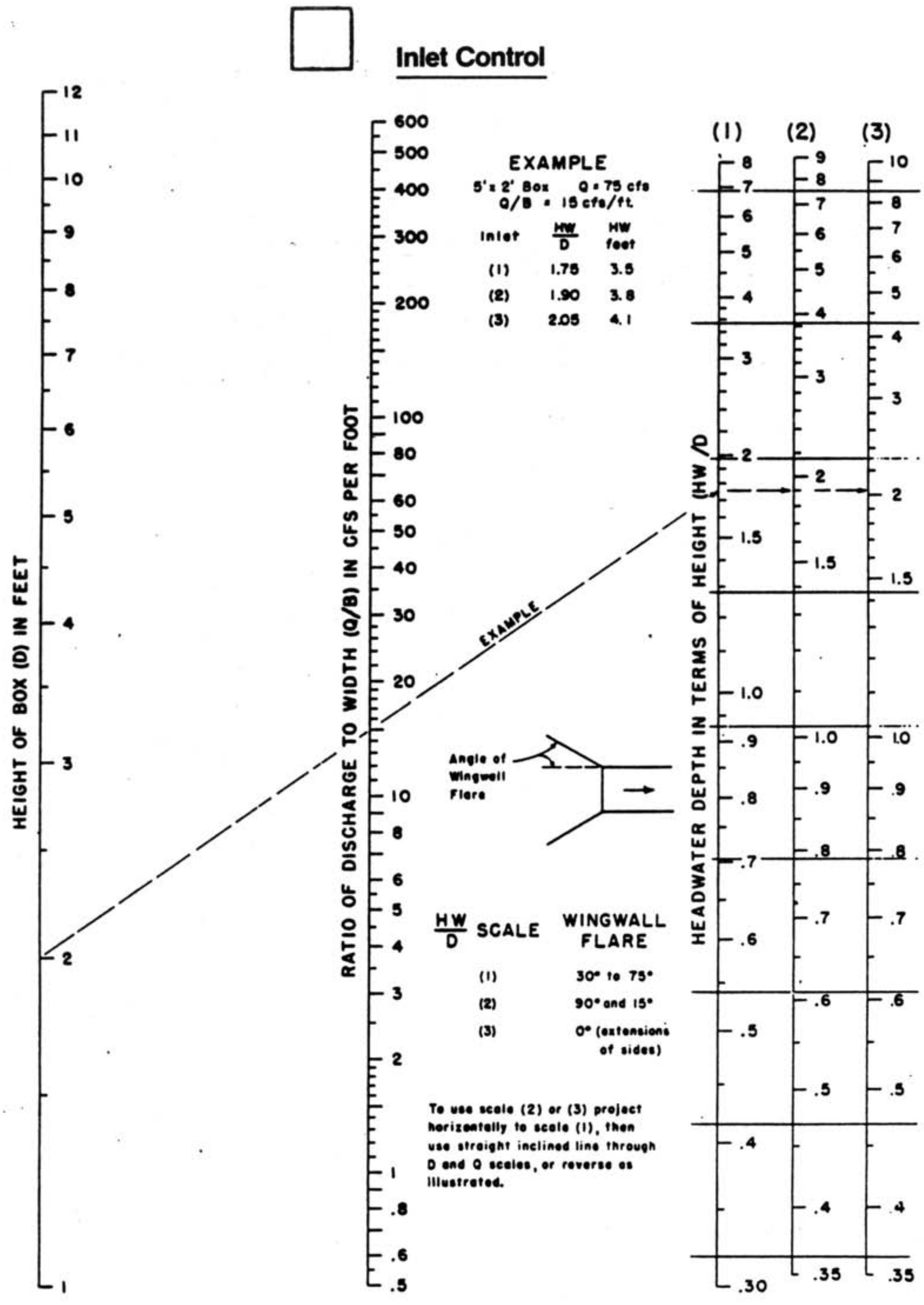
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-8
Outlet Control Chart for Circular CMP Culverts Flowing Full



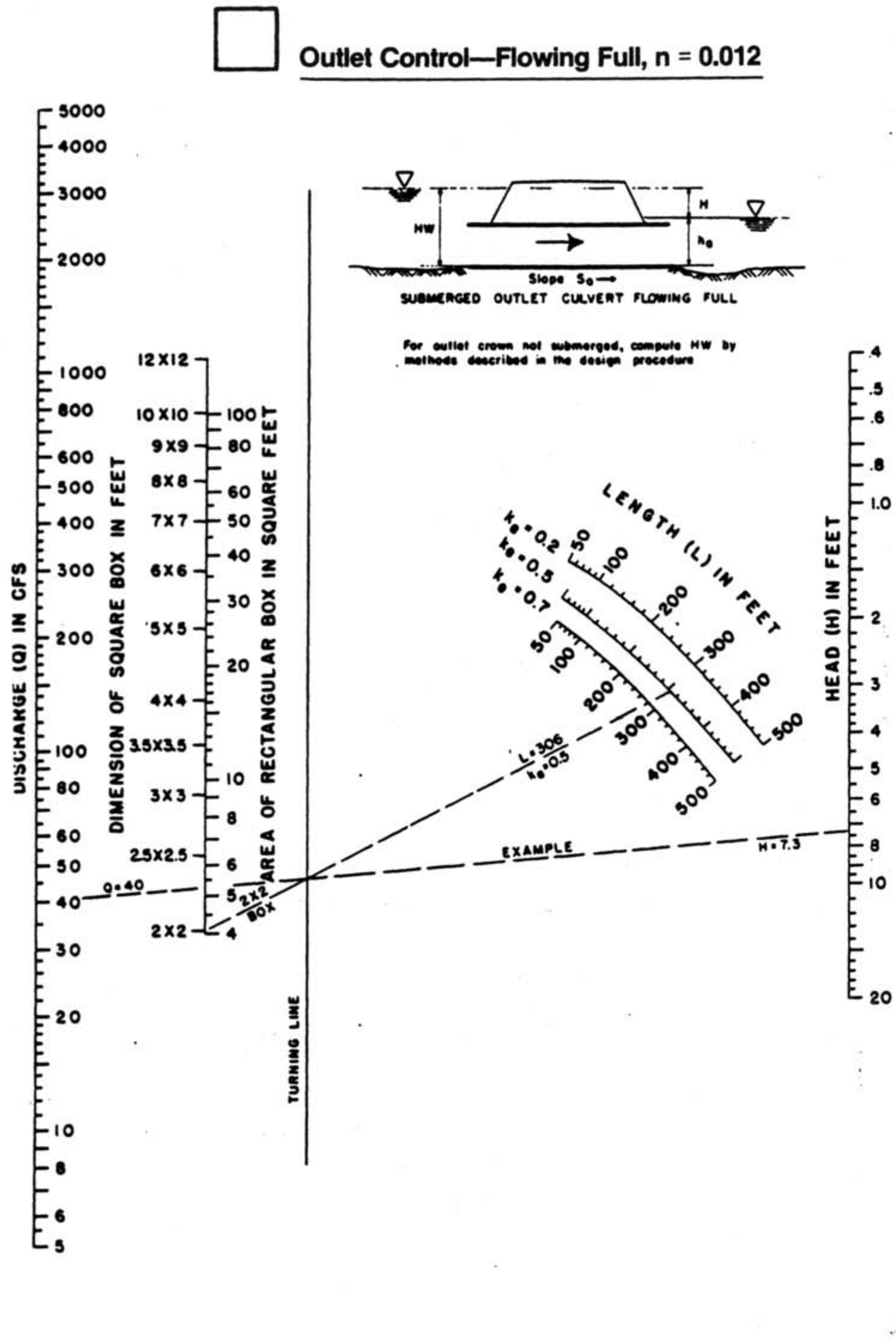
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-9
Outlet Control Chart for Structural Plate
CMP Culverts Flowing Full



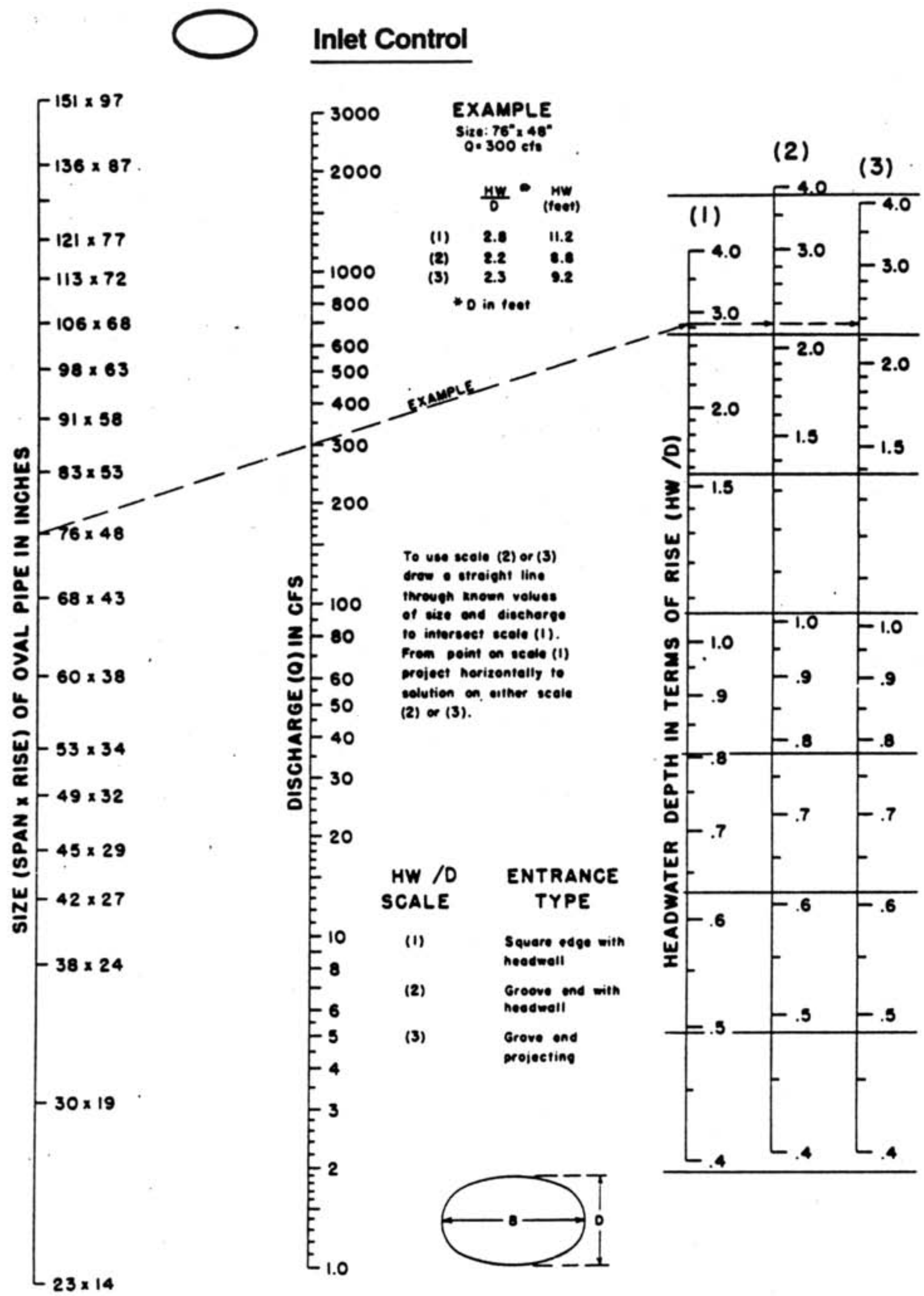
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-10
Inlet Control Chart for Concrete Box Culverts



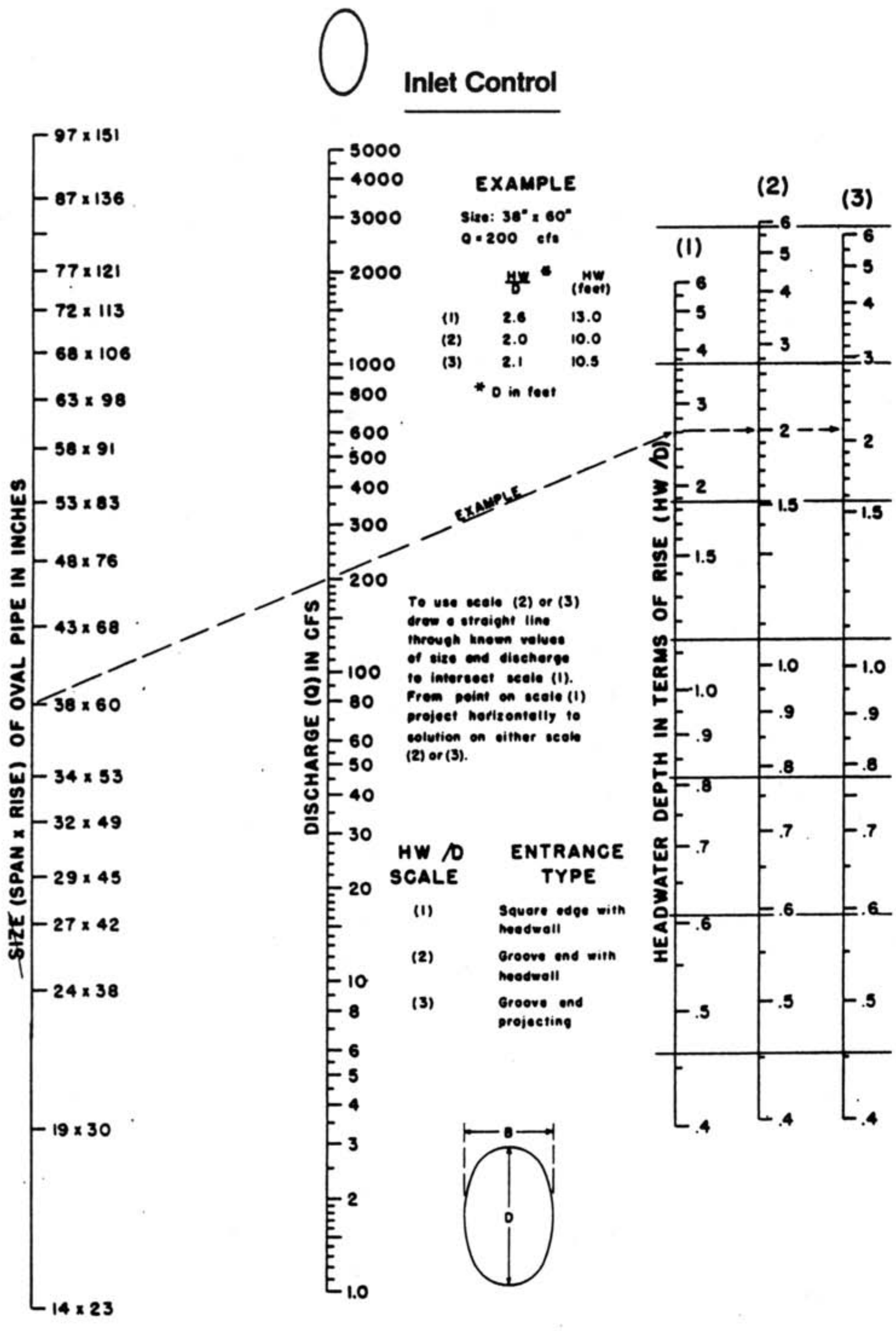
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-11
Outlet Control Chart for Concrete Box Culverts Flowing Full



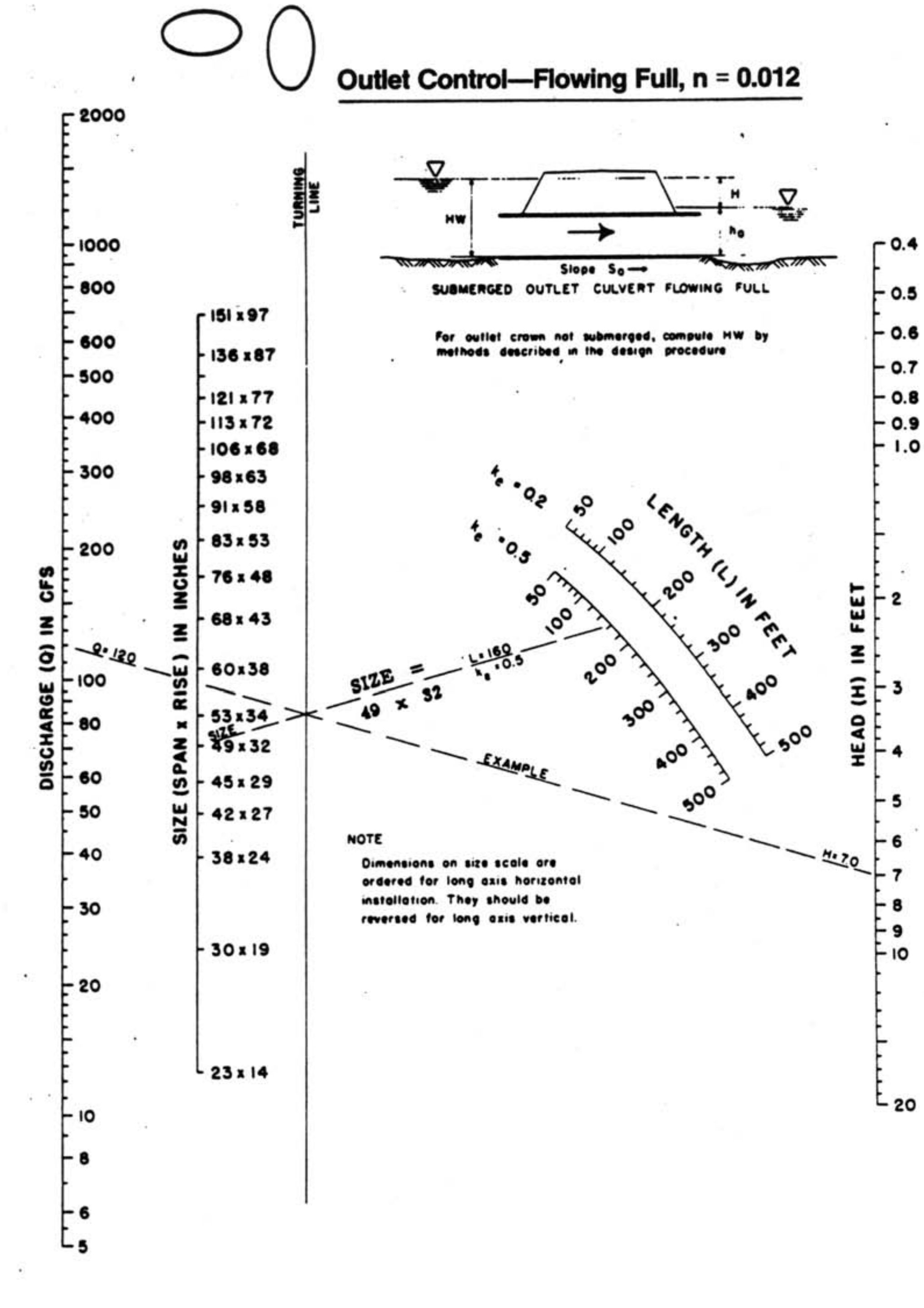
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-12
Inlet Control Chart for Oval Concrete Pipe Culverts – Long Axis Horizontal



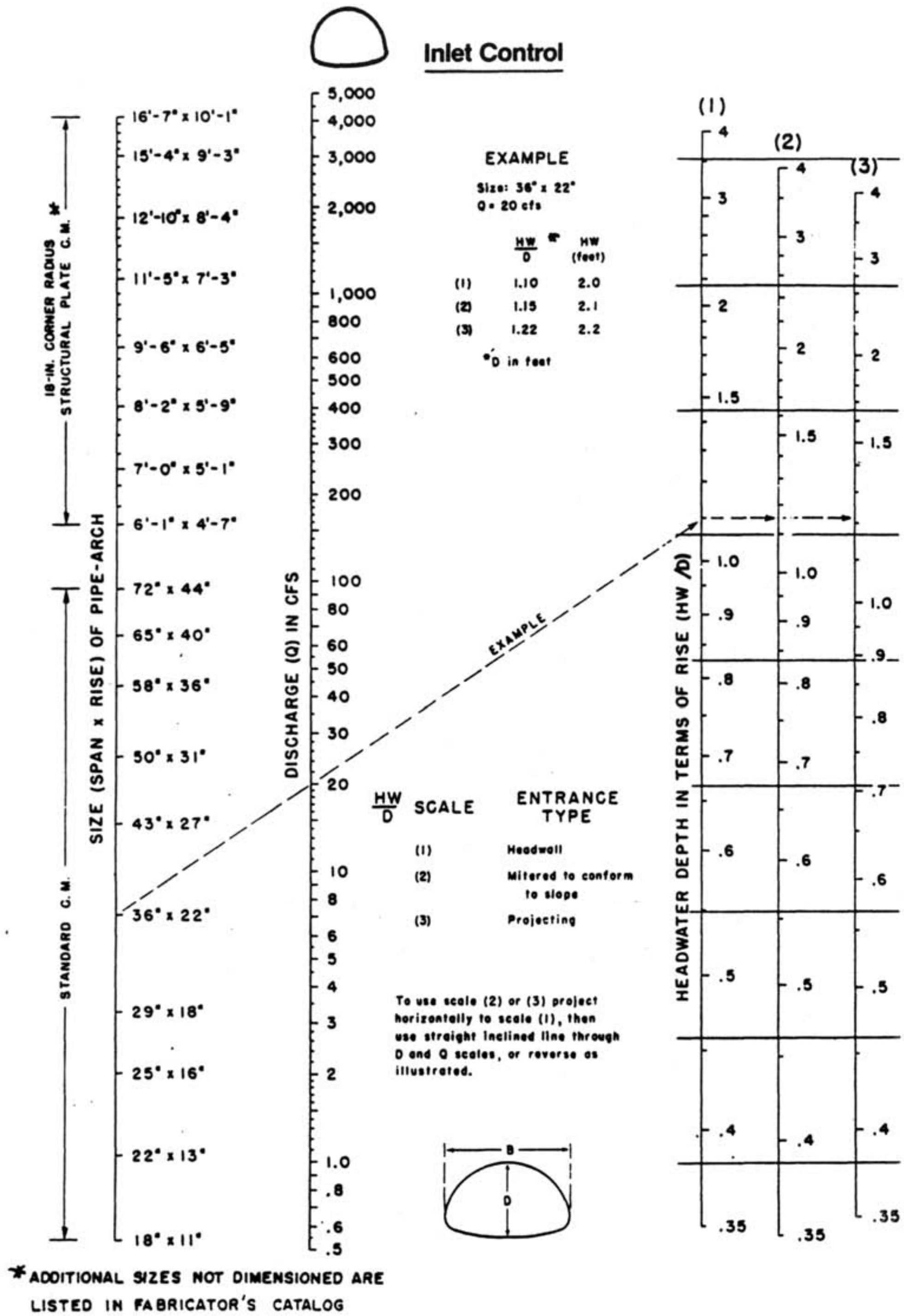
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-13
Inlet Control Chart for Oval Concrete Pipe Culverts – Long Axis Vertical



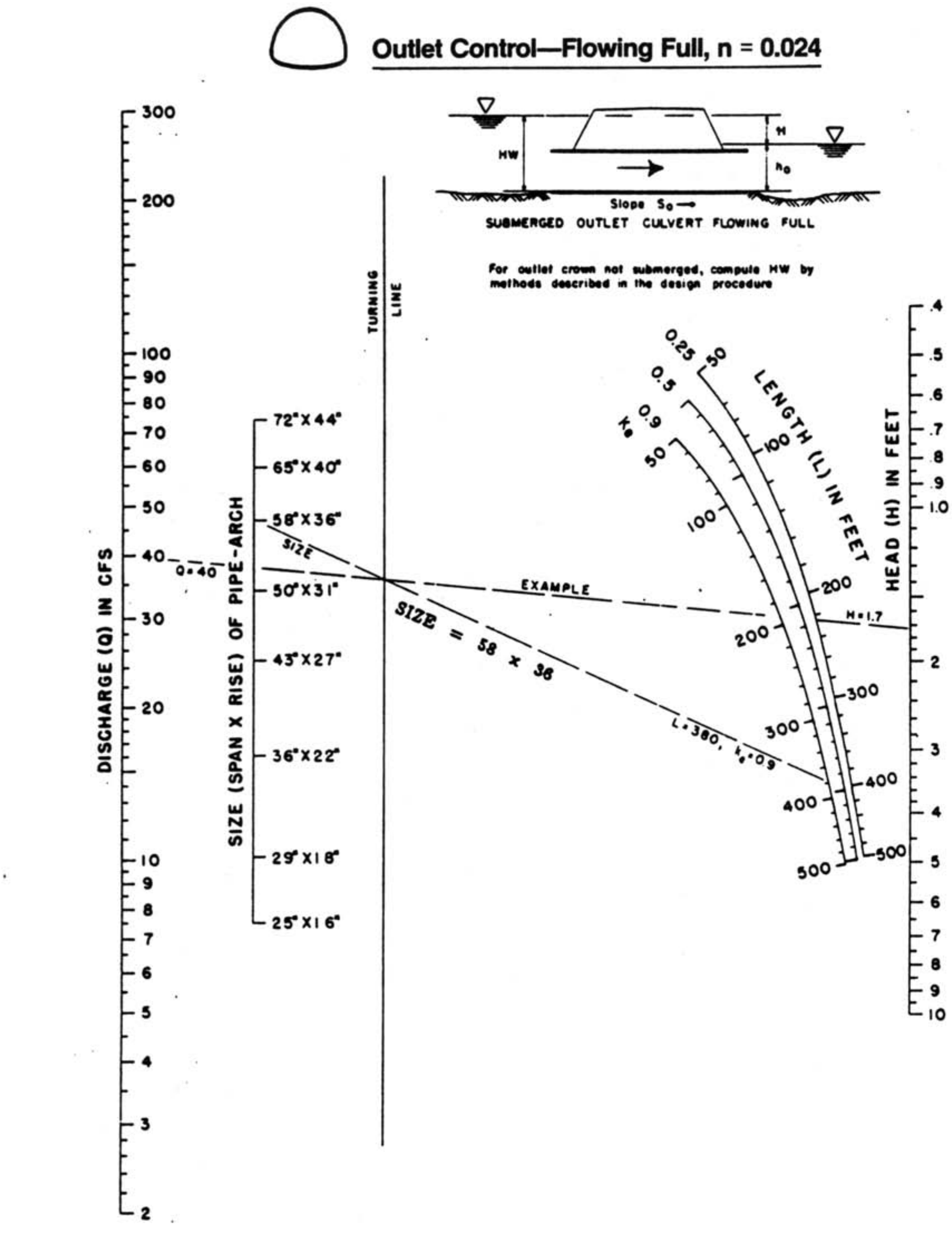
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-14
Outlet Control Chart for Oval Concrete Pipe Culverts Flowing Full –
Long Axis Horizontal or Vertical



Reference: USDOT, FHWA, HDS-5 (1985).

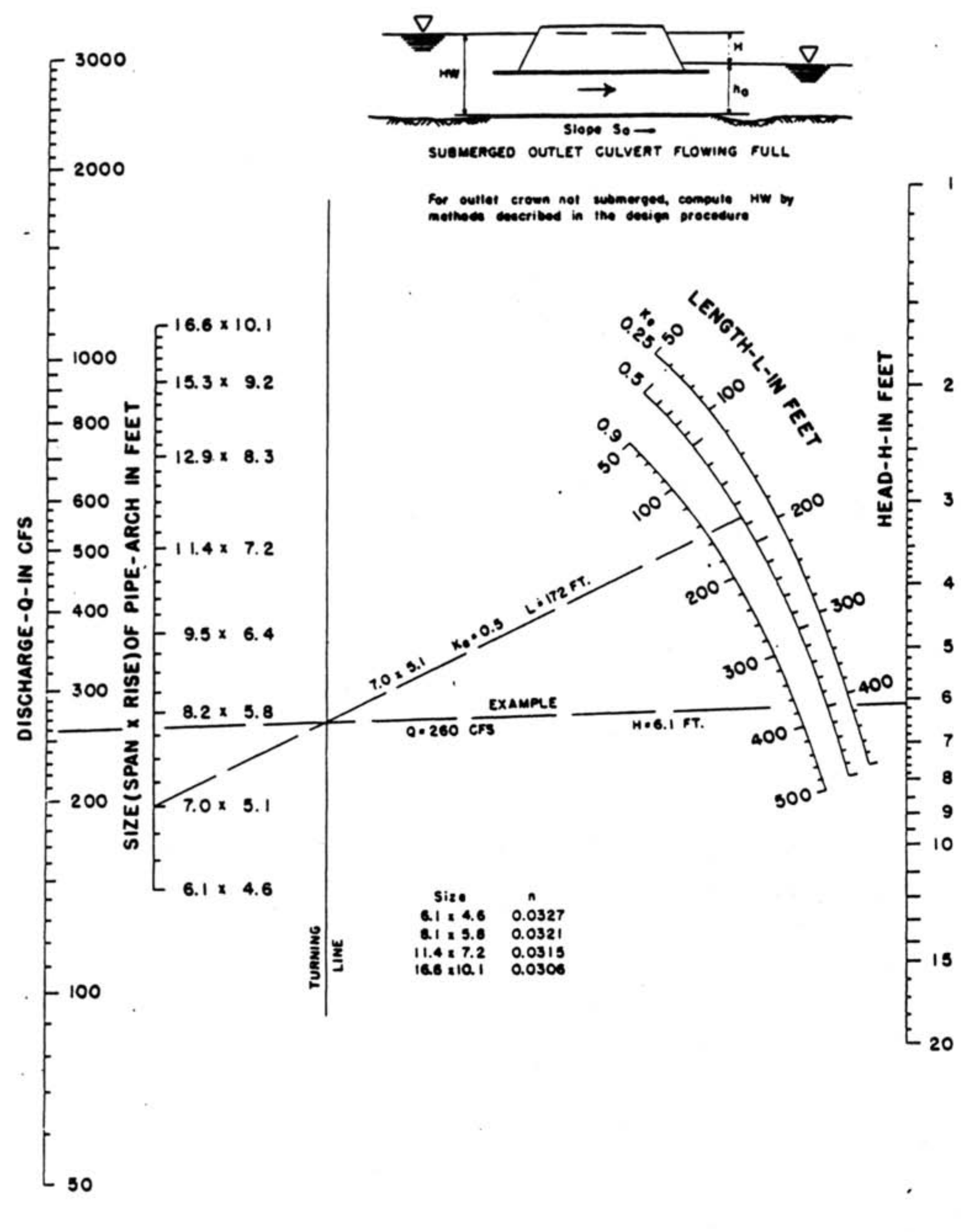
Figure 5-15
Inlet Control Chart for CMP Arch Culverts



Reference: USDOT, FHWA, HDS-5 (1985).

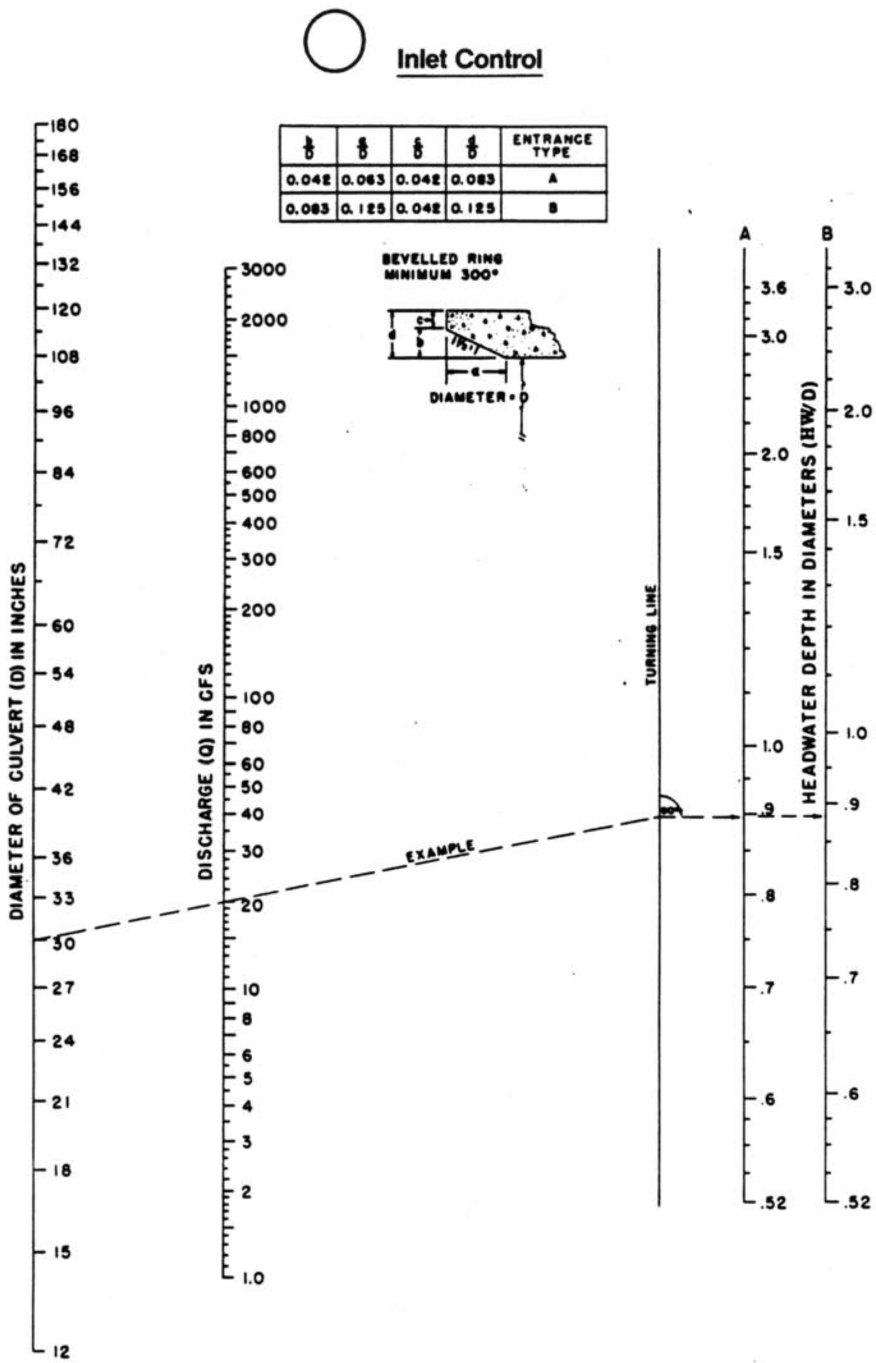
Figure 5-16
Outlet Control Chart for CMP Arch Culverts Flowing Full

Outlet Control—Flowing Full, n = 0.0327 to 0.0306



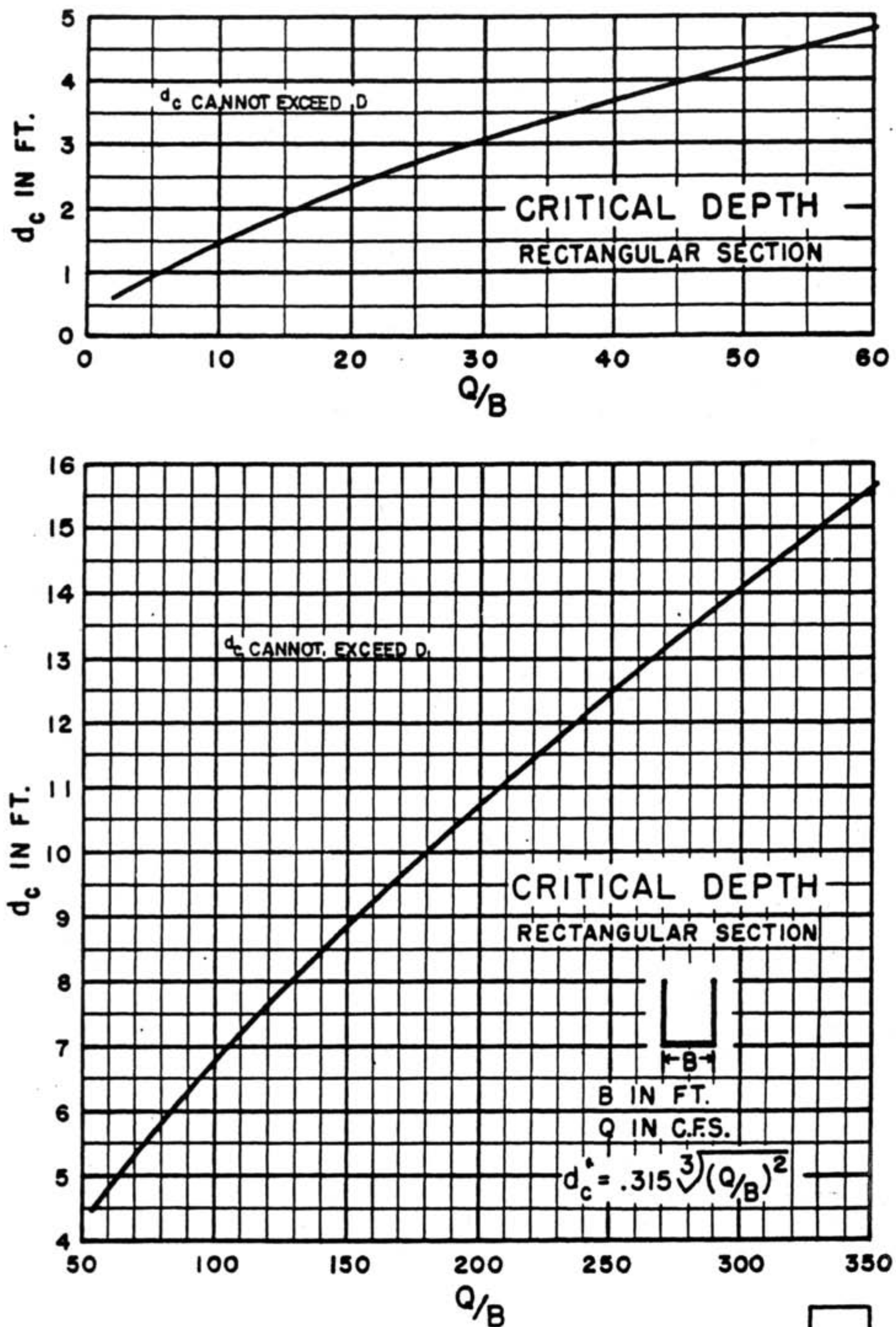
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-17
Outlet Control Chart for Structural Plate CMP Arch Culverts
(18-inch Corner Radius Flowing Full)



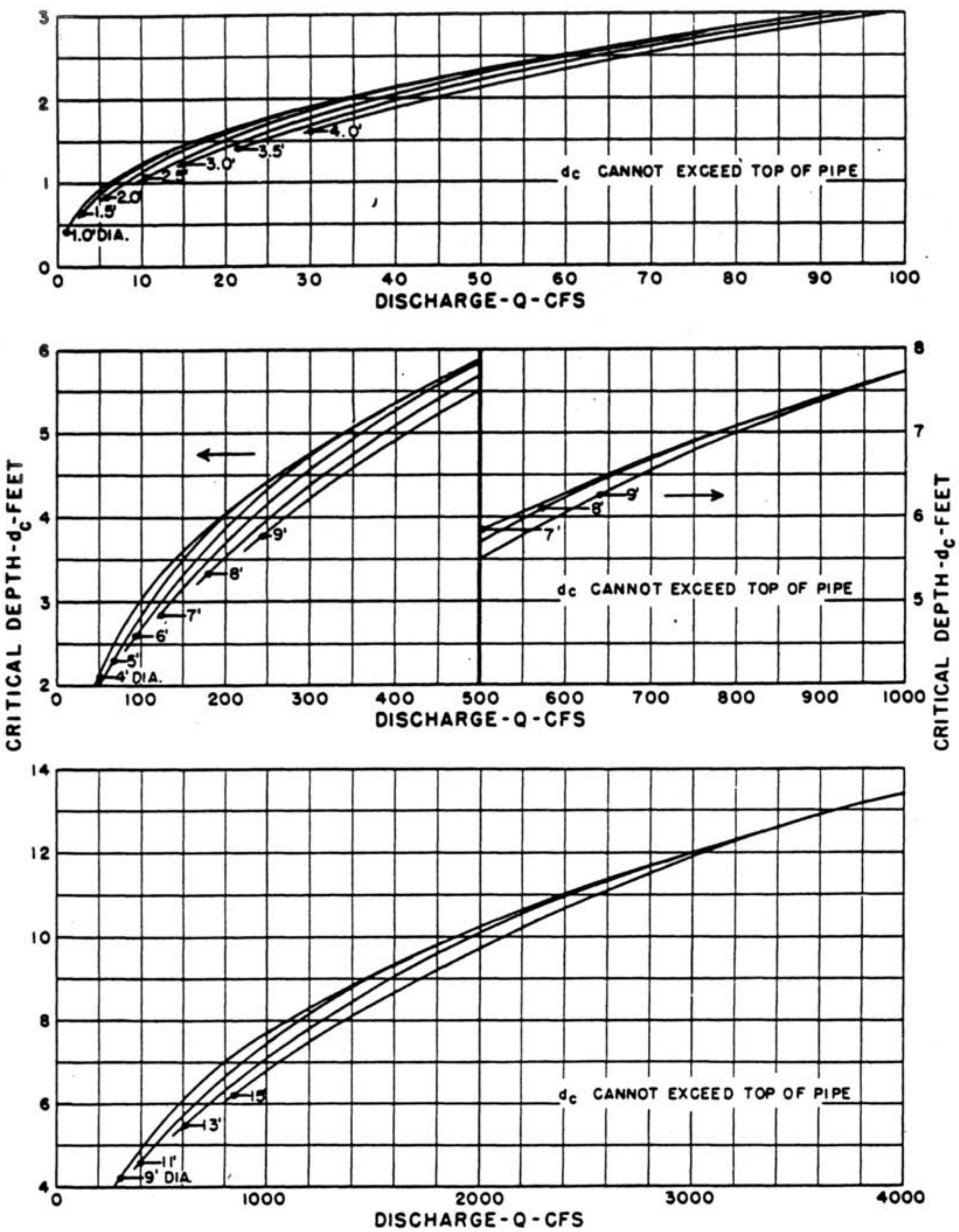
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-18
Inlet Control Chart for Circular Pipe
Culverts with Beveled Ring



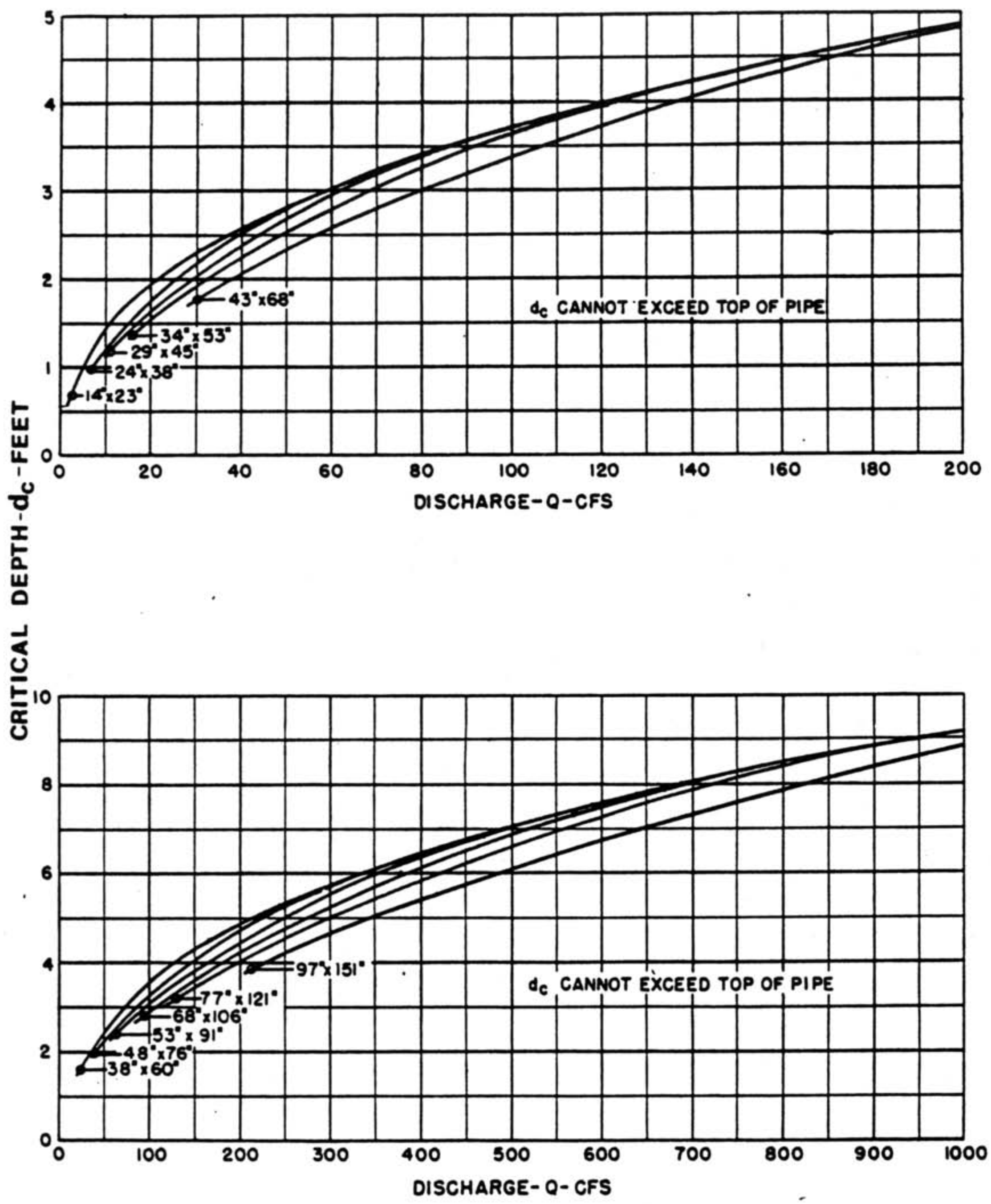
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-19
Critical Depth Chart for Rectangular Sections



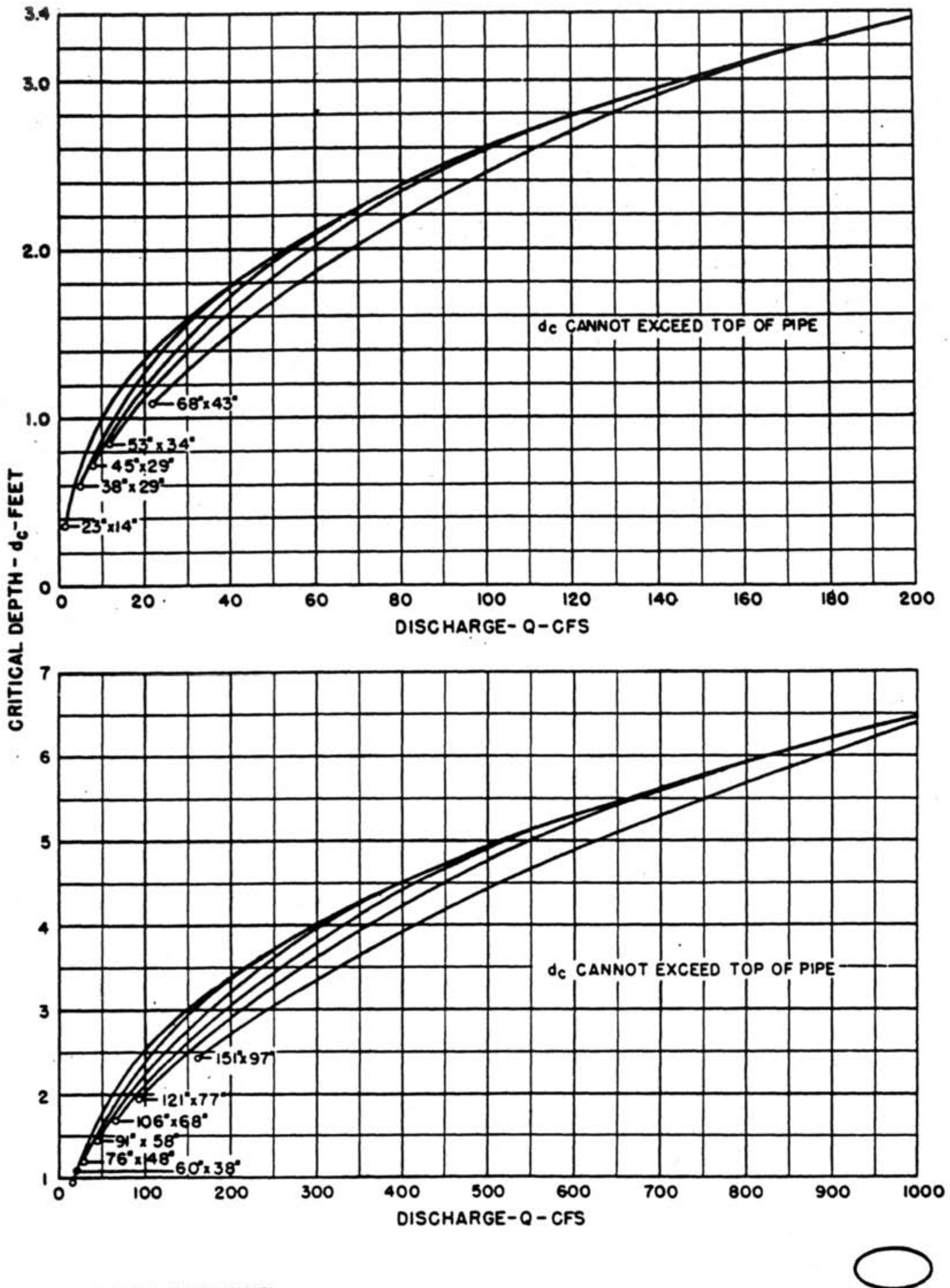
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-20
Critical Depth Chart for Circular Pipe



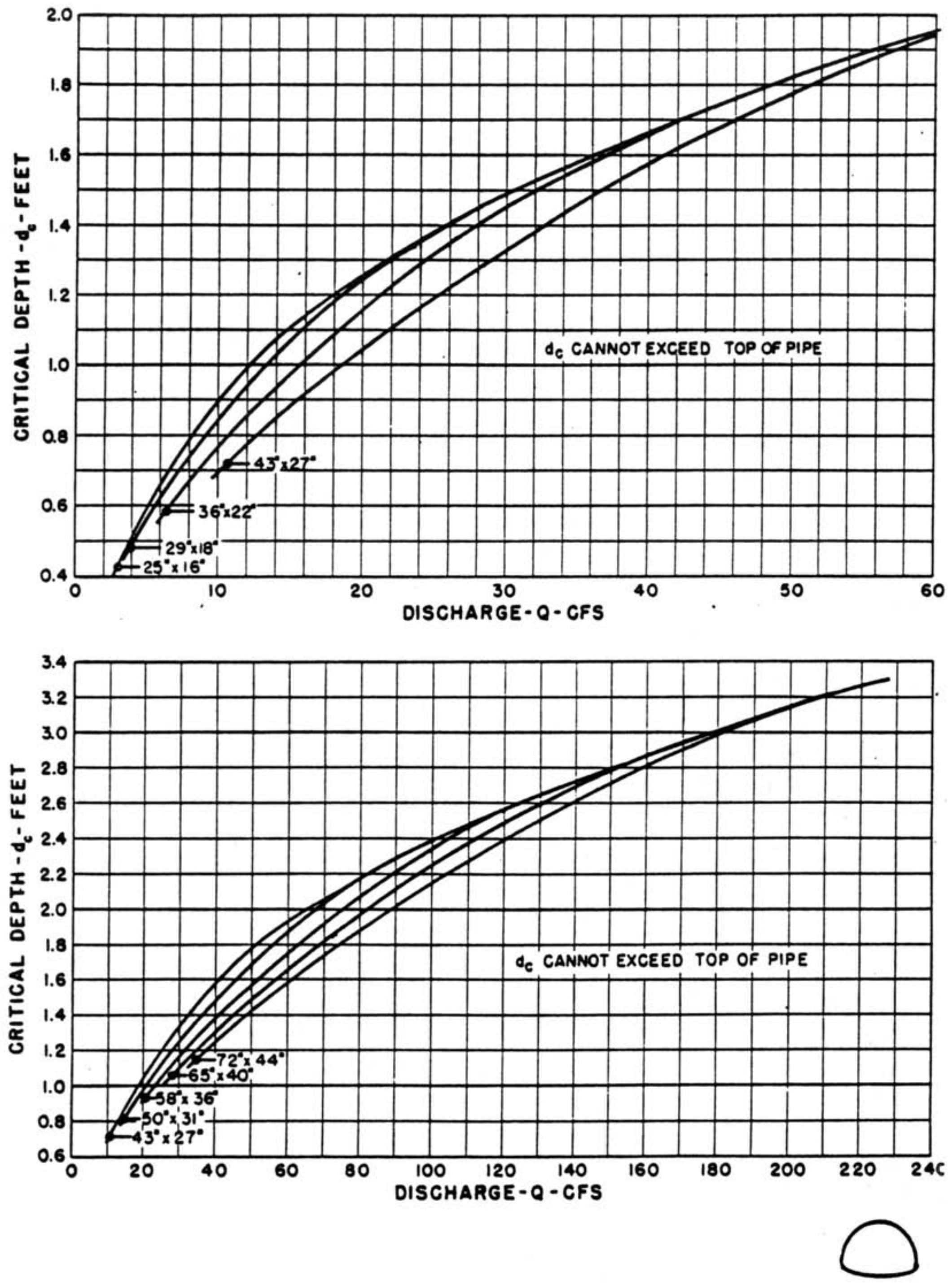
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-21
Critical Depth Chart for Oval Concrete Pipe –
Long Axis Vertical



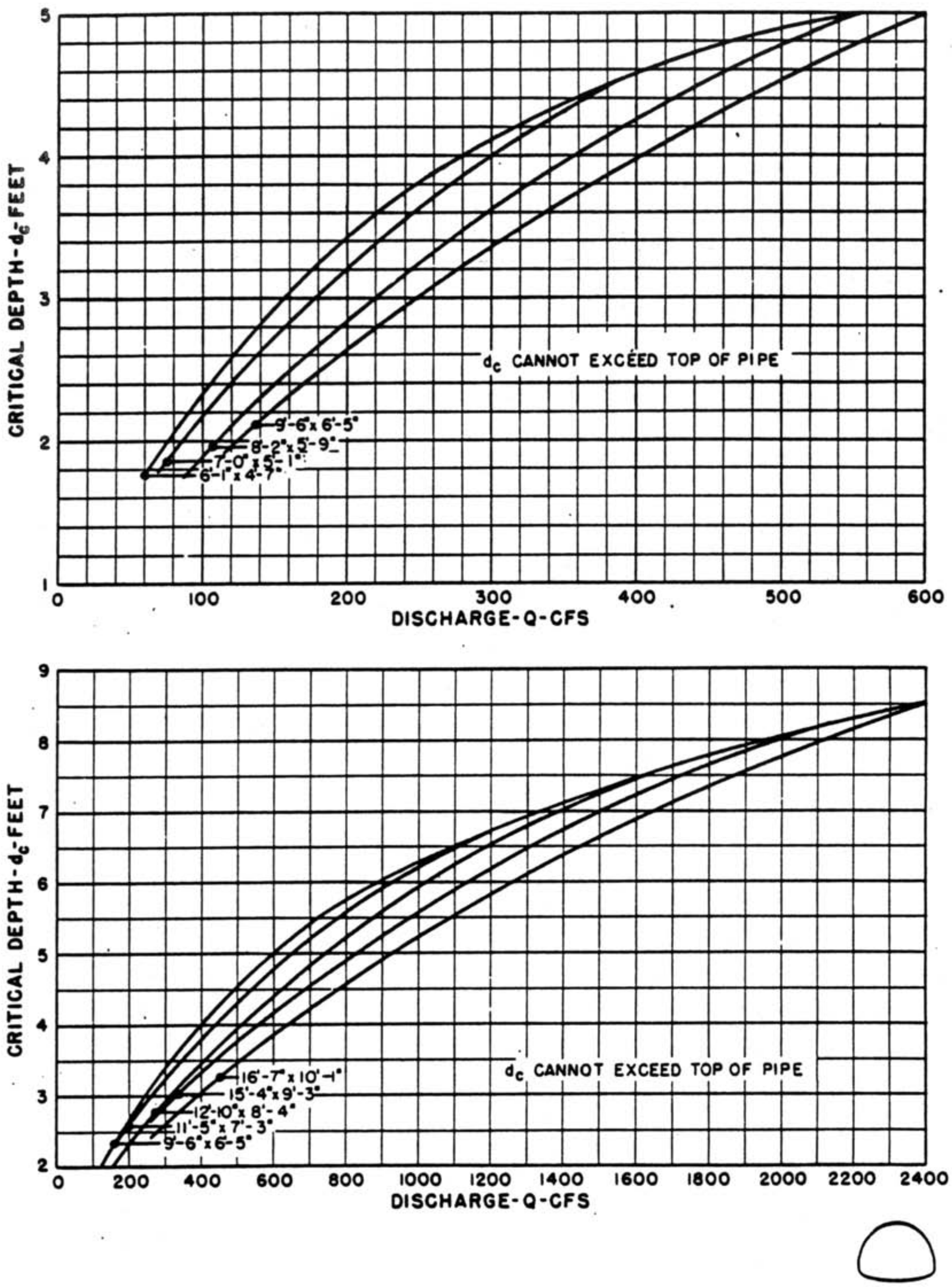
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-22
Critical Depth Chart for Oval Concrete Pipe –
Long Axis Horizontal



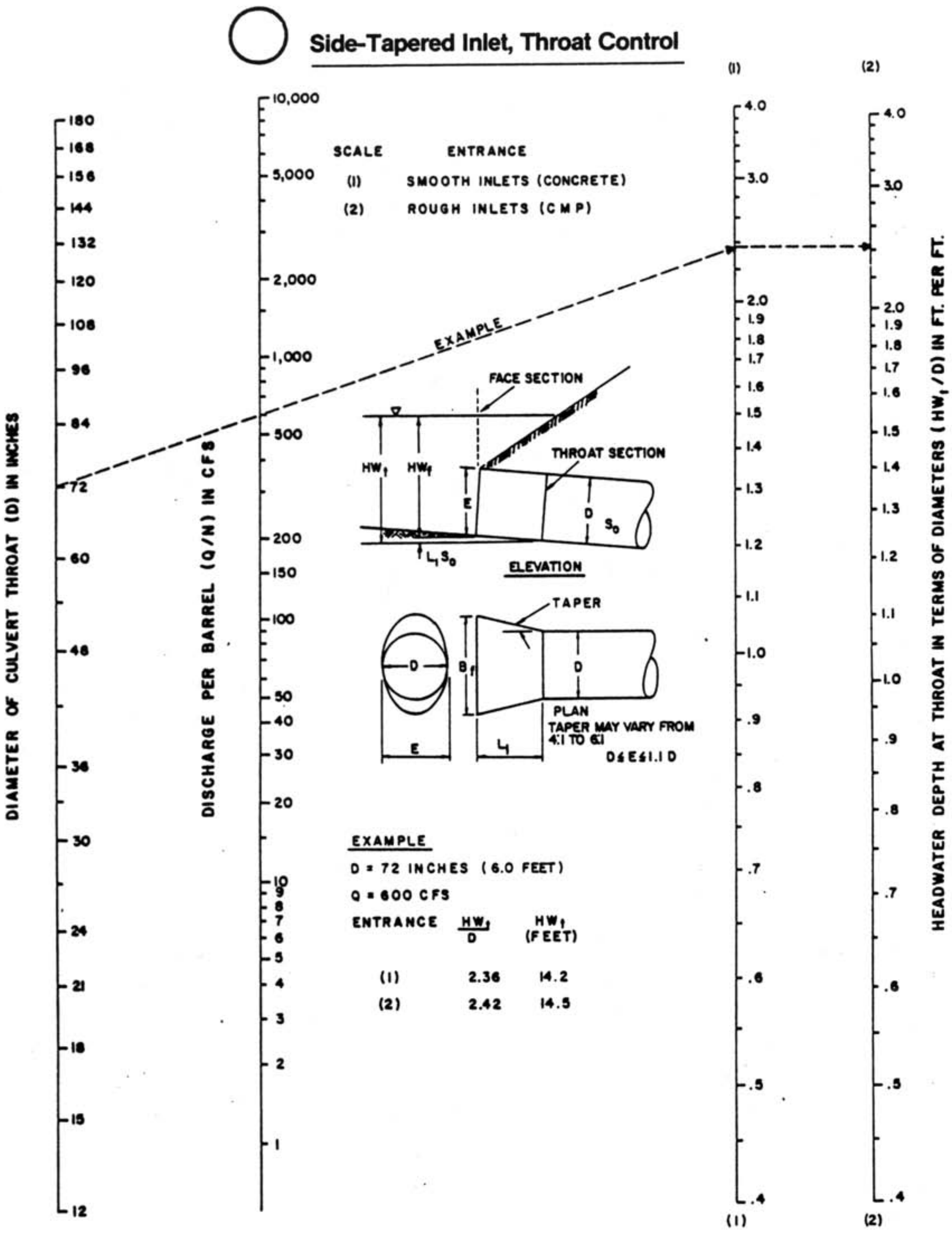
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-23
Critical Depth Chart for Standard CMP Arch



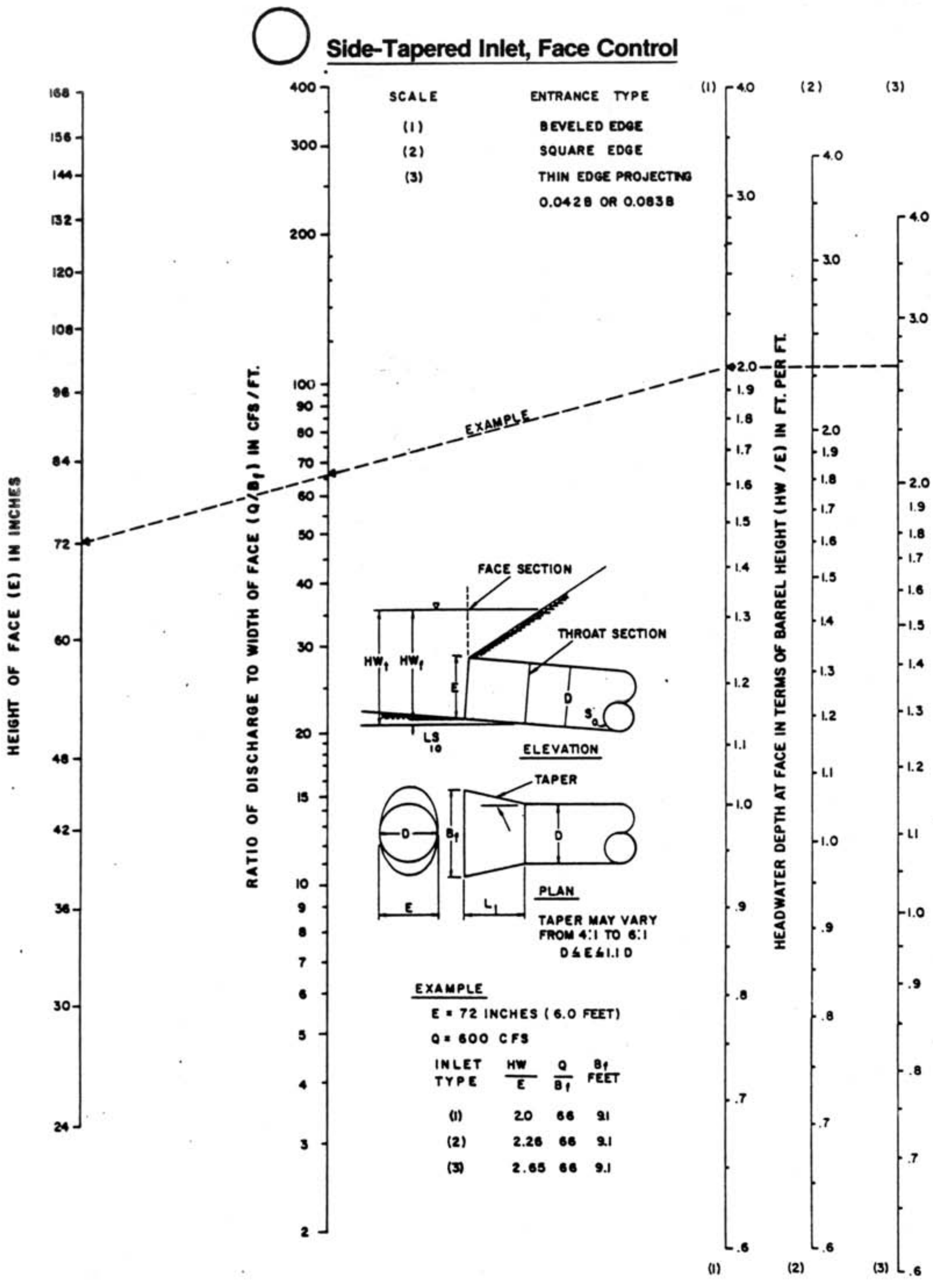
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-24
Critical Depth Chart for Structural Plate CMP Arch



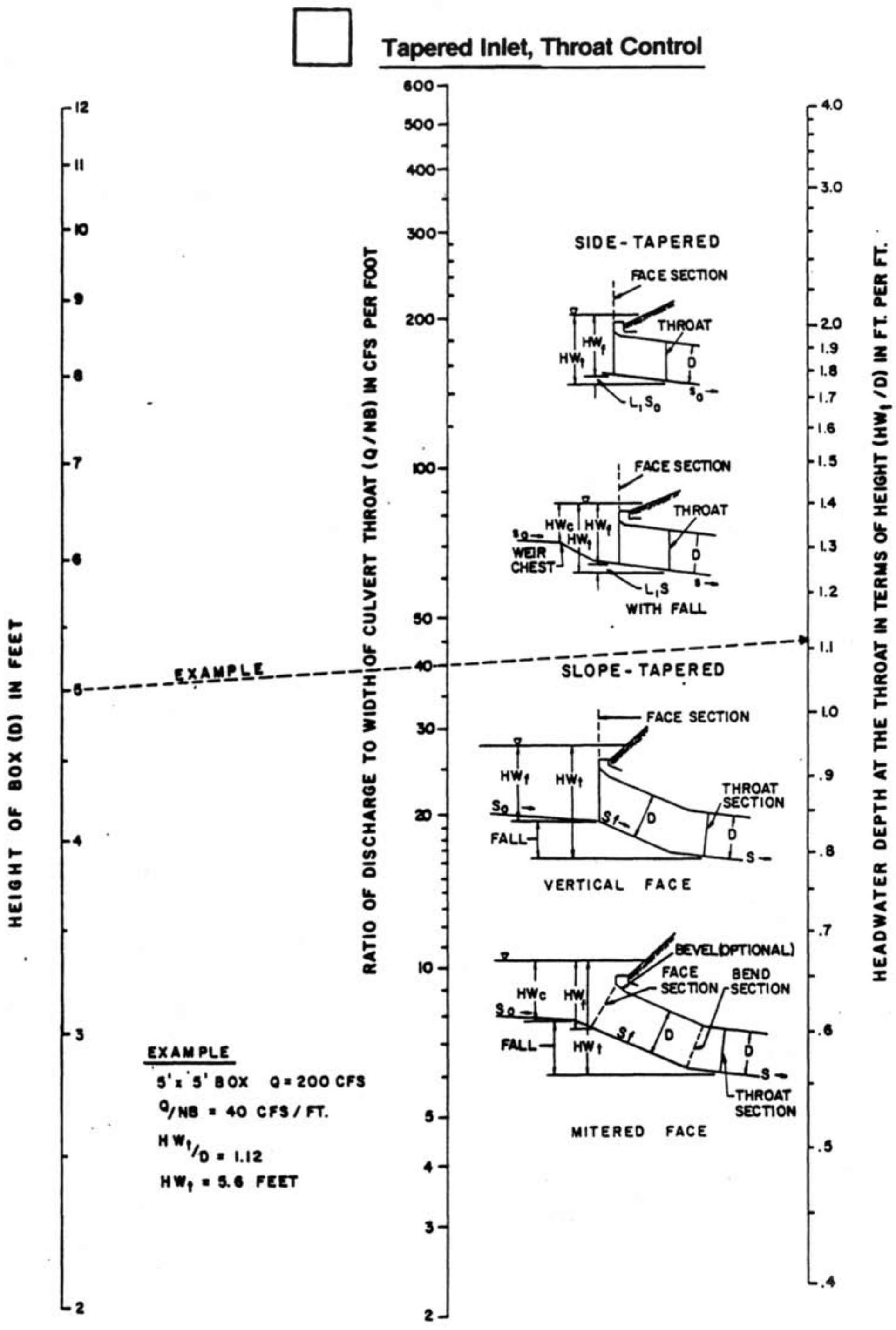
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-25
Throat Control Chart for Side-Tapered Inlets to Pipe Culvert
(Circular Section Only)



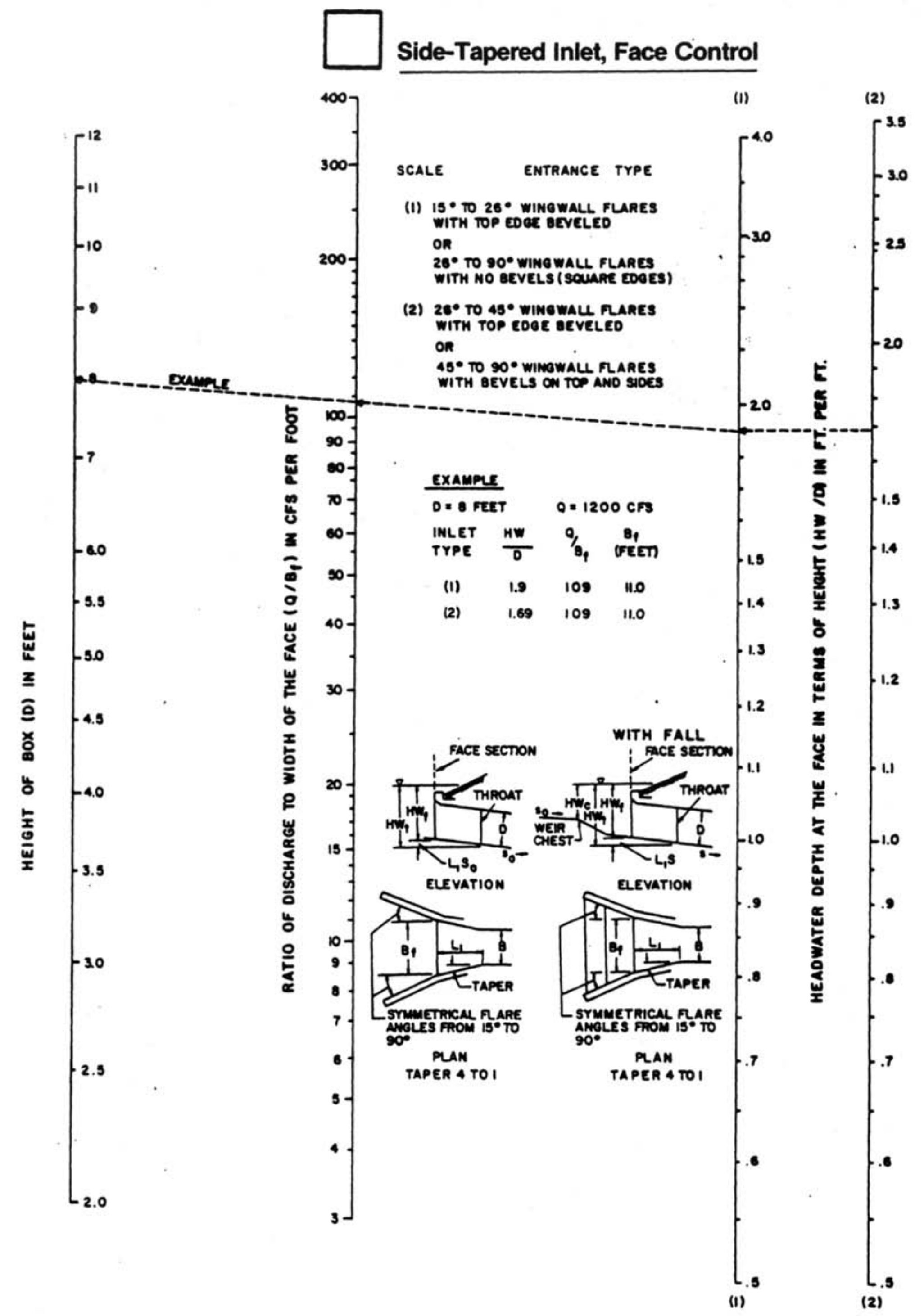
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-26
Face Control Chart for Side-Tapered Inlets to Pipe Culvert
(Non-Rectangular Sections Only)



Reference: USDOT, FHWA, HDS-5 (1985).

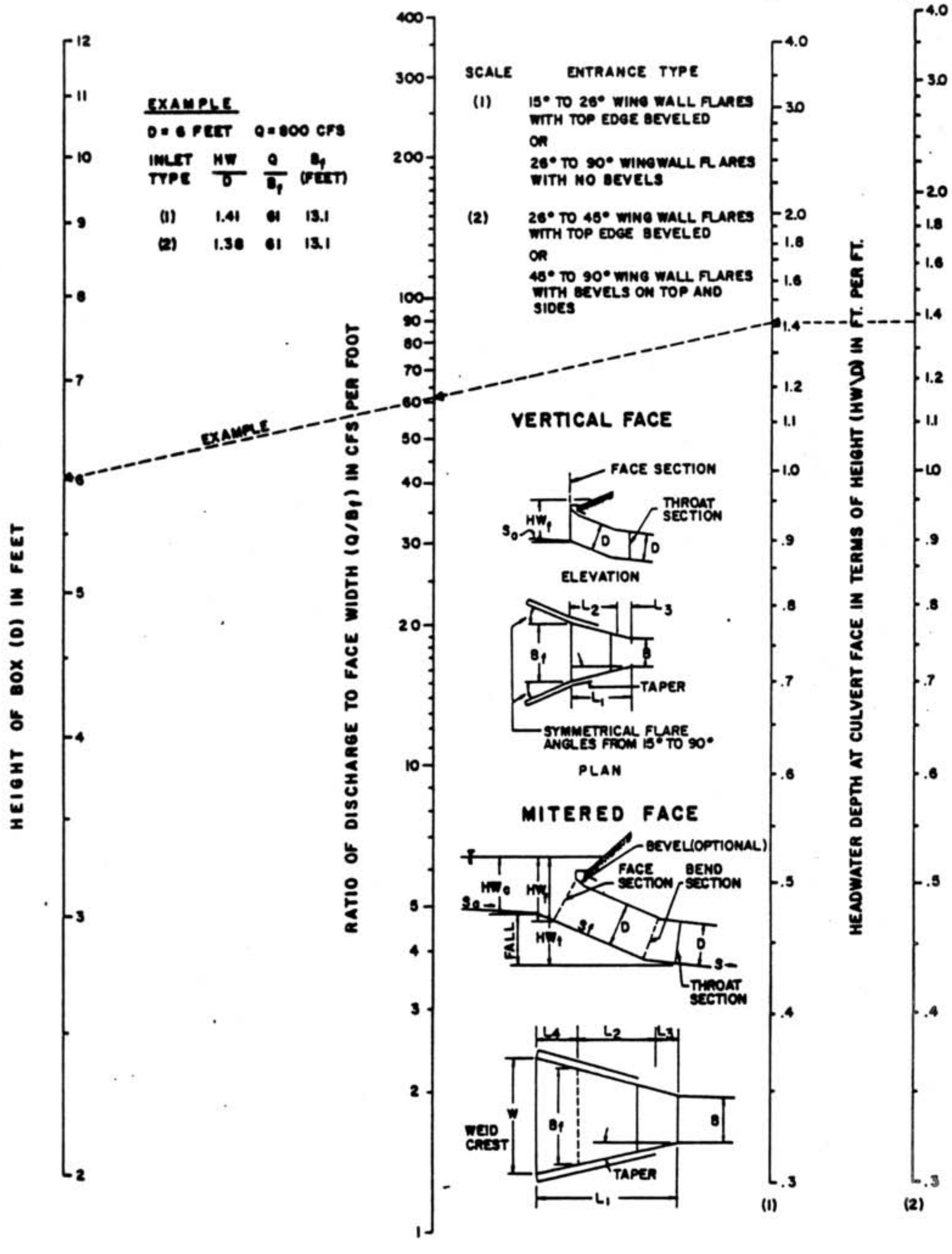
Figure 5-27
Throat Control Chart for Box Culverts with Tapered Inlets



Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-28
Face Control Chart for Box Culverts with Side-Tapered Inlets

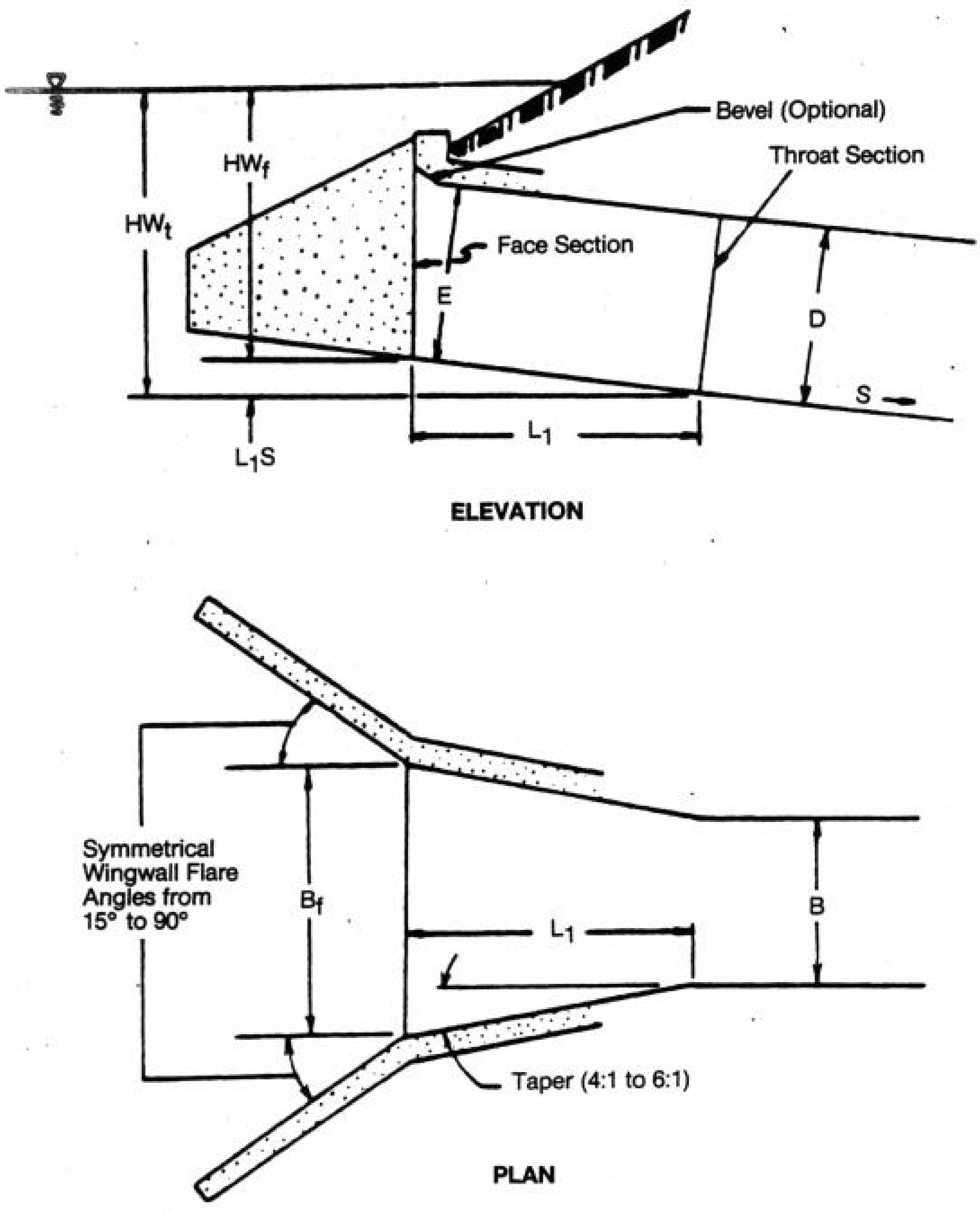
Slope-Tapered Inlet, Face Control



INLET TYPE	HW/D	Q/B_f	B_f (FEET)
(1)	1.41	61	13.1
(2)	1.38	61	13.1

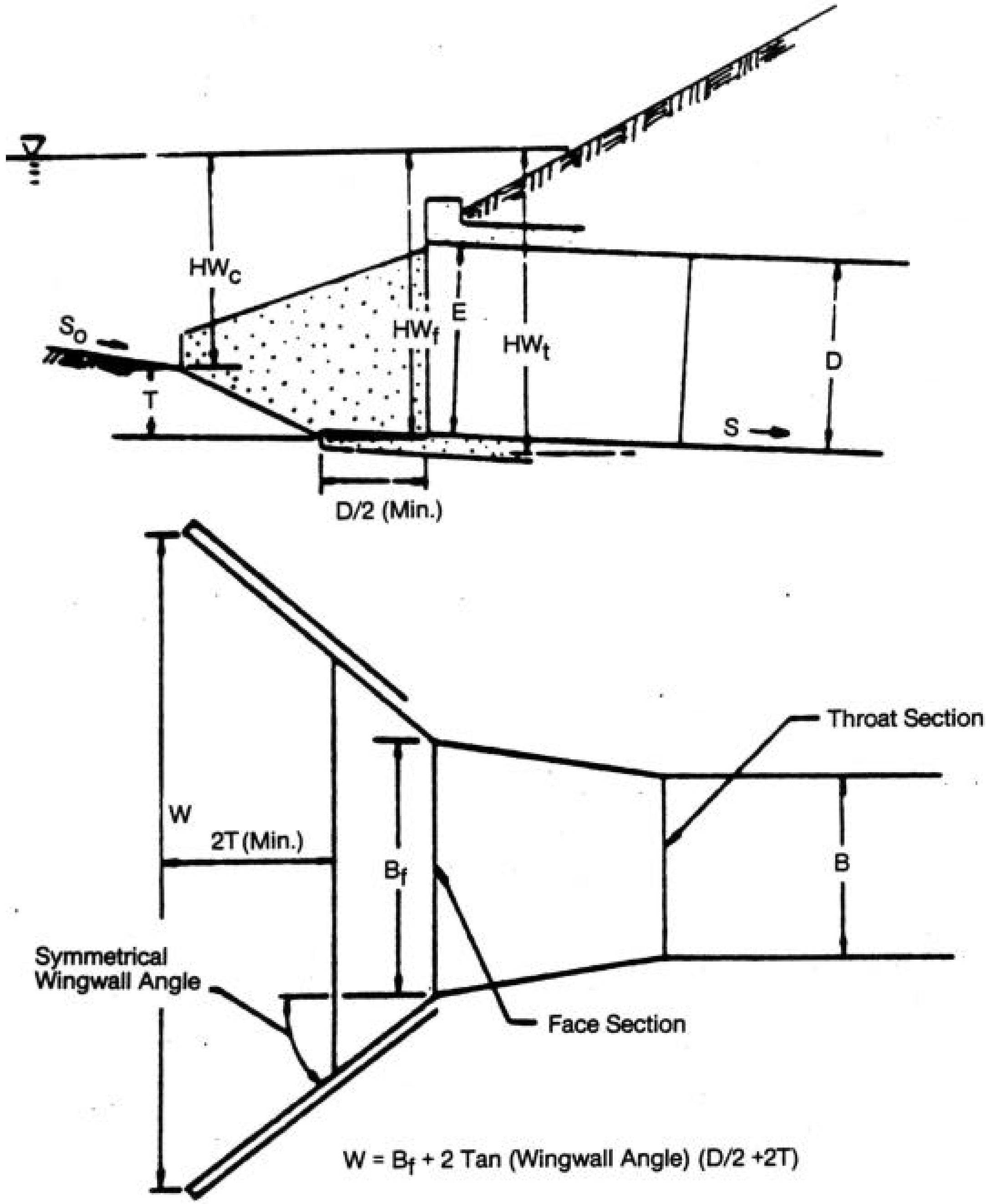
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-29
Face Control Chart for Box Culverts with Slope-Tapered Inlets



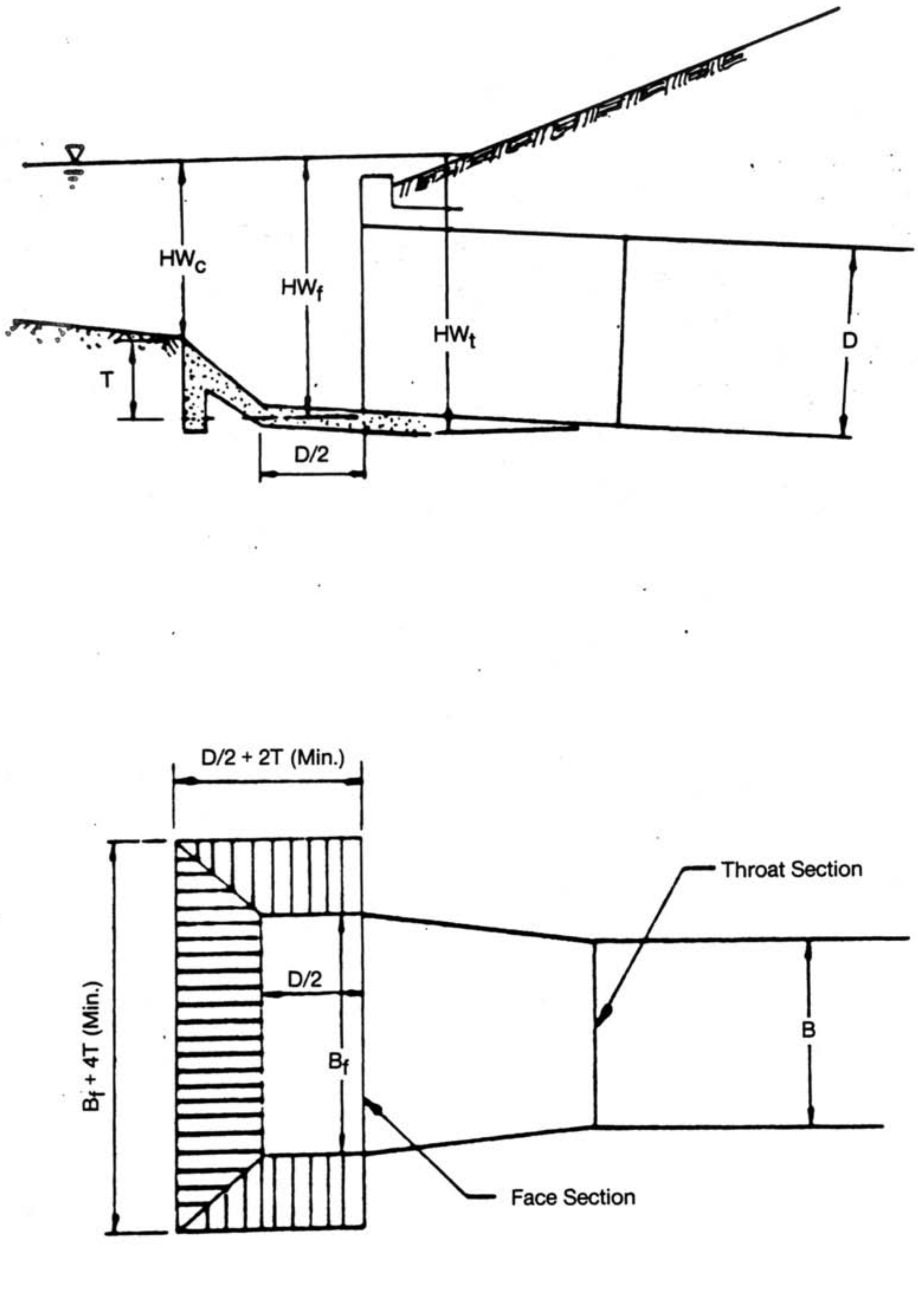
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-30
Typical Side-Tapered Inlet Details



Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-31
Side-Tapered Inlet with Upstream
Depression Contained between Wingwalls



Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-32
Side-Tapered Inlet with Upstream Sump

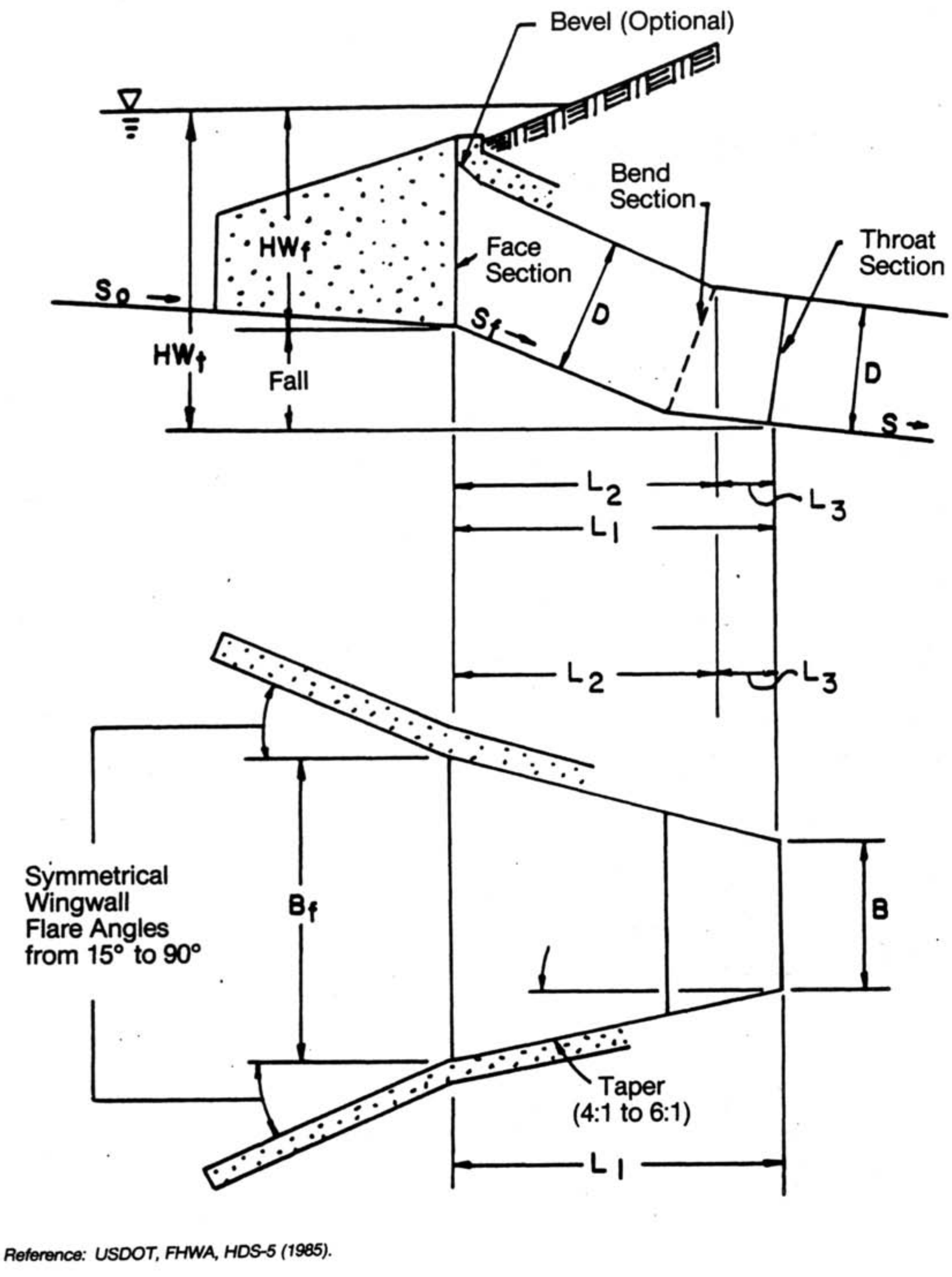
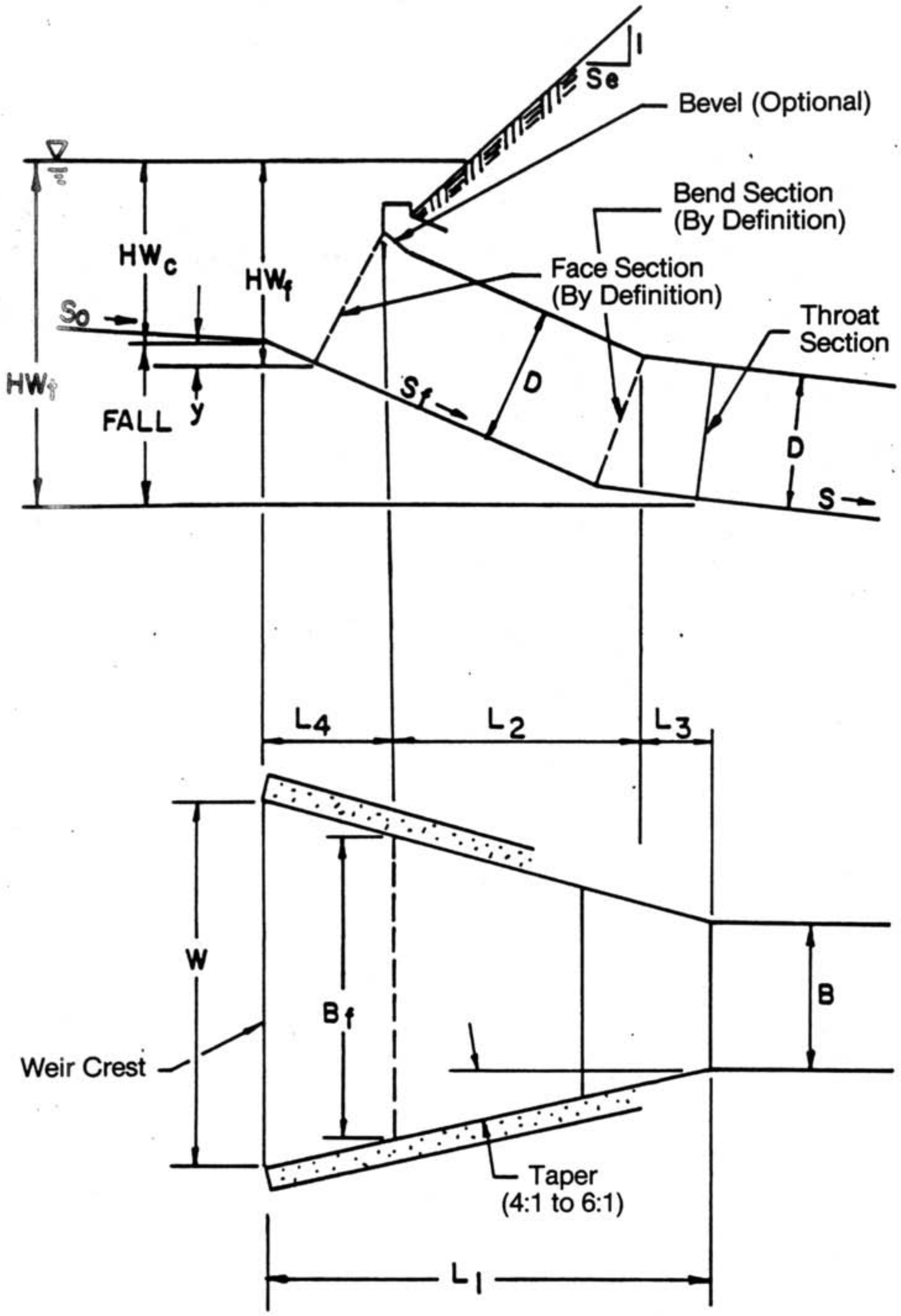
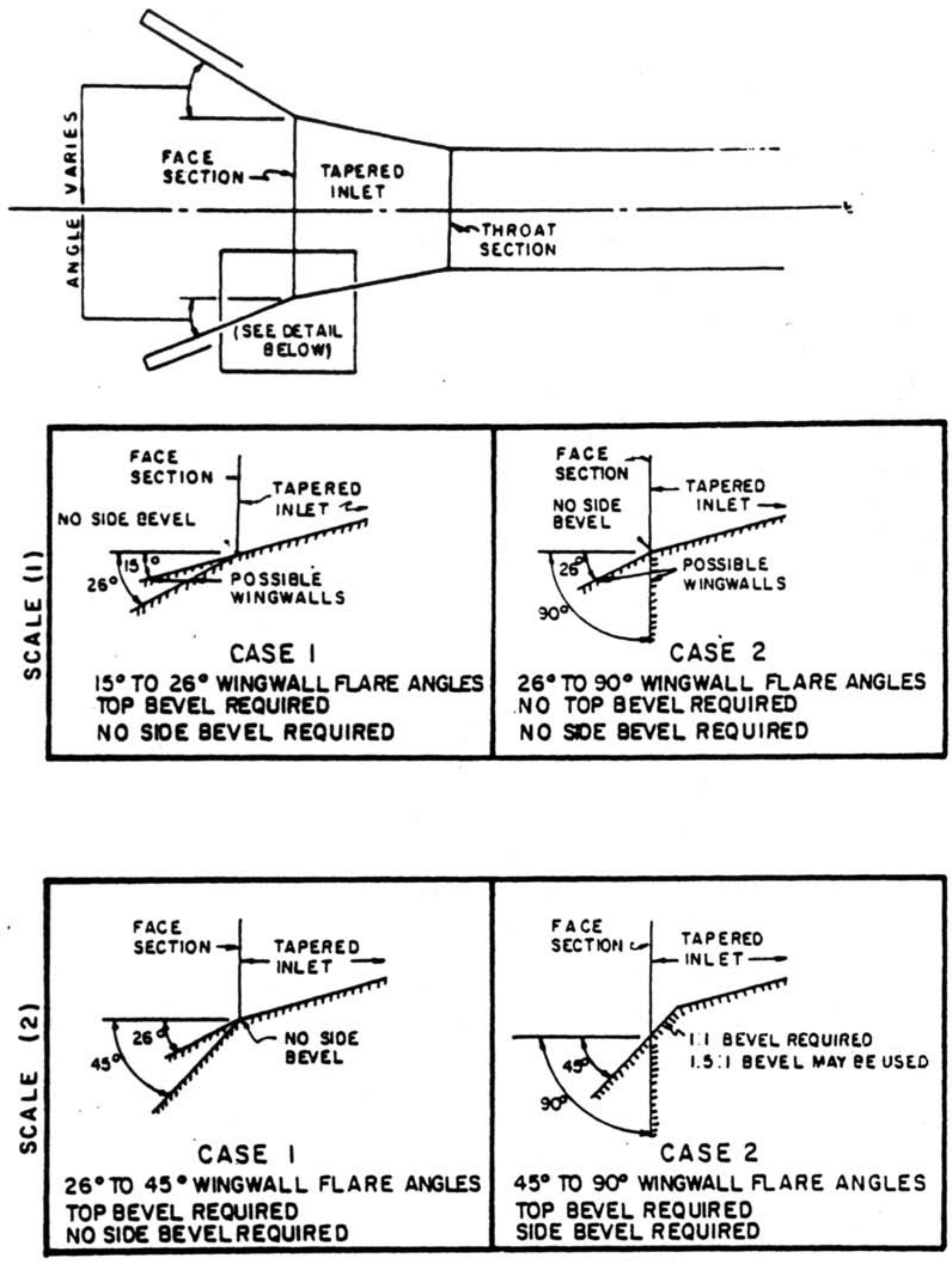


Figure 5-33
Slope-Tapered Inlet with Vertical Face



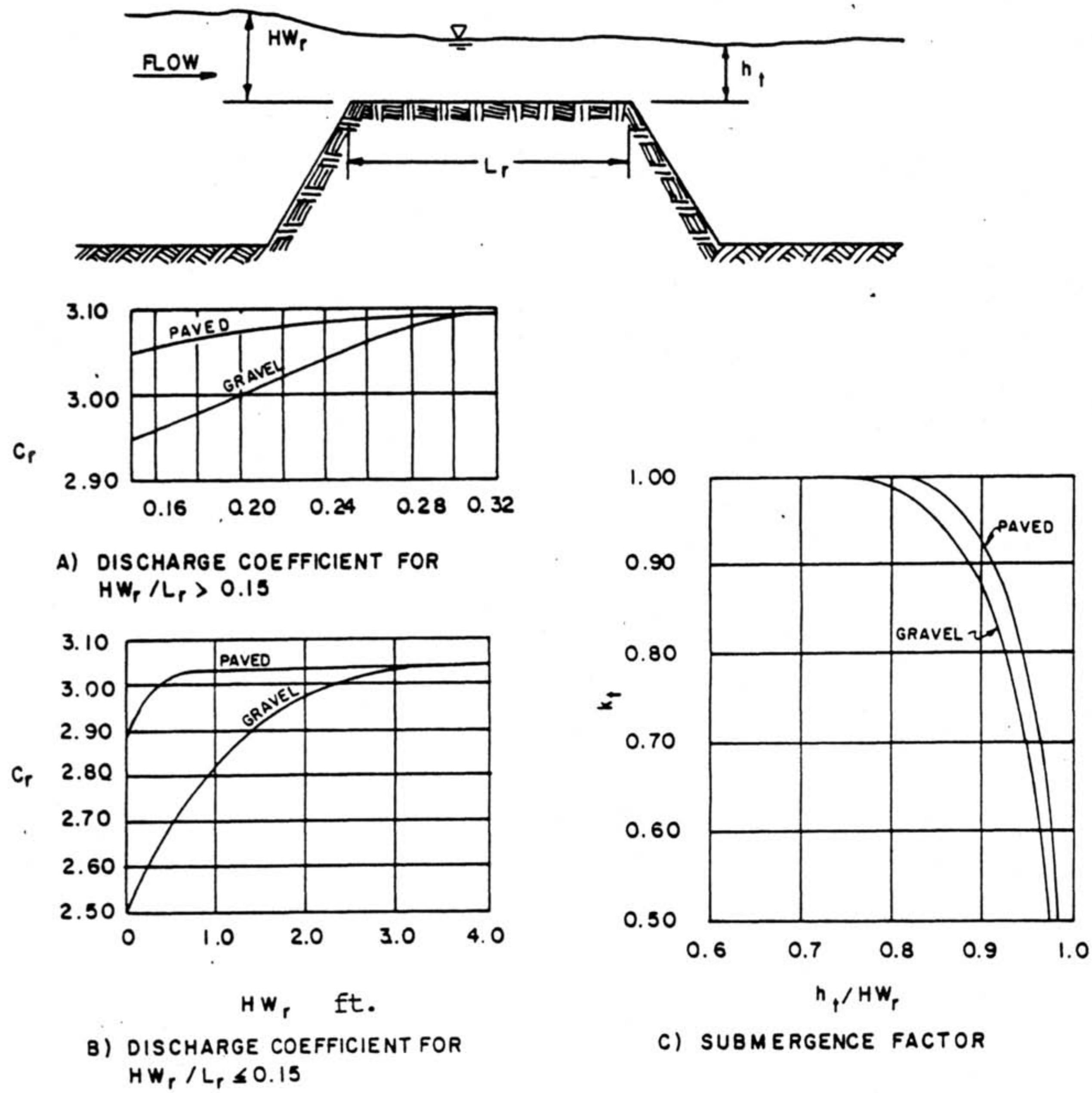
Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-34
Slope-Tapered Inlet with Mitered Face



Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-35
Inlet Edge Conditions, Face Section, Rectangular Tapered Inlets



Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-36
Discharge Coefficients for Roadway Overtopping

PROJECT: EXAMPLE 5-1
CHAPTER 5, SECTION 5.3.6

STATION: 1 + 00
SHEET 1 **OF** 1

CULVERT DESIGN FORM
DESIGNER / DATE: VCA / 6/21
REVIEWER / DATE: JES / 6/21

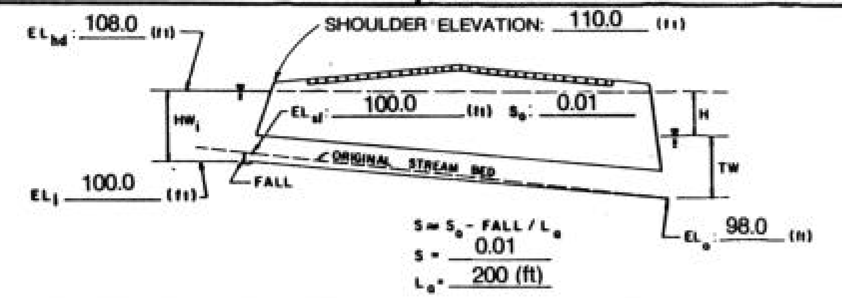
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: RATIONAL
- ☐ DRAINAGE AREA: 125 AC ☐ STREAM SLOPE: 1.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
10	200	3.5



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW_i/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{hi} (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
C.M.P.-Circ-72 in-45° Bevel Headwall	200	200	0.97	5.8	0	105.8	3.5	3.8	4.9	4.9	0.2	2.5	105.4	105.8	9.2	Try Smaller Size
C.M.P.-Circ-60 in-45° Bevel Headwall	200	200	1.42	7.0	0	107.0	3.5	4.1	4.6	4.6	0.2	6.0	108.6	108.6	11.8	Try Concrete
Concrete-Circ-60 in-Groove End, 45° Headwall	200	200	1.36	6.8	0	106.8	3.5	4.1	4.6	4.6	0.2	3.0	105.6	106.8	15.6	Try Smaller Size
Concrete-Circ-54 in-Groove End, 45° Headwall	200	200	1.78	8.0	0	108.0	3.5	3.9	4.2	4.2	0.2	4.8	107.0	108.0	15.2	Okay, Inlet Control

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] v^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

HIGH OUTLET VELOCITY: OUTLET EROSION PROTECTION OR LARGER CONDUIT MAY BE NECESSARY

CULVERT BARREL SELECTED:

SIZE: 54 IN.
SHAPE: CIRCULAR
MATERIAL: CONCRETE n 0.012
ENTRANCE: GROOVE END. 45° HEADWORKS

Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-37
Culvert Design Form Data for Example 5-1

CULVERT DESIGN FORM

PROJECT: EXAMPLE 5-2
CHAPTER 5, SECTION 5.3.6

STATION: 2 + 50
SHEET 1 OF 1

DESIGNER / DATE: VCA / 6/21
REVIEWER / DATE: JES / 6/21

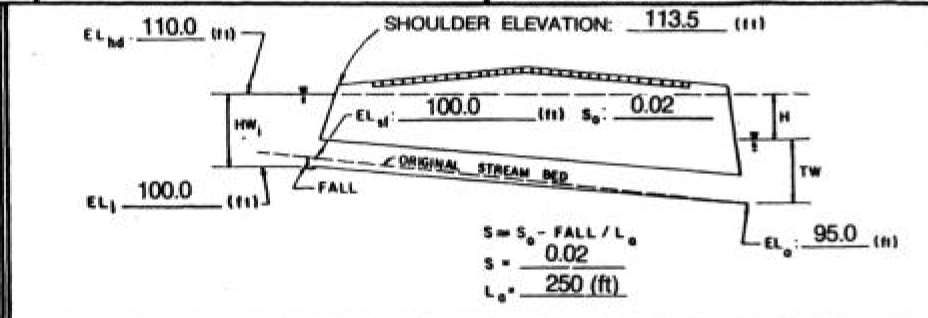
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: SCS
- ☐ DRAINAGE AREA: 200 AC ☐ STREAM SLOPE: 2.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100	300	4.0



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW_i/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{hi} (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Conc.-Box-6'x5'-Square Edge	300	50	1.57	7.9	0	107.9	4.0	4.25	4.6	4.6	0.5	3.6	103.2	107.9	21.1	Try Smaller Box
Conc.-Box-5'x5'-Square Edge	300	60	1.91	9.6	0	109.6	4.0	4.8	4.9	4.9	0.5	5.2	105.1	109.6	21.2	Check Bevels
Conc.-Box-5'x5'-45° Bevel	300	60	1.71	8.6	0	108.6	4.0	4.8	4.9	4.9	0.2	4.6	104.5	108.6	21.2	Okay, Inlet Control

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L) / R^{1.33}\right] v^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

5' x 5' BOX WILL WORK WITH OR WITHOUT BEVELS. BEVELS PROVIDE ADDITIONAL FLOW CAPACITY. HIGH OUTLET VELOCITIES EXIST.

CULVERT BARREL SELECTED:

SIZE: 5 FT. x 5 FT.
SHAPE: RECTANGULAR
MATERIAL: CONCRETE n 0.012
ENTRANCE: 90° HEADWALL

Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-38
Culvert Design Form
Data for Example 5-2

PROJECT: EXAMPLE 5-3
CHAPTER 5, SECTION 5.3.6

STATION: 4 + 00
SHEET 2 **OF** 3

CULVERT DESIGN FORM
DESIGNER / DATE: VCA / 6/21
REVIEWER / DATE: JES / 6/21

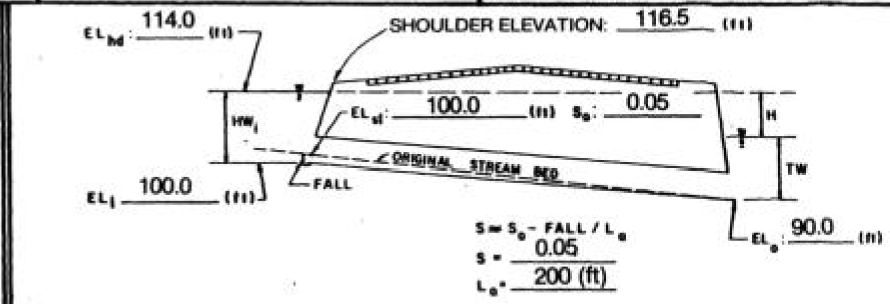
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: SCS
- ☐ DRAINAGE AREA: 400 AC ☐ STREAM SLOPE: 5.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100 (OLD)	600	3.1
100 (NEW)	1000	4.1



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	INLET CONTROL HW_i/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{hi} (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Conc.-Box-7'x7'-Square Edge	850	121.4	2.55	17.9	0	117.9	3.9	≥7	7.0	7.0	0.5	9.0	106.0	117.9	38.7	—
	900	128.6	2.72	19.0	0	119.0	4.0	≥7	7.0	7.0	0.5	10.3	107.3	119.0	39.3	—
	950	135.7	2.95	20.7	0	120.7	4.0	≥7	7.0	7.0	0.5	11.4	108.4	120.7	39.9	—
	1000	142.9	3.21	22.5	0	122.5	4.1	≥7	7.0	7.0	0.5	12.8	109.8	122.5	40.4	—

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] v^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw TAILWATER

COMMENTS / DISCUSSION:

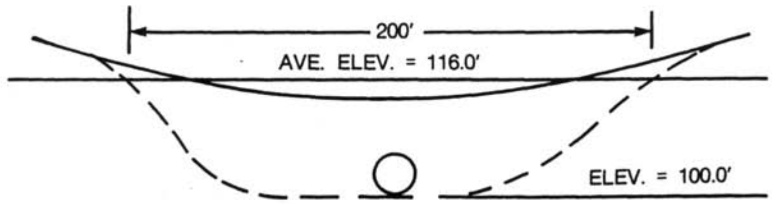
NEW Q_{100} RESULTS IN ROADWAY OVERTOPPING OF ABOUT 2.0' (SEE FIGURE 5-40).

CULVERT BARREL SELECTED:

SIZE: 7 FT. x 7 FT.
SHAPE: RECTANGULAR
MATERIAL: CONCRETE n 0.012
ENTRANCE: SQ. EDGE W/HEADWALL

Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-39
Culvert Design Form
Data for Example 5-3



Culvert Flow (cfs)	EL_h (ft)	HW_r (ft)	C_d	Q_o Weir Flow (cfs)	Q Total Flow (cfs)
400	108.1	–	–	–	400
450	108.9	–	–	–	450
500	109.7	–	–	–	500
550	110.6	–	–	–	550
600	111.6	–	–	–	600
650	112.6	–	–	–	650
700	113.7	–	–	–	700
750	115.0	–	–	–	750
800	116.5	0.5	2.70	191	991
850	117.9	1.9	2.97	1,556	2,406
900	119.0	3.0	3.03	3,150	4,049
950	120.7	4.7	3.03	6,175	7,125
1000	122.5	6.5	3.03	10,043	11,043

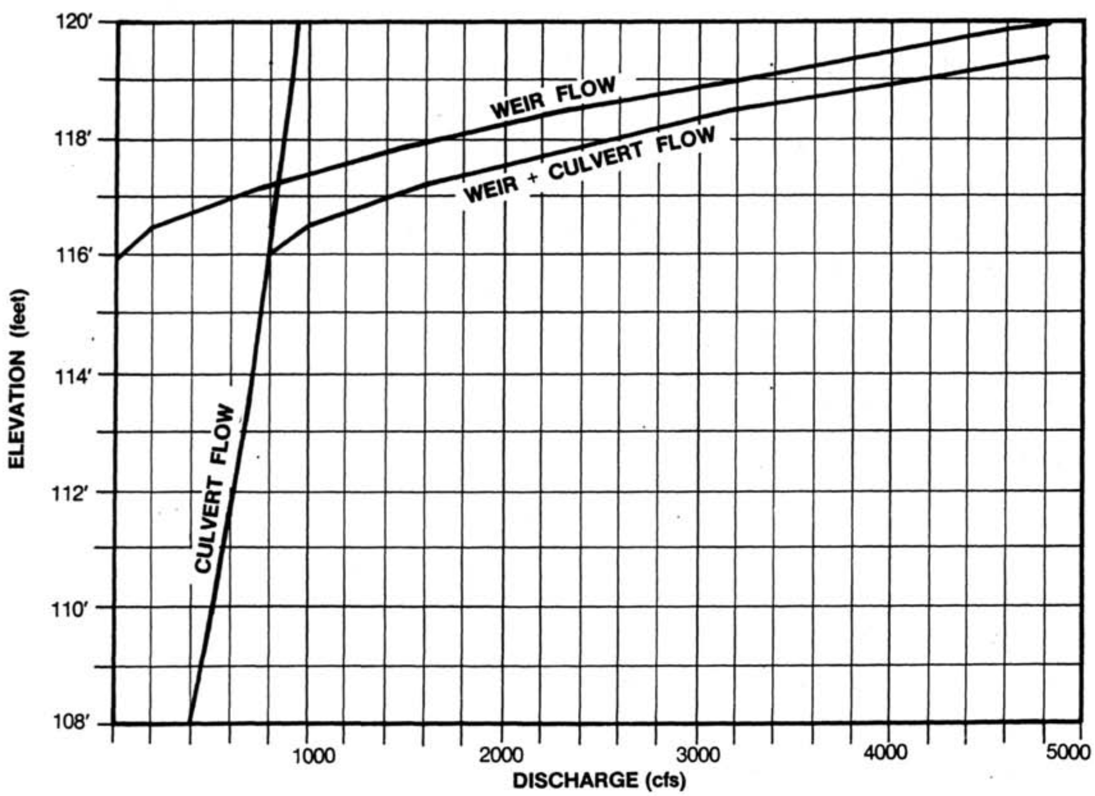


Figure 5-40
Roadway Overtopping Data for Example 5-3

CULVERT DESIGN FORM

PROJECT: EXAMPLE 5-4
CHAPTER 5, SECTION 5.3.6

STATION: 4 + 00
SHEET 1 OF 3

DESIGNER / DATE: VCA / 6/22
REVIEWER / DATE: JES / 6/22

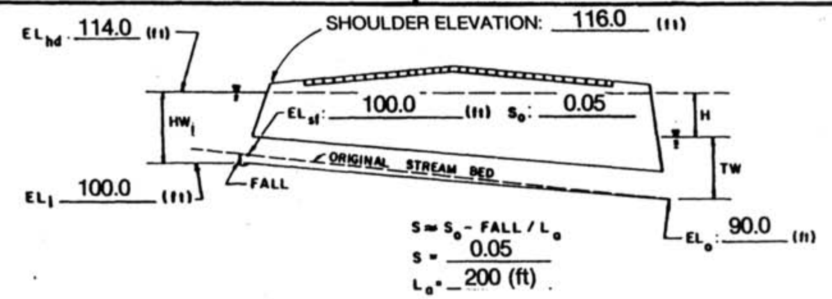
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: SCS
- ☐ DRAINAGE AREA: 400 AC ☐ STREAM SLOPE: 5.0%
- ☐ CHANNEL SHAPE: TRAPEZOIDAL
- ☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
100	1000	4.1



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Conc.-Box-7'x7'-Tapered Inlet	1000	142.9	2.0	14.0	0	114.0	4.1	≥7	7.0	7.0	0.2	10.6	107.6	114.0	40.4	Meets El_{hd}
Conc.-Box-7'x7'-Tapered Inlet	800	114.3	1.61	11.3	0	111.3	3.8	≥7	7.0	7.0	0.2	6.7	103.7	111.3	38.1	Throat Outlet
Conc.-Box-7'x7'-Tapered Inlet	900	128.6	1.84	12.9	0	112.9	4.0	≥7	7.0	7.0	0.2	8.4	105.4	112.9	39.3	Control Performance
Conc.-Box-7'x7'-Tapered Inlet	1100	157.1	2.22	15.5	0	115.5	4.3	≥7	7.0	7.0	0.2	12.7	109.7	115.5	41.3	Data

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

SIDE-TAPERED INLET WILL PASS NEW DESIGN FLOW (1000 cfs) IF ATTACHED TO EXISTING BARREL.

CULVERT BARREL SELECTED:

SIZE: 7' x 7'
SHAPE: RECTANGULAR
MATERIAL: CONCRETE n 0.012
ENTRANCE: TAPERED INLET

Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-41
Culvert Design Form
Data for Example 5-4

PROJECT: EXAMPLE 5-4 CHAPTER 5, SECTION 5.3.6	STATION: 4 + 00. SHEET 2 OF 3	TAPERED INLET DESIGN FORM DESIGNER / DATE: VSA / 6/22 REVIEWER / DATE: JS / 6/22

DESIGN DATA:

Q 100 = 1000 cfs ; EL_{hi} 114.0 ft

EL. THROAT INVERT 100.0 ft

EL. STREAM BED AT FACE 100.0 ft

FALL 0 ft TAPER 4 : 1 (4 : 1 TO 6 : 1)

STREAM SLOPE, So = 0.05 ft/ft

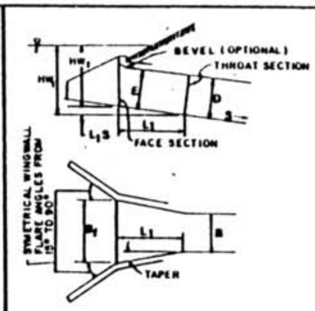
SLOPE OF BARREL, S = 0.05 ft/ft

S_f — : 1 (2:1 TO 3:1)

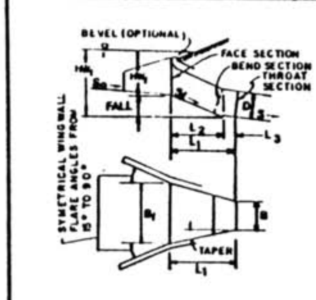
BARREL SHAPE AND MATERIAL: BOX-CONCRETE

N = 1, B = 7.0', D = 7.0'

INLET EDGE DESCRIPTION



SIDE - TAPERED



SLOPE - TAPERED

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	HW_f / E (3)	Q / B_f (4)	MIN. B_f (5)	SELECTED B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)
1000	114.0	100.0	101.0	13.0	1.86	86	11.6	12	—	—	—	—	—	10	—	—	—
1000	113.0	100.0	*100.5	12.5	1.79	83	←	12	Performance Curve Data								
800	110.7	100.0	100.5	10.2	1.45	67	←	12									
900	111.8	100.0	100.5	11.3	1.62	75	←	12									
1100	114.4	100.0	100.5	13.9	1.99	92	←	12									

(1) SIDE - TAPERED: EL. FACE INVERT = EL. THROAT INVERT + 1 ft (APPROX.)
SLOPE - TAPERED: EL. FACE INVERT = EL. STREAM BED AT FACE

(2) $HW_f = EL_{hi}$ - EL. FACE INVERT

(3) $1.1 D \geq E \geq D$: E = HEIGHT OF INLET

(4) FROM DESIGN CHARTS : FIG 5-25—5-29

(5) MIN. $B_f = Q/(Q/B_f)$

(6) MIN. $L_3 = 0.5NB$

(7) L_2 = (EL. FACE INVERT - EL. THROAT INVERT) S_f

(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_3$

(9) IF (8)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_2$

(10) IF (7)>(8), ADJ. TAPER $= (L_2 + L_3) \Big/ \left[\frac{B_f - NB}{2}\right]$

(11) SIDE - TAPERED: $L = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER
SLOPE - TAPERED: $L_1 = L_2 + L_3$

(12) $HW_c = EL_{hi}$ - EL. CREST INVERT

(13) MIN. $W = 0.35Q / HW_c^{1.5}$

*Actual Invert = $El_t + L_1 S = 100 + (10)(.05) = 100.5$ ft

SELECTED DESIGN

B_f 12.0

L_1 10.0

L_2 —

L_3 —

BEVELS ANGLE 0 °

b = 0 in; d = 0 in

TAPER 4 : 1

S_f = 0 : 1

Reference: USDOT, FHWA, HDS-5 (1985).

Figure 5-42
Tapered Inlet Design Form
Data for Example 5-4

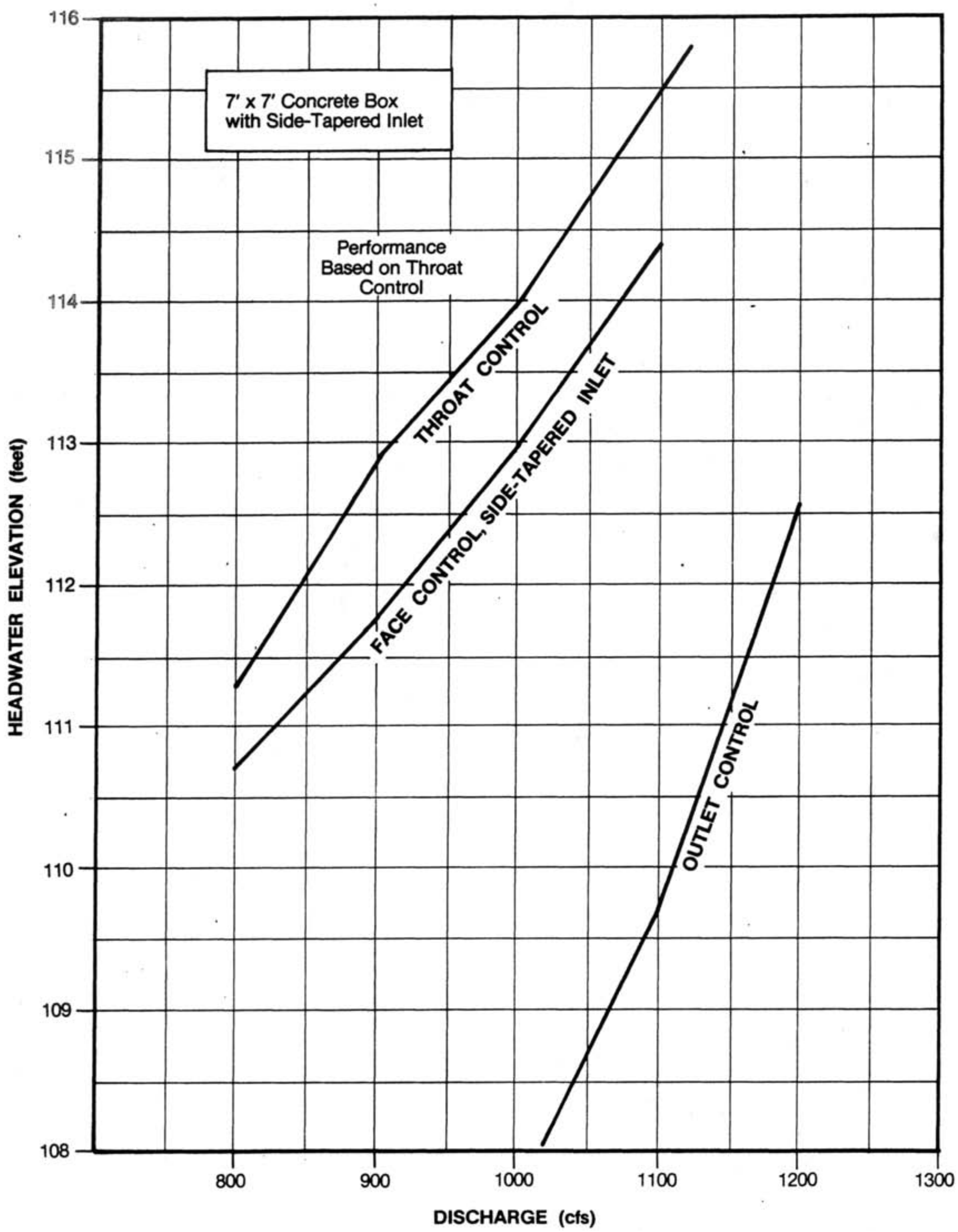


Figure 5-43
Results for Example 5-4

CHAPTER 6
STORM SEWER HYDRAULICS

Chapter 6
STORM SEWER HYDRAULICS

Synopsis

The general approach for storm sewer system design usually involves iterative sequences of system layout, hydrologic and hydraulic calculations, and outfall design. Basic criteria and procedures are presented for the design of storm sewer systems. Conditions requiring variance from these guidelines should be documented and approved by MWS.

6.1 Design Criteria

6.1.1 Return Periods

Closed conduits shall be designed for the total intercepted flow based on the design event (see Volume 1, Section 6.3.1). In general, design event return periods are as follows:

Minor Facilities	10-year
Major Facilities	100-year

Minor and major drainage facilities are defined in Volume 1.

6.1.2 Manning's n Values

Values for Manning's roughness coefficient for concrete pipe, concrete box culvert, and corrugated metal pipe (CMP) are given below:

Concrete pipes and box culverts (precast or cast-in-place)	n = 0.013
CMP (non-spiral flow, annular corrugations)	n = 0.024
CMP (full pipe spiral flow, helical corrugations)	
Sizes 15-24"	n = 0.017
Sizes 30-54"	n = 0.021
Sizes 60-96"+	n = 0.024

Additional details for selecting roughness coefficients for CMP can be obtained from FHWA-TS-80-216 (USDOT, FHWA, 1980).

Full pipe spiral flow occurs only for circular pipes longer than 20 diameters and free of sediment buildup when lining is not used. If the conditions for development of full pipe spiral flow are questionable, the conservative use of the n value for non-spiral flow is more desirable. Conditions where full spiral flow may be appropriate are down drains, detention outlet pipes, and free outlet or gravity storm sewer systems with a design velocity above 4 feet per second.

6.1.3 Slopes and Hydraulic Gradient

The standard recommended maximum and minimum slopes for storm sewers should conform to the following criteria:

1. The maximum hydraulic gradient should not produce a velocity that exceeds 20 feet per second.

2. The minimum desirable physical slope should be that which will produce a velocity of 2.5 feet per second when the storm sewer is flowing full.

Systems should generally be designed for non-pressure conditions. When hydraulic calculations do not consider minor energy losses such as expansion, contraction, bend, junction, and manhole losses (see Section 6.4.2), the elevation of the hydraulic gradient for design flood conditions should be at least 1.0 foot below ground elevation. As a general rule, minor losses should be considered when the velocity exceeds 6 feet per second (lower if flooding could cause critical problems). If all minor energy losses are accounted for, it is usually acceptable for the hydraulic gradient to reach the gutter elevation. The maximum hydraulic gradient allowed is 5 feet above the crown of the conduit (see Volume 1, Section 6.3.2).

6.1.4 Pipe Size and Length

A minimum pipe size of 15 inches is required when access spacing is 50 feet or less. When access spacing exceeds 50 feet, a minimum size of 18 inches is required. Designs should use standard pipe size increments of 6 inches for pipes larger than 18 inches.

A minimum box culvert size of 3 by 3 feet for precast units and 4 by 4 feet for cast-in-place units is recommended. Increments of 1 foot in the height or width should be used above this minimum. The span by height format is used for reporting box culvert dimensions, e.g., in the dimension 10 by 7, the span is 10 feet and the height is 7 feet.

Access spacing shall not exceed 400 feet for conduits less than 54 inches in diameter and shall not exceed 800 feet without approval from MWS. The two materials for pipes allowed within Right of Ways (or pipes that carry public water) are concrete and corrugated metal.

6.1.5 Minimum Clearances

Minimum clearances for storm sewer pipe shall comply with the following criteria:

1. A minimum of 1 foot is required between the bottom of the road base material and the outside crown of the storm sewer.

2. For utility conflicts that involve crossing a storm sewer alignment, the recommended minimum design clearance between the outside of the pipe and the outside of any conflicting utility should be 0.5 foot if the utility has been accurately located at the point of conflict. If the utility has been approximately located, the minimum design clearance should be 1 foot. Electrical transmission lines or gas mains should never come into direct contact with the storm sewer.

3. Storm sewer systems should not be placed parallel to or below existing utilities in a manner that could cause utility support problems. The recommended clearance is 2 feet extending from each side of the storm sewer and 1:1 side slopes from the trench bottom.

4. When a sanitary line or other utility must pass through a manhole, a minimum 1-foot clearance should be maintained between the bottom of the utility and the flow line of the storm main, and greater clearance is recommended. Flow will be less obstructed when the utility is placed above or as close as possible to the crown of the pipe. The head loss caused by an obstruction should be accounted for. (Note: Gas mains shall not pass through inlet and manhole structures.)

6.1.6 Inlet Location and Spacing

The location and spacing of inlets should be based on inlet capacity and width of spread calculations consistent with procedures and criteria presented in Chapter 4.

6.1.7 Easements

Easement requirements are given in Volume 1, Section 6.3.3.

6.2 General Approach

The design of storm sewer systems is usually an iterative process involving the following four steps:

1. System Layout: Selection of inlet locations and development of a preliminary plan and profile configurations consistent with design criteria in Section 6.1.

2. Hydrologic Calculations: Determination of design flow rates and volumes (see Section 6.3).

3. Hydraulic Calculations: Determination of pipe sizes required to carry design flow rates and volumes, as discussed in Section 6.4.

4. Outfall Design: Outlet protection or detention/retention may be required because of downstream constraints; see Chapter 8 for detention/retention , Chapter 10 and Volume 4 TCP-25 or PESC-07 for outlet protection.

6.3 Hydrologic Calculations

The two peak flow methods generally appropriate for hydrologic calculations for storm sewer systems are the Rational Method and the inlet hydrograph method. In general, as the time of concentration, drainage area, and variability in land use increase, more complex procedures are warranted. A rule-of -thumb is that flood hydrograph procedures should be considered when the time of concentration goes beyond the range of 30 to 45 minutes. In addition, the size and complexity of the storm sewer system should be considered. (See Chapter 2 for additional guidance on selecting hydrologic methods.)

To demonstrate the application of the peak flow methods identified above and to provide a point of comparison, the example storm sewer system layout shown in Figure 6-1 is evaluated below. Common data for calculating inlet flow rates are presented in Table 6-1.

6.3.1 Rational Method

The Rational Method, expressed in Chapter 2 as Equation 2-11, implicitly assumes that all runoff from the tributary area is intercepted by the storm sewer system. Bypass must be accounted for by adjusting the tributary drainage area. The method requires a determination of the tributary area, time of concentration, rainfall intensity, and runoff coefficient at each design point.

The time of concentration is the sum of the inlet travel time and the storm sewer travel time and must be calculated for each design point considered. Rainfall intensity is obtained from an IDF curve (see Figure 2-1), based on the time of concentration and design frequency. The runoff coefficient should be the composite factor based on tributary land use and soil conditions. Table 2-3 (see Section 2.3.1) can provide a good starting point for selecting the runoff coefficient for a 10-year return period, but other considerations should include examination of existing facilities and a comparison of historical performance with the results of design calculations, if possible.

Results of Rational Method calculations for the example storm sewer data presented in Figure 6-1 and Table 6-1 are shown in Table 6-2.

6.3.2 Inlet Hydrograph Method

The inlet hydrograph method is a simplified approach that accounts for channel storage and appears to provide better estimates of observed peak runoff rates than the Rational Method (Jens and McPherson, 1964). The following equation is used to route intercepted flow for each upstream inlet to the design point:

$$Q_o = Q_i \left(\frac{2T}{2T + 0.8\frac{L}{v}} \right) \tag{6-1}$$

where:

Q_o = Outflow peak runoff rate at the design point, in cfs

Q_i = Intercepted flow peak runoff rate, in cfs

T = inlet travel time, in minutes

L = Length of storm sewer, in feet

v = Average velocity for storm sewer flow, in feet/minute

Having calculated the peak outflow, Q_o, for intercepted flow from each inlet, Q_i, the composite peak flow at the design point is obtained by summing the ordinates of triangular hydrographs for each inlet. This summation is accomplished graphically by drawing triangular hydrographs for the outflow from each inlet with a peak of Q_o, a rising limb time of T +0.8 (L/v), and a recession time of T. This procedure is illustrated in Figure 6-2, which also illustrates the inflow hydrograph with a peak flow rate equal to the inlet intercept and a time base of 2T. By plotting triangular outflow hydrographs for each inlet tributary to the design point on the same scale, the composite hydrograph can be developed by summing hydrograph ordinates. Dividers are helpful for accomplishing this summation.

Additional information on the use of the inlet hydrograph method can be found in publications by Jens and McPherson (1964) and Kaltenbach (1963).

Results of inlet hydrograph calculations for the example storm sewer data presented in Figure 6-1 and Table 6-1 are shown in Table 6-3. Graphical development of peak flows for each storm sewer segment is shown in Figures 6-3 through 6-6.

6.3.3 Example Comparison

Peak flow calculations from the two methods for the example storm sewer system in Figure 6-1 are compared in Table 6-4. The inlet hydrograph method consistently gave the lowest peak flow results, with the results obtained by the Rational Method corresponding closely. Pipe segments between inlets and manholes are not compared in Table 6-4 because each method produced the same results (because 100 percent intercept was assumed).

The design flow for the pipe between the first two manholes, M_1-M_2, did not vary a great deal between methods. Beginning with pipe segment M_2–M_3, the inlet hydrograph method has lower results. The final pipe segment, M_4-0, had a 13 percent reduction in peak flow rate for the inlet hydrograph method, as compared to the Rational Method.

6.4 Hydraulic Calculations

Hydraulic calculations are used to size conduits to handle the design flows determined from hydrologic calculations (see Section 6.3). The hydraulic capacity of a storm sewer conduit can be calculated for the two types of conditions typically referred to as gravity and pressure flow. Hydraulic procedures provided in this section represent a summary of information from publications by Brater and King (1976), Chow (1959), the American Society of Civil Engineers (1969), the University of Missouri (1958), and the American Iron and Steel Institute (1980). These publications should be consulted if additional details are required.

6.4.1 Pressure Versus Gravity Flow

Guidance is presented in Figure 6-7 for determining whether pressure or gravity flow conditions occur in a storm sewer system. In general, if the hydraulic grade line is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the hydraulic grade line is below the crown of the pipe, gravity flow calculations are appropriate. Storm sewer systems should generally be designed as gravity systems (see Volume 1, Section 6.3.2).

For storm sewers designed to operate under pressure flow conditions, inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on gravity conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system.

Existence of the desired flow condition should be verified for design conditions. Storm sewer systems can alternate between pressure and gravity flow conditions from one section to another.

The discharge point of the storm sewer system usually establishes a starting point for evaluating the condition of flow. If the discharge is submerged, as when the water level of the receiving waters are above the crown of the storm sewer, the exit loss should be added to the water level and calculations for head loss in the storm sewer system started from this point, as illustrated in Figure 6-7. If the hydraulic grade line is above the pipe crown at the next upstream manhole, pressure flow calculations are indicated; if it is below the pipe crown, then gravity flow calculations should be used at the upstream manhole.

When the discharge point is not submerged, a flow depth should be determined at a known control section to establish a starting elevation. As illustrated in Figure 6-7, the hydraulic grade line is then projected from the starting elevation to the upstream manhole. Pressure flow calculations may be used at the manhole if the hydraulic grade is above the pipe crown.

The assumption of straight hydraulic grade lines, as shown in Figure 6-7, is not entirely correct, since backwater and drawdown conditions can exist, but is generally reasonable. It is also usually appropriate to assume the hydraulic grade calculations begin at the crown of the outlet pipe for simple non-submerged systems. If additional accuracy is needed, as with very large conduits or where the result can have a significant effect on design, backwater and drawdown curves should be developed.

6.4.2 Energy Losses

The following energy losses should be considered for storm sewer systems:

1. Friction
2. Entrance
3. Exit

Additional energy loss parameters should be evaluated for complex or critical systems. The following losses are especially important when failure to handle the design flood has the potential to flood offsite areas:

1. Expansion
2. Contraction
3. Bend
4. Junction and manhole

<u>Friction Loss</u>

The energy loss required to overcome friction caused by conduit roughness is generally calculated as:

$$H_f = \left(\frac{29n^2 L}{R^{1.33}} \right) \frac{v^2}{2g} \qquad (6\text{-}2)$$

where:

H_f = Energy loss due to friction, in feet

n = Manning's roughness coefficient

L = Conduit length, in feet

R = Hydraulic radius of conduit, in feet

v = Average velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second2

Entrance, Exit, Expansion, Contraction, and Bend Losses

These head losses due to pipe form conditions are generally calculated as:

$$H_L = K\frac{v^2}{2g} \qquad (6\text{-}3)$$

where:

H_L = Head loss due to pipe form conditions, in feet

K = Loss coefficient for pipe form conditions

v = Average velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second2

The loss coefficient, K, is different for each category of pipe form loss and should be based on operating characteristics of the specific system. Values for the entrance loss coefficient are the same as those developed for culverts (see Chapter 5). Expansion and contraction loss coefficients for circular pipes can be selected based on data from Brater and King (1976) presented in Tables 6-5 and 6-6.

The bend loss coefficient for storm sewer systems can be evaluated using Figure 6-8, which provides various relationships between the angle of a bend and the loss coefficient. Relationships are presented for bends at manholes with and without deflectors, and for curved drain alignments with r/D values equal to 2 and greater than or equal to 6.

Junction and Manhole Losses

Losses associated with junctions and manholes should be evaluated with the procedures reported by the University of Missouri (1958). Although details of the procedures are not given in this manual, the application of important results is discussed below and head loss coefficients for typical manholes and junctions are presented in Table 6-7.

For straight flow-through conditions, the University of Missouri (1958) indicates that pipes should be positioned vertically between the limits of inverts aligned or crowns aligned. An offset in the plan is allowable, provided that the projected area of the smaller pipe falls within that of the larger. It is probably most effective to align the pipe inverts, as the manhole bottom will then support the bottom of the jet issuing from the upstream pipe.

When two laterals intersect at a manhole, pipes should not be oppositely aligned, since the jets could impinge upon each other. If directly opposing laterals are necessary, the installation of a deflector (as shown in Figure 6-9) will significantly reduce losses. The research conducted on this type of deflector is limited to the ratios of outlet pipe to lateral pipe diameters equal to 1.25. In addition, lateral pipes should be located such that their centerlines are separated laterally by at least the sum of the two lateral pipe diameters.

Jets from upstream and lateral pipes must be considered when attempting to shape the inside of manholes. Results reported by the University of Missouri (1958) for pressurized pipe flow conditions indicate that very little, if anything, is gained by shaping the bottom of a manhole to conform to the pipe invert. Shaping the manhole bottom to match the pipe invert may even be detrimental when pressurized laterals flowing full are involved, as the shaping tends to deflect the jet upwards, causing unnecessary head loss. Limited shaping of the manhole bottom for open channel flow conditions is required.

Figure 6-9 depicts several types of deflectors that can be efficient in reducing losses at junctions and bends for full flow conditions. In all cases, the bottoms are flat or only slightly rounded (to handle low flows). As a contrast, several inefficient manhole shapes are shown in Figure 6-10. Several of these inefficient devices would appear to be improvements, indicating that special shapings deviating from those in Figure 6-9 should be used with caution.

6.4.3 Gravity Flow

The capacity of storm sewers designed to operate under gravity flow conditions should be sized using the following form of Manning's Equation:

$$v = \frac{0.592}{n} D^{2/3} S^{1/2} \qquad (6\text{-}4)$$

$$Q = vA \qquad (6\text{-}5)$$

$$Q = \frac{0.465}{n} D^{8/3} S^{1/2} \qquad (6\text{-}6)$$

where:

Q = Design flow rate, in cfs

v = Average velocity of flow, in feet/second

n = Manning's roughness coefficient

D = Pipe diameter, in feet

A = Cross-sectional area, in square feet

S= Slope of the energy gradient, in feet/foot

Storm sewer capacity calculations based on Manning's Equation can be accomplished using Figures 6-11, 6-12, and 6-13 as discussed below or procedures published by Brater and King (1976), the American Concrete Pipe Association (1978 and 1980), Chow (1959), and the American Iron and Steel Institute (1980).

<u>Nomograph</u>

The following steps are used for solving Manning's Equation using the circular pipe nomograph in Figure 6-11:

1. Determine input data, including slope in feet/foot, Manning's n value, and pipe diameter in inches or feet.

2. Connect a line from the slope scale, Point 1, to the Manning's n scale, Point 2, and note the point of intersection on the turning line, Point 3.

3. Connect a line from the pipe diameter, Point 5, to the point of intersection obtained in Step 2, Point 3.

4. Extend the line from Step 3 to the discharge and velocity scales to read the discharge at Point 4 and the velocity at Point 6.

Partial Flow Charts

For partial flow in a circular pipe. Figures 6-12 and 6-13 can be used for capacity and velocity calculations as follows:

1. Determine input data including design discharge, Q, Manning's n value, pipe diameter, D, and channel slope, S.

2. Calculate the circular pipe conveyance factor using the equation:

$$K_p = \frac{Qn}{D^{8/3} S^{1/2}} \qquad (6\text{-}7)$$

where:

K_p = Circular pipe open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 6.1.2)

D = Pipe diameter, in ft

S = Slope of the energy grade line, in feet/foot

3. Enter the x-axis of Figure 6-12 with the value of K_p calculated in Step 2 and run a line vertically to the curve.

4. From the point of intersection obtained in Step 3, run a horizontal line to the y-axis and read a value of the normal depth of flow over the pipe diameter, d/D.

5. Multiply the d/D value from Step 4 by the pipe diameter, D, to obtain the normal depth of flow.

6. Enter the y-axis of Figure 6-13 with the d/D value from Step 4 and run a line horizontally to the curve.

7. From the point of intersection obtained in Step 6, run a line vertically downward and read a value of k_v, which equals $vn/D^{2/3}\ S^{1/2}$, from the x-axis.

8. Calculate the average velocity by the equation:

$$v = \frac{K_v D^{2/3} S^{1/2}}{n} \qquad (6\text{-}8)$$

where:

v = Average velocity, in feet/second

k_v = Pipe velocity factor from Figure 6-13 (Step 7)

D = Pipe diameter, in feet

S = Slope of the energy grade line, in feet/foot

n = Manning's roughness coefficient (see Section 6.1.2)

6.4.4 Pressure Flow

The capacity of storm sewers designed to operate under pressure flow conditions can be sized using inlet and outlet control nomographs developed for the evaluation of culverts (see Chapter 5). A more general procedure involves the application of the Energy Equation, which can be developed to consider unsteady flow conditions.

The capacity of storm sewers flowing full can be evaluated by considering velocity head, pipe form, and friction losses, expressed as:

$$H = H_v + H_L + H_f \qquad (6\text{-}9)$$

or

$$H = \left[1 + K_L + \frac{29n^2 L}{R^{1.33}}\right]\frac{v^2}{2g} \qquad (6\text{-}10)$$

where:

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, in feet

H_v = Velocity head, in feet

H_L = Head loss due to pipe form conditions, in feet

H_f = Head loss due to friction, in feet

K_L = Loss coefficient for pipe form losses

n = Manning’s roughness coefficient

L = Length of storm sewer segment, in feet

R = Hydraulic radius, in feet

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

If H can be determined, the storm sewer capacity is calculated by rearranging Equation 6-8 as follows:

$$v = \left[2gH \div \left(1 + K_L + \frac{29n^2L}{R^{1.33}} \right) \right]^{1/2} \tag{6-11}$$

or

$$Q = A\left[2gH \div \left(1 + K_L + \frac{29n^2L}{R^{1.33}} \right) \right]^{1/2} \tag{6-12}$$

where:

v = Average velocity of flow, in feet/second

Q = Storm sewer capacity, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, in feet

K_L = Loss coefficient for pipe form losses

n = Manning's roughness coefficient

L = Length of storm sewer segment, in feet

R = Hydraulic radius, in feet

The determination of H will generally involve an evaluation of energy losses to establish the hydraulic and energy gradients. Since the velocity is a required input to energy loss calculations, an iterative trial and error procedure is generally required.

6.5 Construction and Maintenance Considerations

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities. Maintenance concerns of storm sewer system design focus on adequate physical access for cleaning and repair. Volume 4 CP-18 and 20 should be considered as a part of the design process.

Table 6-1
DATA FOR DEMONSTRATING THE APPLICATION OF STORM SEWER HYDROLOGIC METHODS

Inlet [a]	Drainage Area (acres)	Time of Concentration (minutes)	Rainfall [b] Intensity (inches/hr)	Runoff Coefficient	Inlet Flow Rate [c] (cfs)
1	2.0	8.0	6.4	.9	11.5
2	3.0	10.0	6.1	.9	16.5
3	2.5	9.0	6.2	.9	14.0
4	2.5	9.0	6.2	.9	14.0
5	2.0	8.0	6.4	.9	11.5
6	2.5	9.0	6.2	.9	14.0
7	2.0	8.0	6.4	.9	11.5

[a] Inlet and storm sewer system configuration are shown in Figure 6-1.

[b] Data for example calculations only. See Chapter 2 for Nashville IDF data.

[c] Calculated using the Rational Equation (see Chapter 2).

Table 6-2
RESULTS OF RATIONAL METHOD CALCULATIONS FOR THE
HYPOTHETICAL STORM SEWER SYSTEM IN FIGURE 6-1

Storm Sewer Segment	Tributary Area [a] (acres)	Time of Concentration [b] (minutes)	Rainfall Intensity [c] (inches/hr)	Runoff Coefficient	Design Flow Rate (cfs)
$I_1 - M_1$	2.0	8.0	6.4	.9	11.5
$I_2 - M_1$	3.0	10.0	6.1	.9	16.5
$M_1 - M_2$	5.0	10.5	6.0	.9	27.0
$I_3 - M_2$	2.5	9.0	6.2	.9	14.0
$I_4 - M_2$	2.5	9.0	6.1	.9	13.7
$M_2 - M_3$	10.0	11.5	5.7	.9	51.3
$I_5 - M_3$	2.0	8.0	6.4	.9	11.5
$I_6 - M_3$	2.5	9.0	6.2	.9	14.0
$M_3 - M_4$	14.5	13.5	5.4	.9	70.5
$I_7 - M_4$	2.0	8.0	6.4	.9	11.5
$M_4 - O$	16.5	14.7	5.2	.9	77.2

[a] Tributary area data are presented in Table 6-1.

[b] See Figure 6-1 for details.

[c] Data for example calculations only. See Chapter 2 for Nashville IDF data.

Table 6-3
RESULTS OF INLET HYDROGRAPH CALCULATIONS FOR THE HYPOTHETICAL STORM SEWER SYSTEM IN FIGURE 6-1

A. Segment $M_1 - M_2$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T + 0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	0.2	0.99	11.5	11.4
2	20	0.5	0.98	16.5	16.2

Inflow to Segment $M_1 - M_2$ = 24.5 cfs (from Figure 6-3)

B. Segment $M_2 - M_3$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T + 0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	1.2	0.94	11.5	10.8
2	20	1.5	0.94	16.5	15.5
3	18	0.2	0.99	14.0	13.9
4	18	0.5	0.98	14.0	13.7

Inflow to $M_2 - M_3$ = 50.0 cfs (from Figure 6-4)

C. Segment $M_3 - M_4$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T + 0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	3.2	0.86	11.5	9.9
2	20	3.5	0.88	16.5	14.5
3	18	2.2	0.91	14.0	12.7
4	18	2.5	0.90	14.0	12.6
5	16	0.2	0.99	11.5	11.4
6	18	0.5	0.98	14.0	13.7

Inflow to $M_3 - M_4$ = 65.5 cfs (from Figure 6-5)

Table 6-3
(Continued)

D. Segment $M_4 – O$

Inlet	2T (minutes)	L/v (minutes)	$\frac{2T}{2T+0.8\frac{L}{v}}$	Q_i (cfs)	Q_o (cfs)
1	16	4.4	0.82	11.5	9.4
2	20	4.7	0.84	16.5	13.9
3	18	3.2	0.88	14.0	12.3
4	18	3.7	0.86	14.0	12.0
5	16	1.4	0.93	11.5	10.7
6	18	1.7	0.93	14.0	13.0
7	16	0.5	0.98	11.5	11.3

Inflow to $M_4 – 0$ = 67.0 cfs (from Figure 6-6)

Note: Q_i values are calculated in Table 6-1.

$$Q_o = Q_i\left(\frac{2T}{2T+0.8\frac{L}{v}}\right) \quad Equation\,6-1$$

Table 6-4
COMPARISON OF HYDROLOGIC METHODS FOR THE HYPOTHETICAL
STORM SEWER SYSTEM IN FIGURE 6-1

Storm Sewer [a] Segment	Rational Method [b] (cfs)	Inlet Hydrograph [c] (cfs)
$M_1 – M_2$	27.0	24.5
$M_2 – M_3$	51.3	50.0
$M_3 – M_4$	70.5	65.5
$M_4 – O$	77.2	67.0

[a] Storm sewer configuration is shown in Figure 6-1.

[b] Results obtained from Table 6-2.

[c] Results obtained from Table 6-3.

Table 6-5
VALUES OF K_2 FOR DETERMINING LOSS OF HEAD DUE TO
SUDDEN EXPANSION IN PIPES, FROM THE FORMULA
$H_2 = K_2 (V_1{}^2/2g)$

d_2/d_1 = Ratio of larger pipe to smaller pipe diameter

v_1 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_1 (feet/second)												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Reference: Brater and King (1976).

Table 6-6
VALUES OF K_3 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN CONTRACTION IN PIPES, FROM THE FORMULA
$H_3 = K_3\,(V_2^{\,2}/2g)$

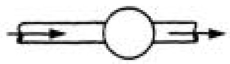
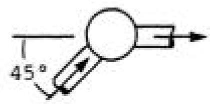
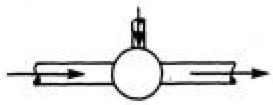
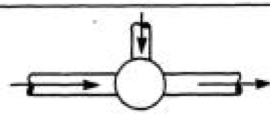
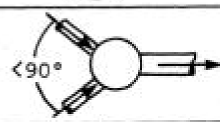
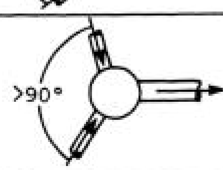
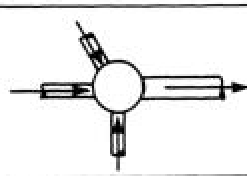
d_2/d_1 = Ratio of larger pipe to smaller pipe diameter

v_2 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_2 (feet/second)												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Reference: Brater and King (1976).

Table 6-7
HEAD LOSS COEFFICIENTS FOR MANHOLES/JUNCTIONS

Single Pipe Junctions		
Type of Manhole/Junction		**Head Loss Coefficient (K)**
Trunkline only with no bend at junction		0.5
Trunkline only with 45° bend at junction		0.6
Trunkline only with 90° bend at junction		0.8
Multiple Pipe Junctions		
Type of Manhole/Junction		**Head Loss Coefficient (K)**
Trunkline with one small lateral		0.6
Trunkline with one large lateral		0.7
Two roughly equivalent entrance lines with angle of <90° between lines		0.8
Two roughly equivalent entrance lines with angle of >90° between lines		0.9
Three or more entrance lines		1.0

Reference: Golding (1987).

Note: Above values of K are to be used to estimate energy or head losses through surcharged junctions/manholes in pressure flow portions of a storm sewer system. The energy loss equation is

$$h_j(ft) = K \frac{[v(ft/sec)]^2}{64.4}$$

with v = larger velocity in main entrance or exit line of junction/manhole.

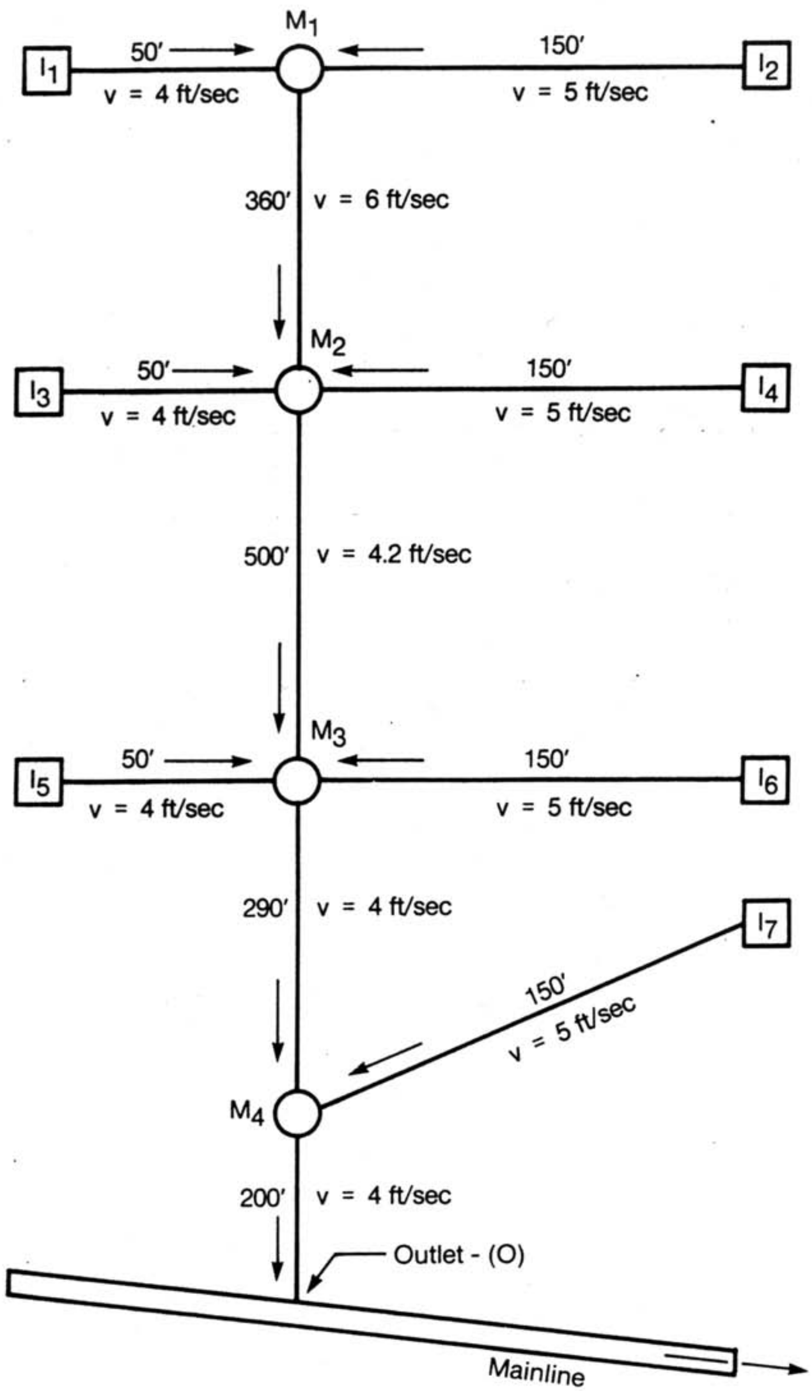


Figure 6-1
Hypothetical Storm Sewer System Layout
For Demonstrating Hydrologic Calculations

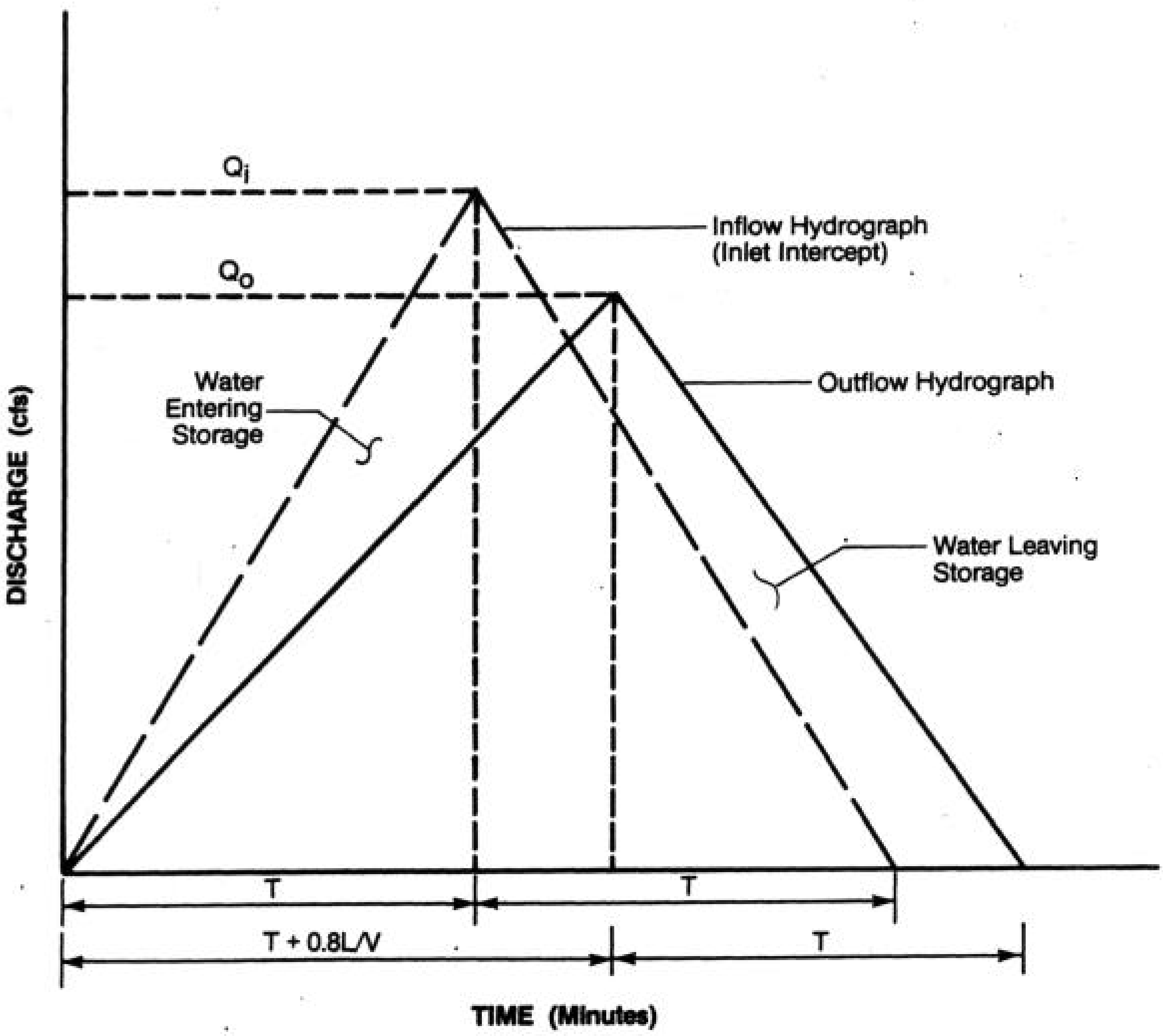


Figure 6-2
Triangular Approximation of Inlet Hydrographs

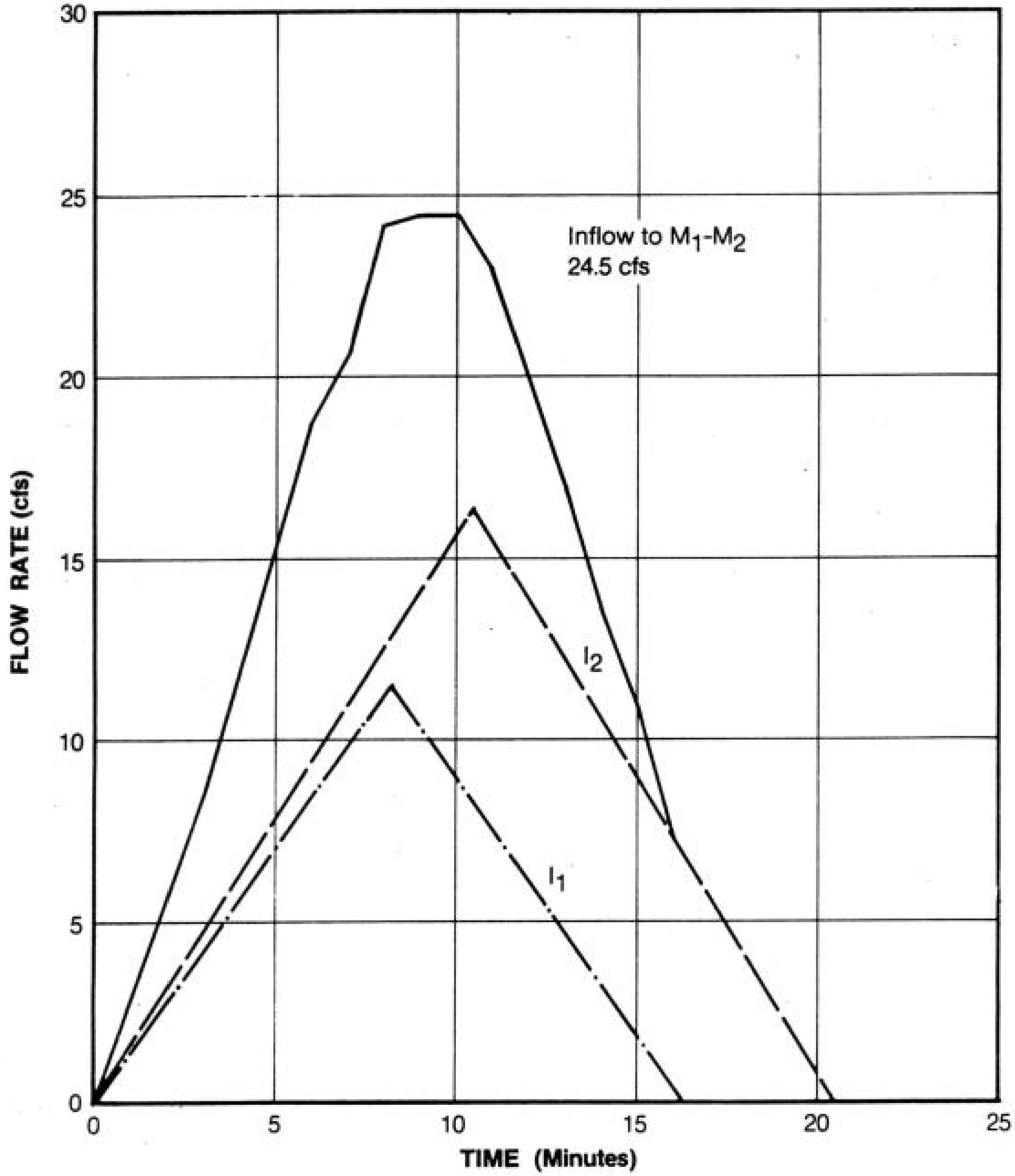


Figure 6-3
Inlet Hydrograph Results for Segment M_1-M_2
of the Hypothetical System in Figure 6-1

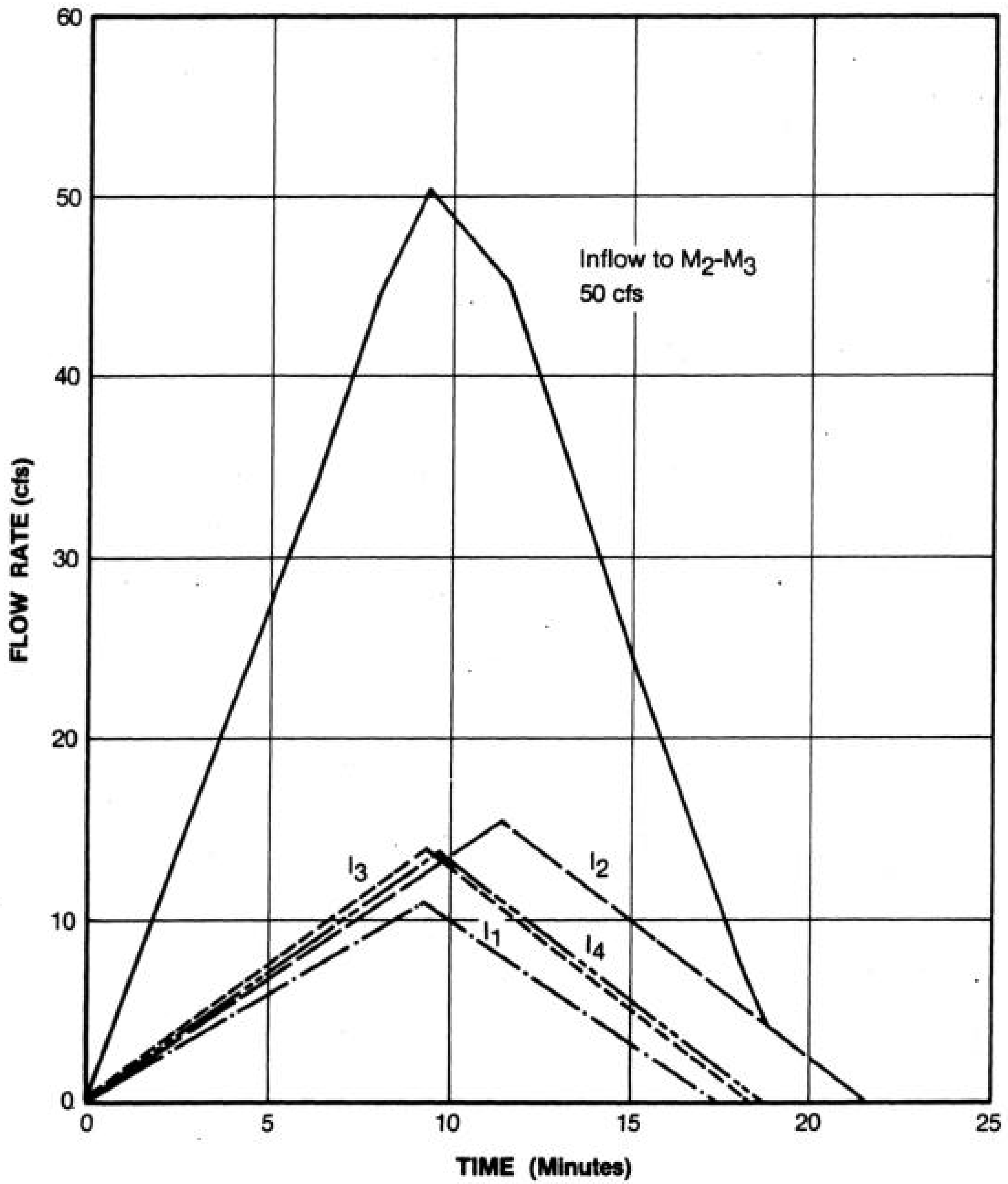


Figure 6-4
Inlet Hydrograph Results for Segment M_2-M_3
of the Hypothetical System in Figure 6-1

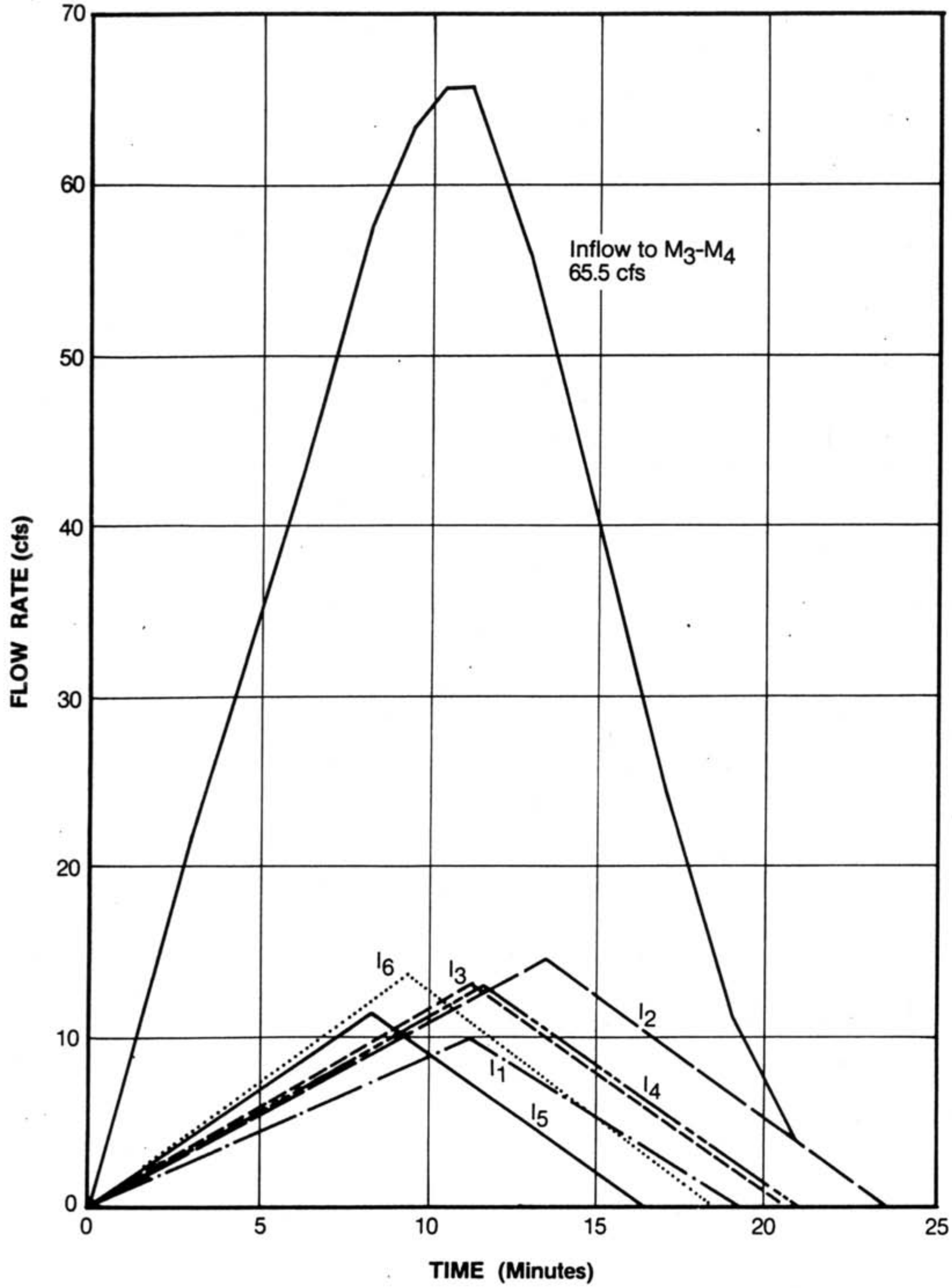


Figure 6-5
Inlet Hydrograph Results for Segment M_3-M_4
of the Hypothetical System in Figure 6-1

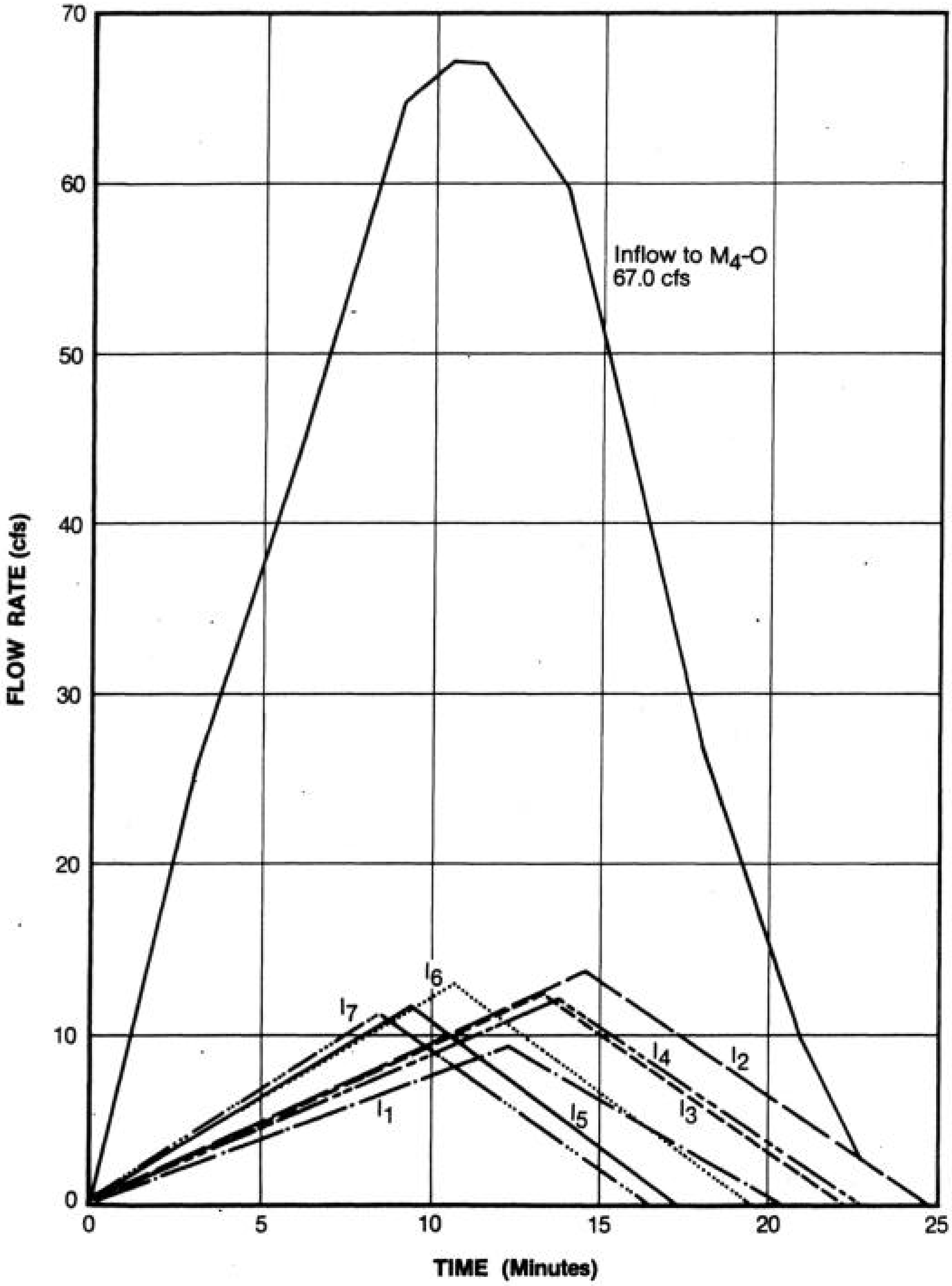
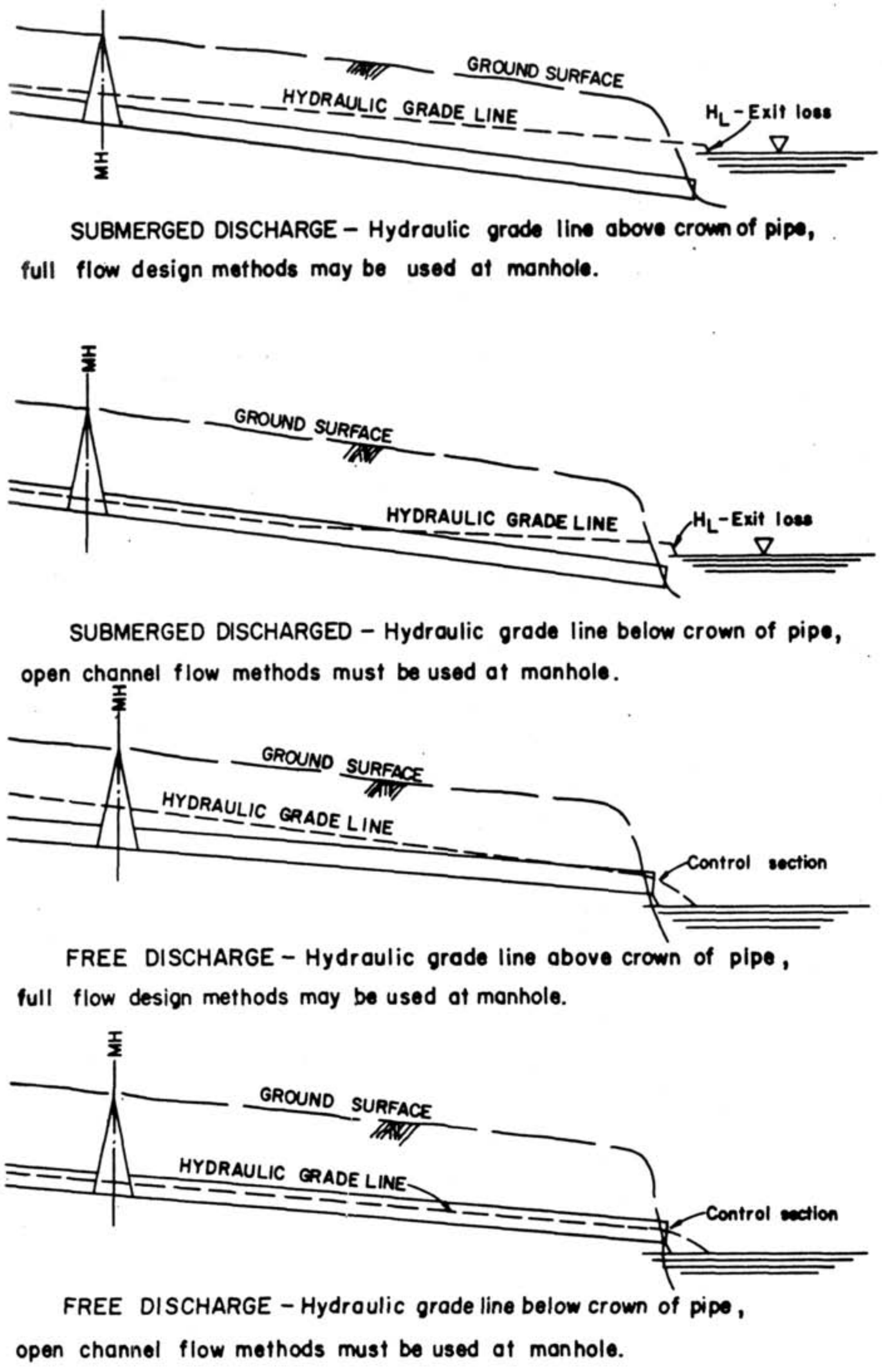
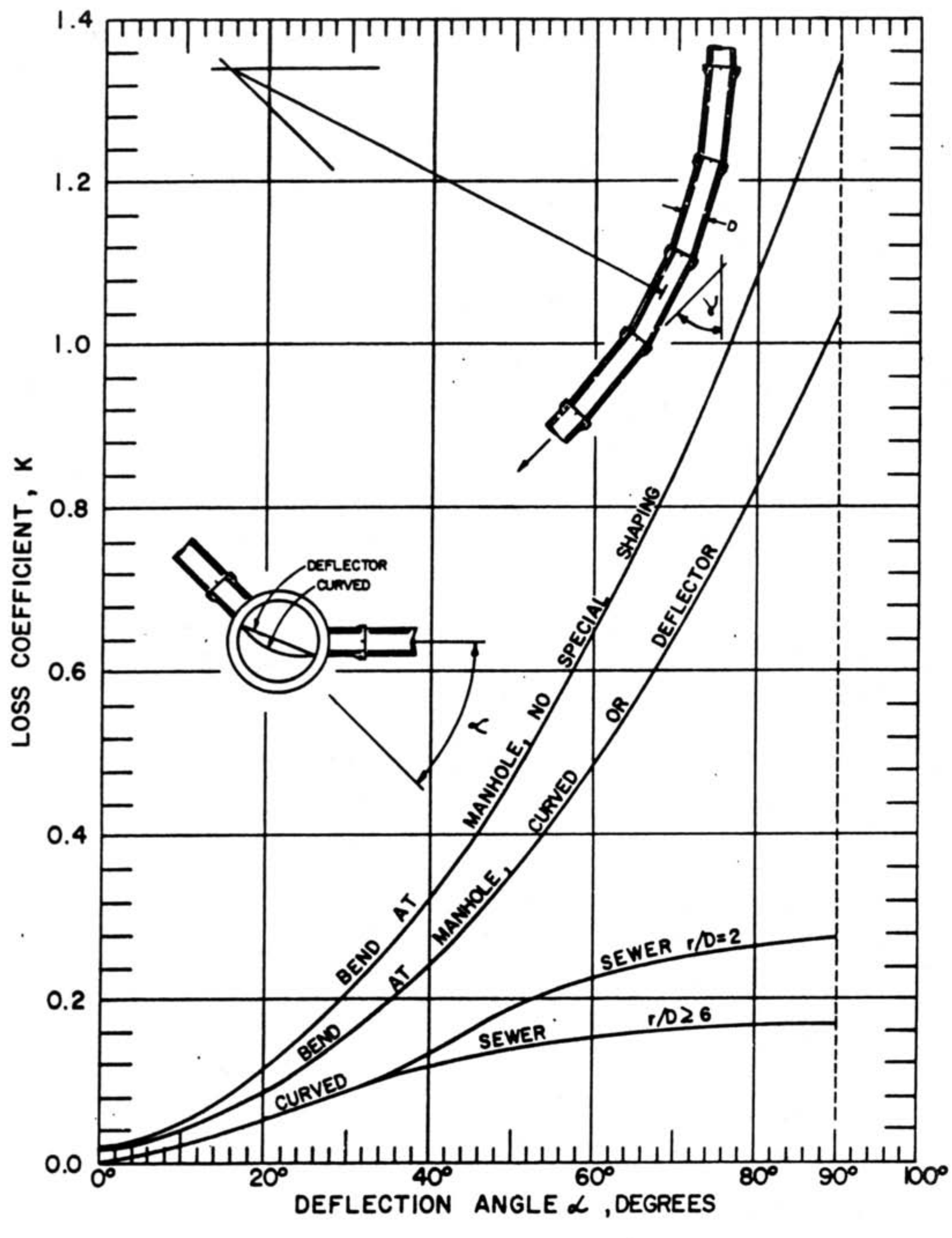


Figure 6-6
Inlet Hydrograph Results for Segment M_4-O
of the Hypothetical System in Figure 6-1



Reference: Wright-McLaughlin Engineers (1969).

Figure 6-7
Determination of Pressure vs. Open Channel Flow Conditions
in Storm Sewer Systems



Reference: Wright-McLaughlin Engineers (1969).

Figure 6-8
Storm Sewer Bend Loss Coefficient

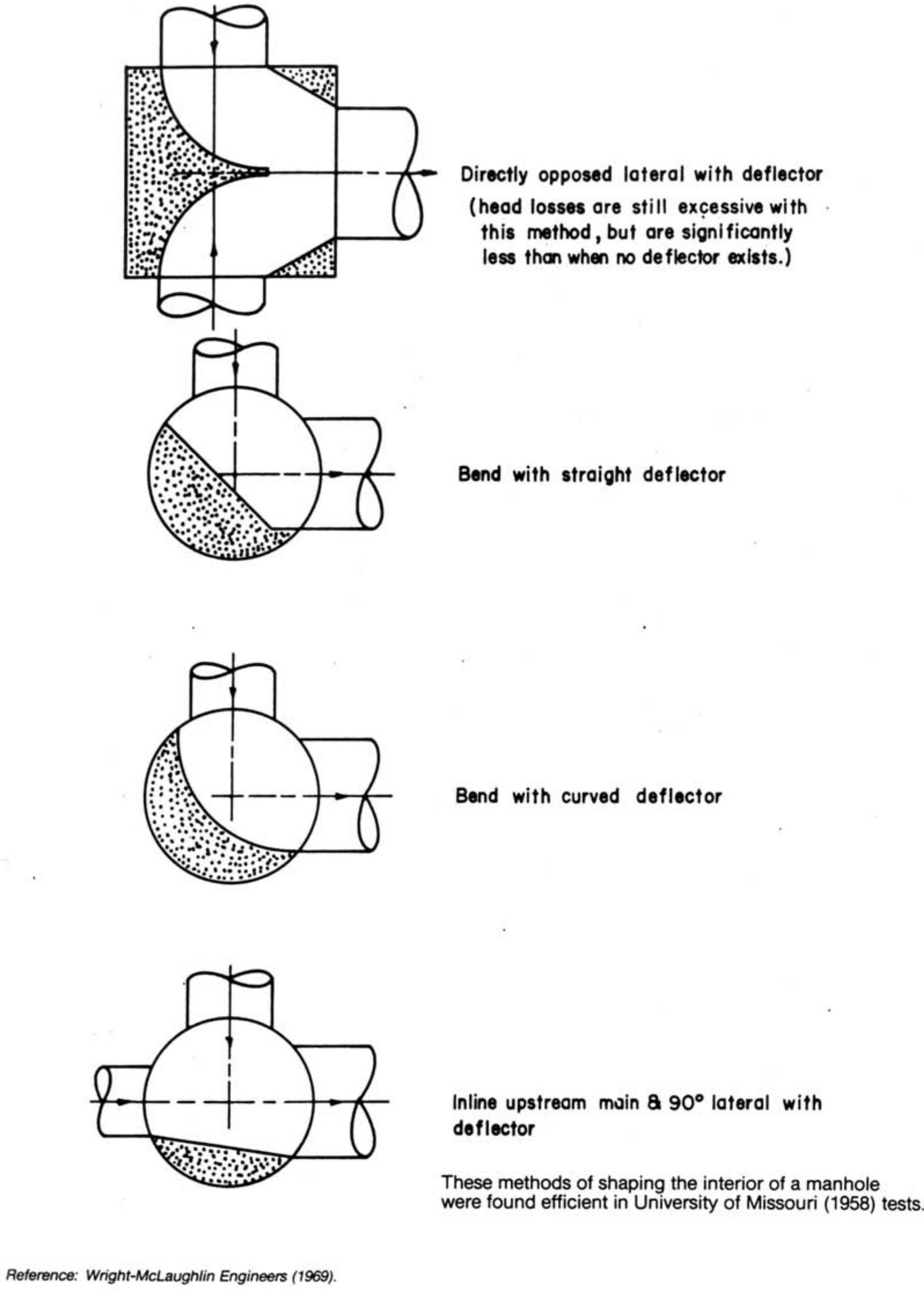
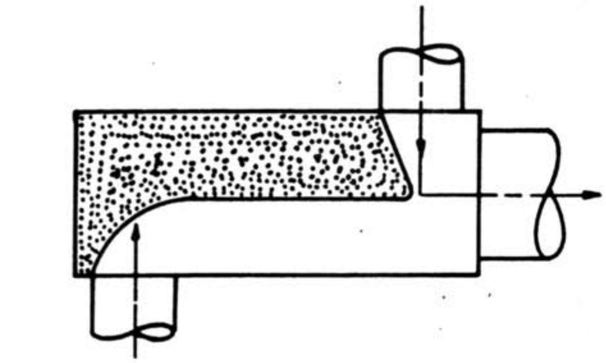
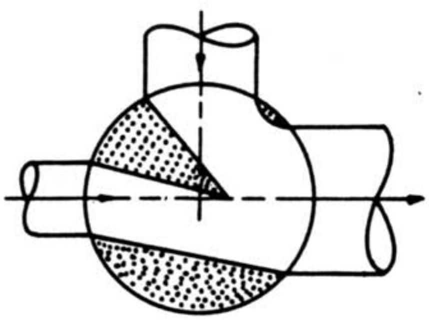


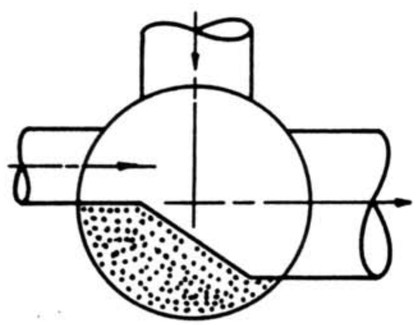
Figure 6-9
Efficient Manhole Shaping



Offset lateral with deflector



Inline upstream main & 90° lateral with divider

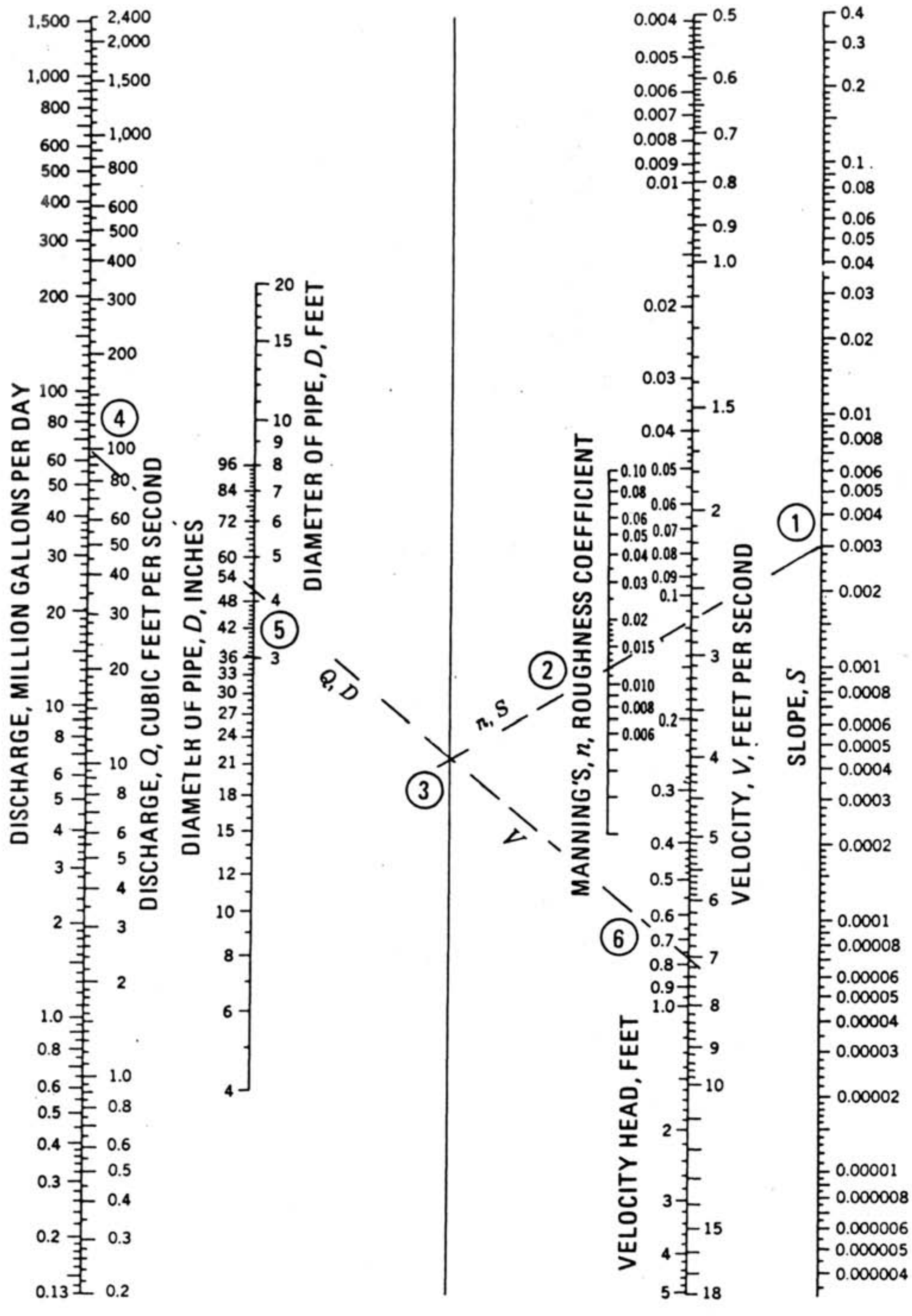


Inline upstream main & 90° lateral with deflector

These methods of shaping the interior of a manhole were found inefficient in University of Missouri (1958) tests, either due to increased head loss or tendency to plug with trash.

Reference: Wright-McLaughlin Engineers (1969).

Figure 6-10
Inefficient Manhole Shaping



CIRCULAR PIPE

Reference: American Concrete Pipe Association (1980).

Figure 6-11
Circular Pipe Nomograph for Solving Manning's Equation

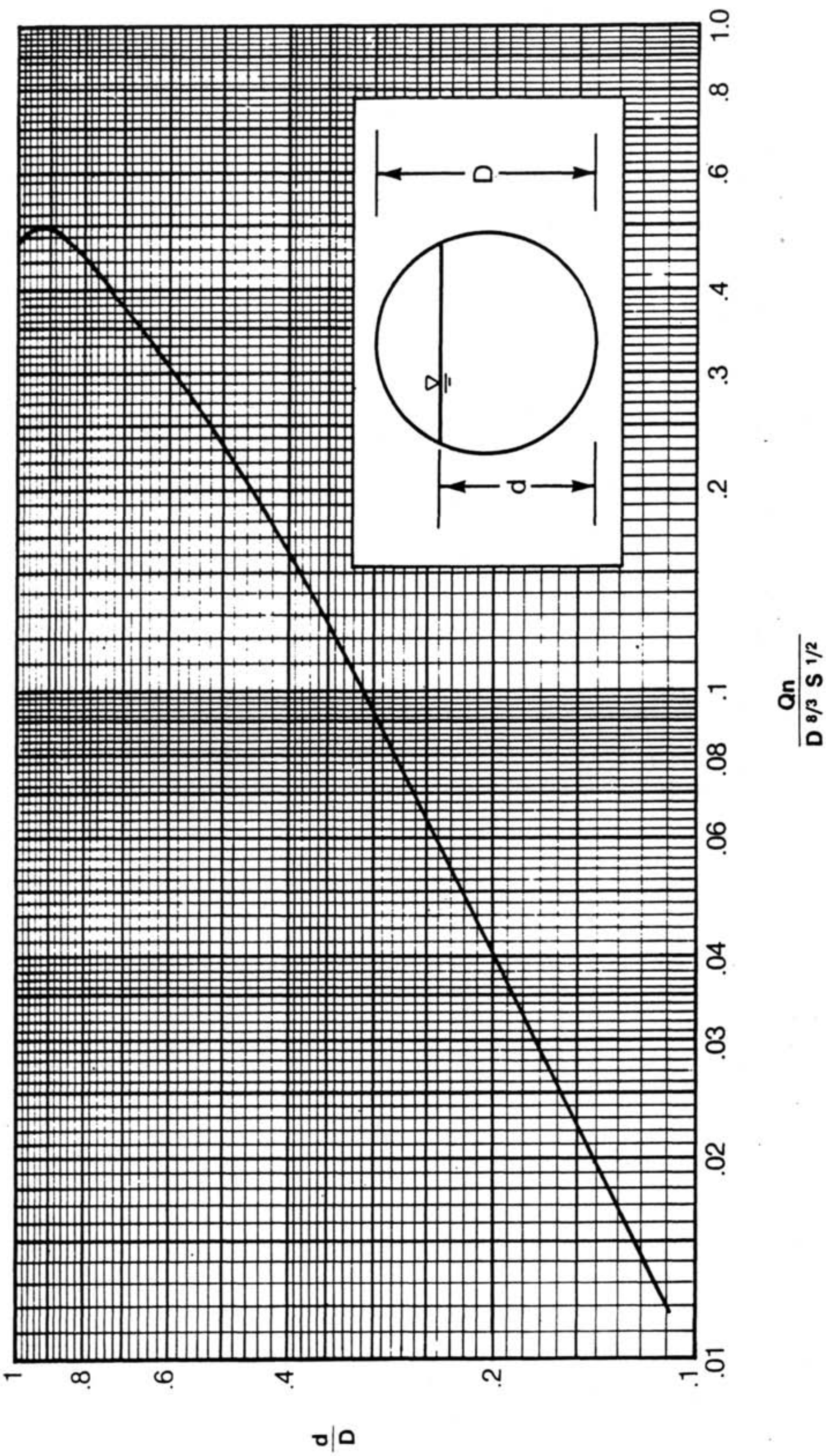


Figure 6-12
Circular Pipe Partial Flow Capacity Chart

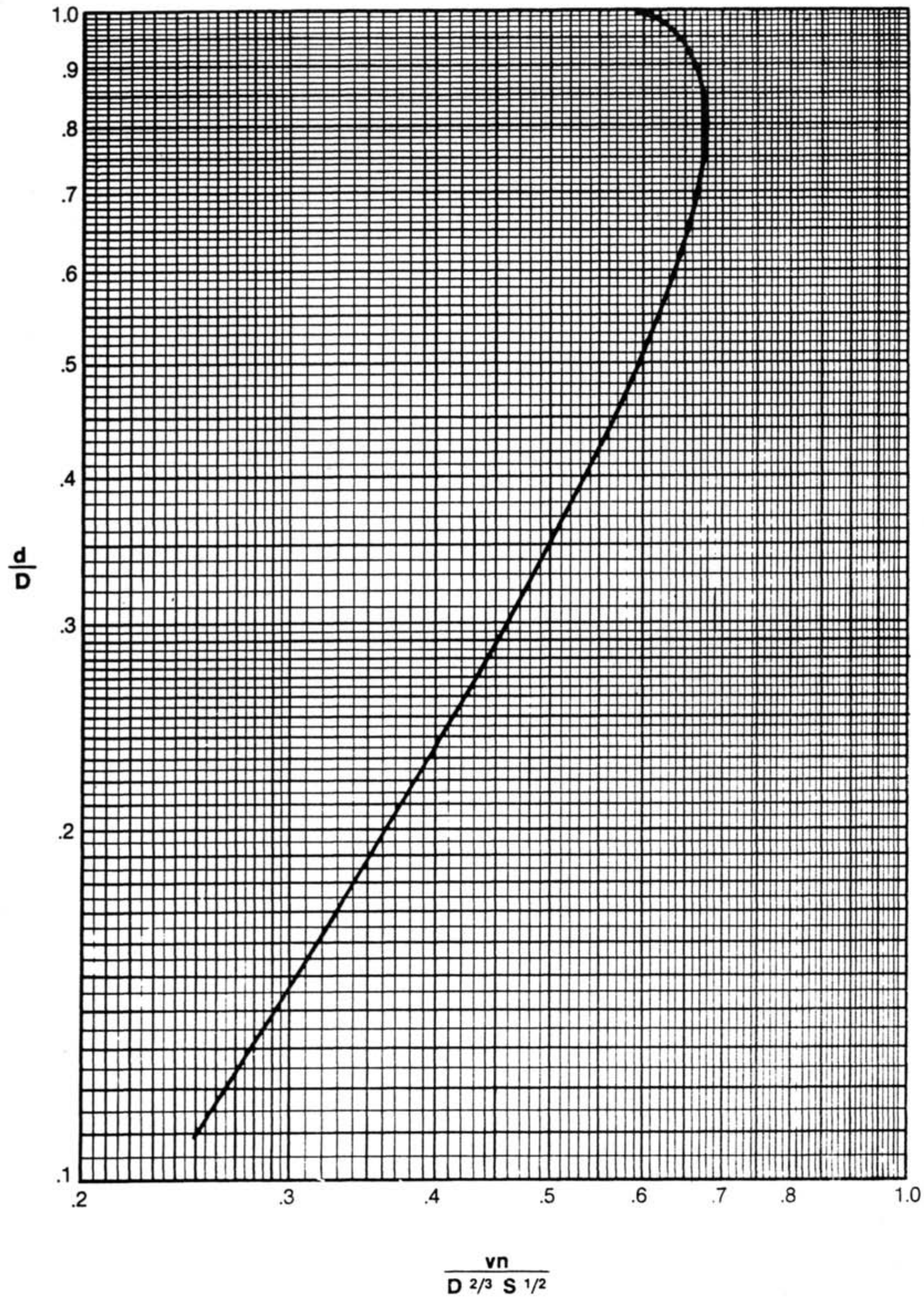


Figure 6-13
Circular Pipe Partial Flow Velocity Chart

CHAPTER 7
BRIDGE HYDRAULICS

Chapter 7
BRIDGE HYDRAULICS

Synopsis

General design criteria for frequency, high water, clearances, average velocity, relief openings, spur dikes, and structural design and alterations are presented, as well as procedures for water surface profile calculations. Key references for information on bridge hydraulics are the WSPRO (HY-7) computer program research report (USDOT, FHWA, 1986), the HEC-RAS computer user's manual, hydraulic reference manual and applications guide (U.S. Army Corps of Engineers, 1998), and HDS-1 (USDOT, FHWA, 1978).

7.1 Design Criteria

Each bridge requires development of site-specific design criteria that will meet the needs of the specific crossing. Standard bridge design criteria are presented below.

7.1.1 Return Period

The peak discharge design return period for bridges with spans of 20 feet or greater, as specified in Volume 1, Section 6.7, shall be the 100-year storm event. The design shall comply with flood plain/floodway encroachment criteria from Volume 1. Because the risks and requirements for each bridge are unique, site-specific factors may affect the selection of an appropriate design return period.

7.1.2 High Water

The design high water at a bridge location establishes the minimum elevation for the approach embankments, which ensures the integrity of the roadway base and pavement. Design high water is not necessarily the highest water of record or the water surface elevation of the design flood, but instead is determined by good judgment based on frequency, duration, area drained, and the outfall condition. Arriving at an elevation to which the water has risen in the past can be helpful, but determining the frequency and duration associated with that observed high water is often difficult.

7.1.3 Clearances

Low member and horizontal bridge clearance requirements should consider the site-specific potential for plugging by debris and the need for passage of boat traffic. The method for establishing design values should be clearly documented.

A method used by the St. Paul District of the U.S. Army Corps of Engineers (1985) to evaluate plugging potential is included in Table 7-1. The Table leads the designer through a consideration of various aspects of bridge plugging to select a high, medium, or low potential for plugging. After selecting the plugging potential, bridges are designed with area reduction percentages as follows:

1. Low potential—5 to 10 percent

2. Medium potential—15 to 20 percent

3. High potential—25 to 30 percent

In general, a 50 percent area reduction would be an absolute maximum to be considered. The design engineer is ultimately responsible for making a proper evaluation that considers site-specific conditions.

<u>Vertical Clearance</u>

Unless a regulatory agency has established higher values, the following minimum vertical clearances are recommended:

1. To allow debris to pass without causing damage, recommended minimum clearance between design flood stage and the low member of the bridge should be:

    a. Interstate highways—3 feet

    b. High use or essential highways--2 to 3 feet

    c. Other highways—1 to 2 feet

2. For crossings subject to small boat traffic, recommended minimum clearance should be:

    a. Rivers and streams--6-foot clearance above mean annual flood stage

    b. Across lakes or canals--6-foot clearance above prevailing water elevation

<u>Horizontal Clearance</u>

Horizontal clearance should be adequate to minimize encroachment and adverse backwater conditions caused by a flow constriction (see Section 7.2). If costs are not substantially higher, bridges are preferred to multi-barreled culverts. Except for perpendicular crossings, horizontal clearance does not equal span distance. The span is measured center to center from piers, while horizontal clearance is a projected area that varies with the angle of the crossing.

The projected length is generally taken as the perpendicular distance between the closest bridge substructure or the fender system.

7.1.4 Average Velocity

The average velocity is generally considered when the capacity of a bridge opening is being evaluated; localized velocities may become significant, however, if the potential for scour exists. The applicability of using an average velocity diminishes when significant differences occur across the flow area in terms of roughness and flow depth. Consideration should be given to the use of riprap or bank protection on fill slopes when the maximum allowable velocities specified in Chapter 3 are exceeded for the soil type and conditions encountered.

7.1.5 Relief Openings

When the flow distribution in the approach channel at flood stage is over a broad area (in unconfined or flood plain channels) and the placement of approach embankments will cause extensive encroachment, consideration should be given to the use of relief openings in addition to the main channel bridge. The relief openings are usually located at a less defined channel away from the main channel. This secondary channel is often at an elevation higher than the normal flow in the main channel.

Relief openings can reduce scour at the main bridge and can reduce backwater. They should be designed to carry a specific discharge, if possible, since they are susceptible to scour hazards. These openings should be located and designed so they will not "invite" the main river to flow through them and thereby leave the main opening to convey less than the planned amount of flood. Procedures for evaluating relief opening requirements are described in Chapter 4 of the WSPRO (HY-7) computer program documentation (USDOT, FHWA, 1986).

7.1.6 Spur Dikes

Most bridge abutments in Davidson County are set in bedrock. However, where approach embankments encroach on wide flood plains and constrict the normal flow, special attention should be given to scour in the vicinity of bridge abutments. A typical spur dike, as shown in Figure 7-1, provides a structural method for reducing the gradient and velocity along the embankment by moving the mixing action of the merging flow away from the abutment to the upstream end of the dike. Before a spur dike is selected as a bridge component, regulatory constraints on fill in flood plains should be considered.

The three principal considerations for proportioning a spur dike are shape, height, and length. A dike shaped in the form of a quarter of an ellipse, with a ratio of the major (length) to the minor (offset) axis of 2.5:1, is recommended (USDOT, FHWA, HDS-1, 1978). The spur dike height should be based on the design high water level. It should have sufficient height and freeboard to avoid overtopping and be protected from wave action. Unless dikes are constructed entirely of stone or earth dikes are properly armored with graded stone facing, they can be severely damaged or completely destroyed by overtopping.

The length of a spur dike can be determined using procedures presented in HDS-1 (USDOT, FHWA, 1978). In general, the length of a spur dike should be increased with an increase in flood plain discharge, with an increase in velocity under the bridge, or with both. At the recommended minimum length of 100 feet or more, curvilinear flow is directed around the end of the dike, to merge with the main channel flow and establish a straight course downriver before reaching the bridge abutment.

7.1.7 Structural Design

Bridges are to be designed in accordance with the latest edition of AASHTO Specifications for Highway Bridges. The bridge shall be designed to resist the hydraulic force produced by a 100-year storm.

7.1.8 Structural Alterations

Under some conditions, existing bridges may be retained or modified (widened or lengthened) when a roadway is upgraded. When such alterations are designed, the level of effort should be consistent with that required for a new structure. When a bridge is being widened, special attention should be placed on evaluating vertical clearance, new pier losses, and deck drainage.

7.2 Water Surface Profile Calculations

The procedure for performing water surface profile calculations at bridges should be consistent with the needs of the project. When changes to elevations and regulatory floodways presented on a Flood Insurance Rate Map or Floodway Maps are evaluated, consistent procedures should be used. This generally involves using the program and values of the original Flood Study to evaluate changes for approval.

Two commonly used water surface profile computer programs are HEC-RAS and SWMM EXTRAN block. The HEC-RAS computer program, developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (1998), and SWMM 4.3 was developed by the U.S. Environmental Protection Agency (Huber, 1992; Roesner 1994) and is well suited for performing the water surface profile computations associated with most bridges. They are recommended for most projects, particularly if bridge hydraulic requirements are not significant.

7.3 Construction and Maintenance Considerations

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities. Typical problems encountered with bridges include excessive scour at the entrance toe of a main channel embankment, collection of debris and sedimentation in one or more bridge openings under low or normal flow conditions, and improper handling of bridge deck runoff on the overbank area of a channel. As repairing many problems at bridges can require a lengthy regulatory process, these problems should be considered during design to minimize maintenance requirements.

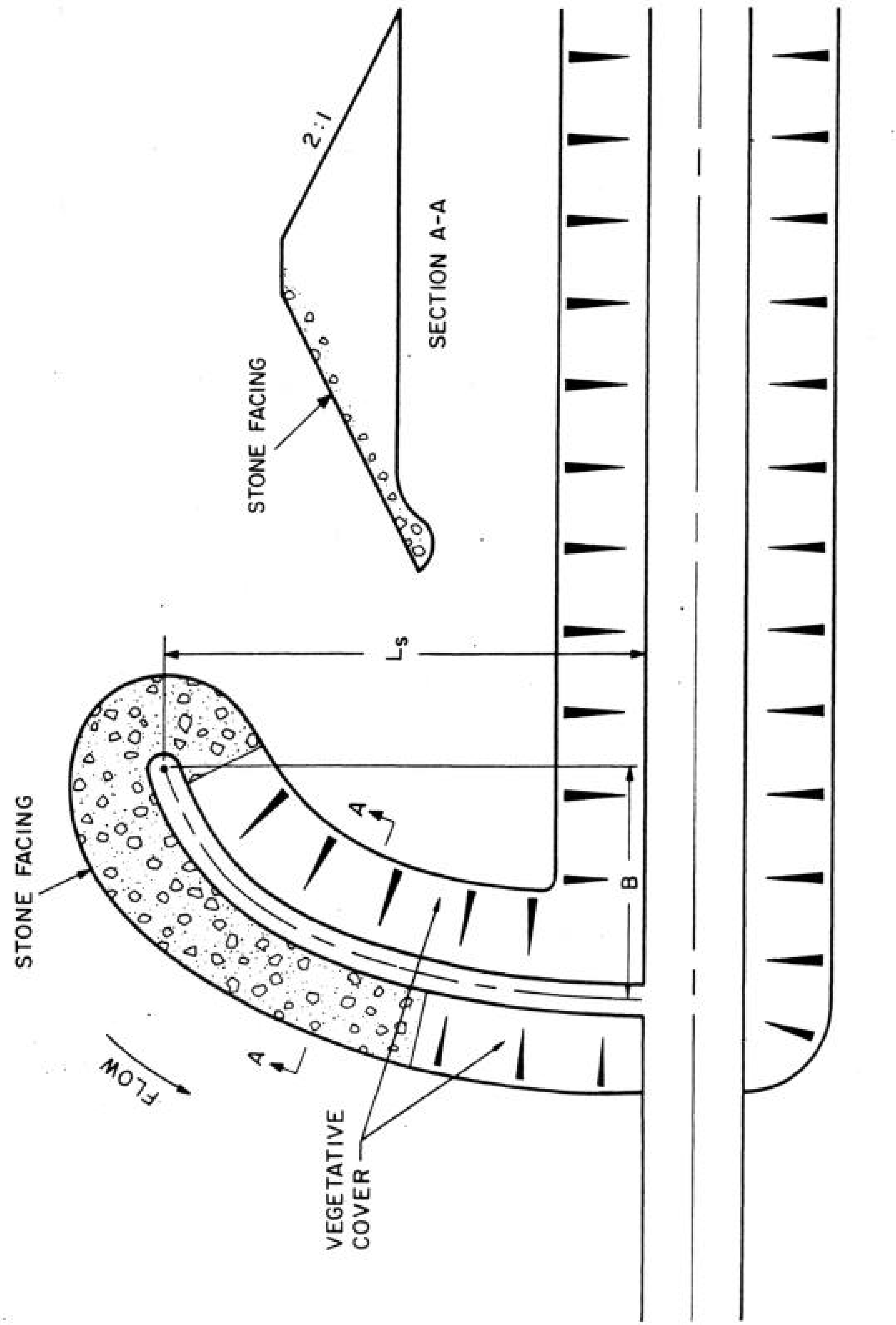
Table 7-1
EXAMPLE TABULATION SHEET FOR EVALUATING BRIDGE PLUGGING POTENTIAL

No.	Item	Associated Level of Risk: Low	Associated Level of Risk: High	Risk Level Used: Low	Risk Level Used: Medium	Risk Level Used: High
1.	Distance between piers or abutments	>100 ft	<10 ft	______	______	______
2.	Number of openings greater than 30-100 feet	>2	1	______	______	______
3.	Low chord elevation clearance to water	>3 ft	<3 ft	______	______	______
4.	Depth of flow below low chord member	>20 ft	<10 ft	______	______	______
5.	Bridge is perched and overbanks carry flows	Yes	No	______	______	______
6.	Potential source of debris	Little Debris	Heavy Debris	______	______	______
7.	Ability of channel to transport debris through bridge	1 to 4 fps	>5 fps	______	______	______
8.	Upstream structures such as bridges that can prevent debris from being transported	1 or more	None	______	______	______
9.	Past history of plugging or experiences with similar structures under similar circumstances	None	Some	______	______	______
10.	Potential for actions to remove the debris using cranes, etc.	Great	Slight	______	______	______
11.	Type of stream with regard to the rate of rise	Slow	Fast	______	______	______
12.	Percentage of the flow area obstructed by the bridge deck	<5%	>10%	______	______	______
13.	Impact upstream of bridge if plugging occurred	Minor Damage	Heavy Damage, Loss of Life	______	______	______

Other Considerations:

Recommendations:

Reference: U.S. Army Corps of Engineers, St. Paul District, Engineering Division (June 1985).



Reference: USDOT, FHWA, HDS-1 (1978).

Figure 7-1
Plan and Cross Section of Spur Dike

CHAPTER 8
DETENTION / RETENTION HYDRAULICS

Chapter 8
DETENTION/RETENTION HYDRAULICS

Synopsis

Land development activities often alter the hydrologic characteristics of a watershed, which may in turn affect both the timing, velocity, magnitude, and quality of runoff. Stormwater detention/retention to address quantity and quality is required by Nashville Regulations (see Volume 1, Sections 2.4, 2.5, 6.1, 6.6, 6.7) to control and mitigate adverse impacts caused by development. A BMP Treatment Train approach to provide a series of controls is discussed in Volume 4 Section 1. Detention and retention systems are a component in the BMP Treatment Train for a given site.

This chapter provides general design criteria for detention/retention basins as well as procedures for performing preliminary sizing and final reservoir routing calculations. The Storage Indication Method is presented as an acceptable method for detention routing calculations. Land-locked and flood prone drainage areas will require retention/detention storage areas designed to address special conditions to protect public safety from flooding, facility failure, and/or sinkholes. Wet detention facilities with a permanent pool also require special design considerations. This chapter is supported and augmented by Volume 4 Section 6 Permanent Treatment Practices (PTP).

8.1 Design Criteria

In general, detention facilities offer temporary storage accompanied by controlled release of the stored water, while, retention refers to stormwater storage without access to a positive outlet. Some detention and retention facilities may have a permanent pool of water below the outlet elevation; in such cases, water budget calculations are required and should consider average annual, dry season and wet season conditions. Retention facilities shall be implemented with special considerations. These systems require infiltration to recover storage. Additional information about infiltration systems is presented in Volume 4 PTP-14.

The design considerations and criteria for detention/retention facilities should include:

1. Multiple systems and Best Management Practices (BMPs)
2. Release rates for multiple design storms
3. Detention volumes (“live” pool and permanent pool if applicable)
4. Grading, depth, and geometry requirements
5. Outlet structure(s)

Additional selection, design, sizing, and maintenance criteria are presented in Volume 4 Section PTP.

8.1.1 Multiple Systems and Best Management Practices (BMPs)

Detention and retention can be used separately or together in series or parallel with other stormwater BMPs to offer cumulative benefit to stormwater quantity and/or quality. Selecting a series of practices is discussed in Volume 4 Section 1.6, including how to use the BMP Treatment Train to select facility types. Volume 4 PTP-08 discusses multiple systems in additional detail.

8.1.2 Release Rates

Outlet structure release rates should approximate pre-development peak runoff rates for the water quality volume, 2-year through 100-year 24-hour duration storms. The procedures for calculating the live pool for a wet and dry detention pond are presented in Volume 4 PTP-01 and 06, respectively. The release rate for a dry (without permanent pool) detention pond designed to benefit stormwater quality is 24 to 48 hours of the "live" pool volume and 24 to 60 hours for a wet (with permanent pool) detention pond. The live pool contains the water quality volume is dependant upon land use characteristics and average annual runoff capture percentage of at least 85 to 95% or 0.50 to 1.25-inches per acre depending on runoff coefficients. The residence time for a permanent pool in a wet detention pond should be at least two to four weeks.

Design calculations are required to demonstrate that the 2 and 100-year design storms are controlled and that water release rates are designed to facilitate at least a 85 to 95% average annual runoff capture percentage. If these storm events are managed, intermediate storm return periods can be assumed to be adequately managed. V-notch and multistage outlet structures are preferred. V-notch weirs are discussed in Section 8.4.5 and Volume 4 PTP-01 and 06. Multi-stage control structures, such as the one shown in Figure 8-1, are required to provide the proper drawdown time of the water quality volume and control the 2- and 100-year storms.

8.1.3 Detention Volume

Detention volume shall be adequate to attenuate the post-development peak discharge rates to allowable rates determined for Section 8.1.2. Routing calculations shall be consistent with procedures in Section 8.6. Facilities that are to be used as temporary (construction phase) sediment control management practices shall have the excavated detention volume oversized to account for the anticipated amount of sediment to be trapped. If siltation during construction is in excess of sedimentation estimates then the permanent detention volume, design dimensions shall be restored before as-built certification is submitted. Furthermore, as discussed in Volume

4 PTP-02 and 03, the detention volume should be oversized to allow for long-term (5to10 years) sediment storage.

8.1.4 Grading and Depth

The construction of detention/retention facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments should be less than 10 feet in height and should have side slopes no steeper than 3:1 (horizontal to vertical). Embankments may be higher with special considerations presented in Section 8.1.5. Stormwater quality facilities with a littoral zone should be graded at a 6:1 (horizontal to vertical) slope in those areas. The remainder of the grading should be no steeper than 4:1 (horizontal to vertical). Riprap-protected embankments should be no steeper than 2:1. Geo-technical slope stability analysis is recommended for embankments greater than 3 feet in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks, including those by Spangler and Handy (1982) and Sowers and Sowers (1970).

The shape of the facility is important for water quality treatment. The pond should be designed to minimize short-circuiting by including energy dissipaters on inlets that are placed as far away from the outlet structure as possible. The facility should have a shape with at least a 3:1 (preferably up to a 7:1) length to width ratio. If topography or aesthetics require the pond to have an irregular shape then the pond area and volume should be increase to compensate for the dead (very low flow) spaces, but this volume can be considered for stormwater quantity management.

Areas above the normal high water elevation of detention/retention facilities should be sloped at a minimum of 5 percent toward the facilities to allow drainage and to prevent standing water except in areas design control that flow such as landscape swales and biofilters.

The bottom area of dry detention/retention facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2 percent bottom slope is recommended. Concrete lined low flow or pilot channels constructed across the facility bottom from the inlet to the outlet are not preferred. Low flows should be distributed evenly into sheet flow across the bottom of the facility. It should be noted that it is preferred that detention/retention facilities should be designed as off-line structures, where possible, to improve treatment efficiencies. On-line facilities are acceptable depending on pond geometry and specific site characteristics.

The maximum depth of stormwater detention/retention facilities will normally be determined during the permitting process. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of attached weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A maximum depth of 6 to 12 feet is generally reasonable. Aeration may be required in permanent pools deeper than 12 feet to prevent thermal stratification and that could result in anaerobic conditions and odor problems.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements, and required freeboard. Aesthetically pleasing features are also important. A minimum freeboard of 1 foot above the 100-year design storm high water elevation should be provided for impoundment depths of less than 20 feet. No freeboard is required for underground detention systems. Impoundment depths greater than 20 feet are subject to the requirements of the State Dam Safety Act (see Section 8.1.5). Additional guidance is provided in Volume 4 PTP-01 and 06.

8.1.5 Outlet Structure

Outlet structures selected for detention/retention facilities should typically include a principal outlet riser or weir and an emergency overflow and must be able to accomplish the necessary functions of the facility. Outlet structures can take the form of drop inlets or any combination of pipes, weirs, and orifices. The principal outlet is intended to convey the water quality volume and quantity design storm without allowing flow to enter an emergency outlet. Selecting a magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The sizing of a particular outlet structure should be based on results of hydrologic routing calculations (see Section 8.6), consistent with criteria in Sections 8.1.2, 8.1.3, and 8.1.4.

8.1.6 State Dam Safety Program

National responsibility for the promotion and coordination of dam safety lies with the Federal Emergency Management Agency (FEMA). State responsibility for administration of the Tennessee Safe Dams Act of 1973 (T.C.A. 70-2501) in coordination with the provisions of the Federal Dam Safety Act of 1983 (PL 99-662) is assigned to the Department of Health and Environment. Rules and regulations relating to applicable dams are promulgated by this department.

Under these regulations, a dam is an artificial barrier that does or may impound water and that is 20 feet or greater in height or has a maximum storage volume of 30 acre-feet or more. A number of exemptions are allowed from the Safe Dams Act and the applicable state office should be contacted to resolve questions.

Dams are classified as either new or existing, by hazard potential, and by size. Hazard potential categories are listed below:

1. Category 1 dams are located where failure would probably result in any of the following: loss of human life, excessive economic loss due to damage of downstream properties, public hazard, or public inconvenience due to loss of impoundment and/or damage to roads or any public or private utilities.

2. Category 2 dams are located where failure may damage downstream private or public property, but such damage would be relatively minor and within the general financial capabilities of the dam owner. Public hazard or inconvenience due to loss of roads or any public or private utilities would be minor and of short duration. Chances of loss of human life would be possible but remote.

3. Category 3 dams are located where failure may damage uninhabitable structures or land but such damage would probably be confined to the dam owner's property. No loss of human life would be expected.

Size categories are listed in Table 8-1.

Detailed engineering requirements are given in the regulations for new dams. Existing dams constructed of earth embankment must be stable and protected from erosion. For all dams, the 6-hour design storm is specified in Table 8-2. Applicable regulations should be consulted for further details and engineering requirements.

8.2 General Water Quality Procedures

Procedures for designing stormwater quality detention are presented in Volume 4 PTP-01 and 06. However, it should be noted that many stormwater quantity facilities can be designed to account for stormwater quality by modifying the outlet structure, some minor grading and shape changes, and inclusion of a sediment and floatable debris removal forebay or other BMPs. Other information on new or retrofit stormwater quality retention/detention facility design computations and other considerations can be found in Hartigan (1988), Roesner et al. (1998), Hasty, McCormick and Schmidt (1999) . The remained of this chapter will discuss stormwater quantity procedures and computations Volume 4 Sections 1 and 6 (PTP) should be referenced for detail about stormwater quality procedures and computations.

8.3 General Water Quantity Procedures

The following three relationships should be considered when sizing a stormwater detention facility:

1. Inflow hydrographs for a range of design storms (see Chapter 2). This should include the 2, 5, 10, 25, 50, and 100-year events

2. Stage-area-storage curve for the detention basin (see Figure 8-4 for an example)

3. Stage-discharge curve for basin outlet control structure (see Figure 8-5 for an example) to match historic stage-discharge for the site.

A trial and error design procedure is often required, since only the inflow hydrographs are generally known. A general procedure for evaluating these variables is presented below:

1. Compute inflow hydrographs for 2 through 100-year design storms, as required in Volume 1, using procedures from Chapter 2. Both pre- and post-development hydrographs are required for the 2 through 100-year design storms.

2. Perform preliminary calculations to evaluate detention storage requirements (see Section 8.5) for the hydrographs from Step 1. If detention/retention requirements are satisfied for the 2 and 100-year design storms, intermediate storms are assumed to be controlled.

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

4. Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the Storage Indication Method (see Section 8.6.1). If the routed post-development peak discharges from the 2 through 100-year design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

6. Evaluate the downstream effects of detention outflow to ensure that the recession limb of the outflow hydrograph does not cause downstream flooding problems. The potential significance of downstream effects from detention can be evaluated by comparing the recession limbs of the pre-development and routed post-development hydrographs. When the maximum difference in discharge rates and the hydrograph time base both increase by more than 20 percent for the routed post-development hydrograph, then watershed modeling or information from a watershed master plan is required to show that downstream impacts can be controlled.

7. Evaluate the control structure outlet velocity and provide stabilization if velocity is greater than 3 ft/s for any storm event.

Since this procedure can involve a significant number of reservoir routing calculations, a computer method is useful for conducting final routing computations (See Chapter 12). Other information on retention/detention basin design computations can be found in articles by Mason and Rhomberg (1982, 1983), McKinnon (1984), Mein (1980), Rossmiller (1982), and Sandvik (1985).

8.4 Outlet Hydraulics

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlet works, procedures presented in Chapter 5 should be used to develop stage-discharge data. Slotted riser pipe outlet facilities should be avoided.

8.4.1 Sharp-Crested Weirs—No End Contractions

A sharp-crested weir with no end contractions is illustrated in Part A of Figure 8-5. The discharge equation for this configuration (Chow, 1959) is expressed as:

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right)LH^{1.5} \qquad (8\text{-}1)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 8-5, Part C)

H_c = Height of weir crest above channel bottom, in feet (see Figure 8-5, Part C)

L = Horizontal weir length, in feet

8.4.2 Sharp-Crested Weirs—Two End Contractions

A sharp-crested weir with two end contractions is illustrated in Part B of Figure 8-5. The discharge equation for this configuration (Chow, 1959) is expressed as:

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right)(L - 0.2H)H^{1.5} \qquad (8\text{-}2)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 8-5, Part C)

H_c = Height of weir crest above channel bottom, in feet (see Figure 8-5, Part C)

L = Horizontal weir length, in feet

8.4.3 Sharp-Crested Weirs—Submerged Discharge

The effect of submergence on a sharp-crested weir should be considered when applying Equations 8-1 and 8-2. When the tailwater rises above the weir crest elevation, the discharge over the weir will be reduced. To account for this submergence effect, the free discharge obtained by Equations 8-1 or 8-2 should be modified using the following equation (Brater and King, 1976):

$$Q_s = Q_f \left[1 - \left(\frac{H_2}{H_1} \right)^{1.5} \right]^{0.385} \qquad (8\text{-}3)$$

where:

Q_s = Submergence flow, in cfs

Q_f = Free flow, in cfs

H_1 = Upstream head above crest, in feet

H_2 = Downstream head above crest, in feet

8.4.4 Broad-Crested Weirs

The general form of the broad-crested weir equation (Brater and King, 1976) is expressed as:

$$Q = C\,L\,H^{1.5} \qquad (8\text{-}4)$$

where:

Q = Discharge, in cfs

C = Broad-crested weir coefficient

L = Broad-crested weir length, in feet

H = Head above weir crest, in feet

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087 for a broad-crested weir. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head, from Brater and King (1976), is included in Table 8-3.

8.4.5 V-Notch Weirs

The discharge through a v-notch weir can be evaluated using the equation (Merritt, et al, 1995):

$$Q = C_1 \tan\left(\frac{q}{2}\right) H^{2.5} \qquad (8\text{-}5)$$

where:

C_1 = discharge coefficient (See Figure 8-2)

Q = Discharge, in cfs

θ = Angle of v-notch, in degrees

H = Head on vortex of notch, in feet

8.4.6 Proportional Weirs

Although more complex to design and construct, a proportional weir may reduce the required detention/retention volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs from Sandvik (1985) are as follows:

$$Q = 4.97\, a^{1/2}\, b(H - a/3) \qquad (8\text{-}6)$$

$$x/b = 1 - \frac{1}{p}\left(\arctan\sqrt{y/a}\right) \qquad (8\text{-}7)$$

where Q is the weir discharge, in cfs, and the dimensions a, b, H, x, and y are shown in Figure 8-6.

8.4.7 Orifices

The discharge through an orifice can be evaluated using the equation:

$$Q = CA\ (2gH)^{0.5} \qquad (8\text{-}8)$$

where:

Q = Discharge, in cfs

C = Orifice coefficient (a value of 0.6 is usually appropriate; see Table 8-3 if additional information is desired)

A = Area of orifice, in square feet

g = Acceleration due to gravity, 32.174 feet/second^2

H = Head above orifice centroid, in feet

8.5 Preliminary Detention Calculations

8.5.1 Storage Volume

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 8-7.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = 0.5\ T_i\ (Q_i\text{-}Q_o) \qquad (8\text{-}9)$$

where:

V_s = Storage volume estimate

Q_i = Peak inflow rate

Q_o = Peak outflow rate

T_i = Duration of basin inflow

Any consistent units may be used for Equation 8-9.

An alternative preliminary estimate of the detention volume required for a specified peak flow reduction also can be obtained by the following regression equation procedure:

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

2. Calculate a preliminary estimate of the ratio V_s/V_r using the input data from Step 1 and the following equation (Wycoff & Singh, 1976):

$$\frac{V_s}{V_r} = \frac{1.291\left(1 - \frac{Q_o}{Q_i}\right)^{0.753}}{\left(\frac{t_b}{t_p}\right)^{0.411}} \qquad (8\text{-}10)$$

   where:

   V_s = Volume of storage, in inches

   V_r = Volume of runoff, in inches

   Q_o = Outflow peak flow, in cfs

   Q_i = Inflow peak flow, in cfs

   t_b = Time base of the inflow hydrograph, in hours, determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak

   t_p = Time to peak of the inflow hydrograph, in hours

3. Multiply the ratio V_s/V_r from Step 2 by the volume of runoff in the inflow hydrograph to obtain the estimated storage volume required to keep from exceeding the allowable peak outflow rate for each design storm event.

8.5.2 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected detention volume can be obtained by the following regression equation procedure:

1. Determine input data, including the volume of runoff, V_r, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, the time to peak of the inflow hydrograph, t_p, and the storage volume, V_s.

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff & Singh, 1976) :

$$\frac{Q_o}{Q_i} = 1 - 0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546} \quad (8\text{-}11)$$

where:

Q_o = Outflow peak flow, in cfs

Q_i = Inflow peak flow, in cfs

V_s = Volume of storage, in inches

V_r = Volume of runoff, in inches

t_b = Time base of the inflow hydrograph, in hours, determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak

t_p = Time to peak of the inflow hydrograph, in hours

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated in Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

8.6 Routing Calculations

The Storage Indication Method is recommended for reservoir routing calculations for detention facility final design.

8.6.1 Storage Indication Method

The following procedure is used to perform a reservoir routing by the Storage Indication Method:

1. Develop an inflow hydrograph, a stage-discharge curve, and a stage-storage curve for the proposed detention facilities. Example stage-storage and stage-discharge curves are presented in Figures 8-3 and 8-4, respectively.

2. Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph.

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S \pm O\Delta t/2$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 8-4 and Figure 8-8.

4. For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 - O_1\Delta t/2$ can be determined from the appropriate storage characteristics curve (e.g., Figure 8-8).

5. Determine the value of $S_2 + O_2At/2$ from the following relationship:

$$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \qquad (8\text{-}12)$$

where:

S_2 = Storage volume at time 2, in cubic feet

O_2 = Outflow rate at time 2, in cfs

Δt = Routing time period, in seconds

S_1 = Storage volume at time 1, in cubic feet

O_1 = Outflow rate at time 1, in cfs

I_1 = Inflow rate at time 1, in cfs

I_2 = Inflow rate at time 2, in cfs

Other consistent units are equally appropriate, as demonstrated in the examples below.

6. Enter the appropriate storage characteristics curve (e.g., Figure 8-8) at the value of $S_2 + O_2\Delta t/2$ determined in Step 5 and read off a new depth of water, H_2.

7. Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve (e.g., Figure 8-4).

8. Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1, and H_1 equal to the previous I_2, O_2, S_2, and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

8.6.2 Example Problems

<u>Example 8-1. Routing Using the Storage Indication Method</u>

An example application of the Storage Indication Method using data presented in Figures 8-3, 8-4, and 8-8 is presented in Table 8-5. The inflow hydrograph is given in columns 1 and 2 of Table 8-5. The objective is to find the outflow using the Storage Indication Method. A step-by-step discussion of the calculations summarized in Table 8-5 is presented below.

1. Using the data tabulated in Column 2 of Table 8-5, calculate:

$$\frac{(I_1 + I_2)\Delta t}{2}$$

   and tabulate these values in Column 3 of Table 8-5. For routing calculations in this example, inflows and outflows are converted to units of acre-feet per minute. The final outflow is reported in cfs.

2. Given that $S_1 - O_1\Delta t/2 = 0.05$ acre-foot for $H_1 = 0$ foot, find $S_2 + O_2\Delta t/2$ by adding 0.05 + 0.01 (Column 5 value plus Column 3 value) and tabulate 0.06 acre-foot in Column 6 of Table 8-5.

3. Enter the $S + O\Delta t/2$ storage characteristics curve in Figure 8-8 and read the stage at the value of 0.06 acre-foot. This value is found to be 100.10 feet and is tabulated as stage H_2 in Column 7 of Table 8-5.

4. Using the stage of 100.10 feet found in Step 4, enter the stage-discharge curve (Figure 8-4) and find the discharge corresponding to that stage. In this case, O is approximately 1 cfs and is tabulated in Column 8 of Table 8-5.

5. Assign the value of H_2 to H_1, find a new value of $S_1 - O_1\Delta t/2$ from Figure 8-8, and repeat the calculations for Steps 2, 3, and 4. Continue repeating these calculations until the entire inflow hydrograph has been routed through the storage basin.

6. The Storage Indication Method calculations give a peak outflow of 220 cfs. The inflow hydrograph has a peak rate of 360 cfs, so a reduction of approximately 40 percent is calculated.

Example 8-2. Multi-Design Storm Quantity Management Detention/Retention Calculations

This example demonstrates the application of the methodology presented in this chapter for typical detention/retention facility design. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using appropriate hydrologic methods from Chapter 2.

Part 1. Design Discharge and Hydrographs

As specified in Section 8.1.1, the detention/retention design should consider both the 2- and 10-year design storm events. Example peak discharges from these storm events are as follows:

Pre-development 2-year peak discharge	150 cfs
Pre-development 10-year peak discharge	200 cfs
Post-development 2-year peak discharge	190 cfs
Post-development 10-year peak discharge	250 cfs

Since the post-development peak discharge must not exceed the pre-development peak discharge (Section 8.1.1), the allowable design discharges are 150 and 200 cfs for the 2- and 10-year storms, respectively.

Example runoff hydrographs are shown in Table 8-6. Inflow durations of the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for the 2- and 10-year storms.

Part 2. Preliminary Detention Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section 8.5.1. For the 2- and 10-year storms, the required storage volumes, V_s, are computed as follows:

2-year storm:

$$V_s = (0.5)(3{,}600)(1.2)(190-150)/43{,}560$$

$$V_s = 1.98 \text{ acre-feet}$$

10-year storm:

$$V_s = (0.5)(3,600)(1.25)(250\text{-}200)/43,560$$

$$V_s = 2.58 \text{ acre-feet}$$

Part 3. Facilities Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a retention/detention facility that should provide adequate peak flow attenuation for both the 2- and 10-year design storms are presented in Table 8-7. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates for both the 2-and 10-year design storms to be provided when the corresponding allowable peak discharges occurred. Storage values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 feet, and no tailwater submergence. The capacity of storage relief structures (see Section 8.7) was assumed to be negligible.

Reservoir routing was conducted using the Storage Indication Method (see Section 8.6.1) for both the 2- and 10-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Storage characteristics required for routing are presented in Table 8-7 and Figure 8-9.

Routing results using 0.1-hour time steps are shown in Tables 8-8 and 8-9 for the 2- and 10-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-year design storms.

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, the 100-year design storm should be routed through the facilities to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability and maintenance, grading to prevent standing water, and provisions for public safety.

Part 4. Downstream Effects

An estimate of the potential effects (increased peak flow rate and recession time) of detention/retention on downstream facilities (see Section 8.3, Step 6) may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown for the 2- and 10-year design storms in Figures 8-10 and 8-11, respectively.

8.6.3 NRCS TR-55 Graphical Method

The NRCS (formerly SCS) has a graphical method for estimating the peak flow reduction capability of detention ponds. The information in the 1975 version of TR-55 has been extensively revised (USDA, NRCS, 1986) and is not duplicated in this manual, since it is only recommended for preliminary calculations.

8.7 Land-Locked Retention

Watershed areas that drain to a central depression with no positive outlet are typical of karst topography and can be evaluated using a mass flow routing procedure to estimate flood elevations. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires good field measurements and a thorough understanding of local conditions. Since outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

The steps presented below for the mass routing procedure are illustrated by the example in Figure 8-12:

1. Obtain cumulative rainfall data for the 100-year frequency, 10-day duration design event from Figure 8-13.

2. Calculate the cumulative inflow to the land-locked retention basin using the rainfall data from Step 1 and an appropriate runoff procedure from Chapter 2. Plot the mass inflow to the retention basin (see Figure 8-12 for an example).

3. Develop the basin outflow from appropriate field measurements of hydraulic conductivity, taking into consideration worst-case water table conditions. Hydraulic conductivity should be established using in situ test methods, then compared to observed performance characteristics of the site. Plot the mass outflow as a straight line with a slope corresponding to worst-case saturated hydraulic conductivity estimated a professional engineer from geotechnical field tests outflow in inches/hour (see Figure 8-12 for an example).

4. Draw a line tangent to the mass inflow curve from Step 2, which has a slope parallel to the mass outflow line from Step 3.

5. Locate the point of tangency between the mass inflow curve of Step 2 and the tangent line drawn for Step 4. The distance from this point of tangency and the mass outflow line represents the maximum storage required for the design runoff.

6. Determine the flood elevation associated with the maximum storage volume determined in Step 5. Use this flood elevation to evaluate flood protection requirements of the project. The zero volume elevation should be established as the normal wet season water surface or water table elevation or the pit bottom, whichever is highest. This should be determined from a geotechnical test and analysis.

7. If the project area discharges into a stormwater system tributary to the land-locked depression, detention storage facilities are required to comply with the pre-development discharge requirements for the project (see Section 8.1).

Unless the retention basin is designed to have a permanent pool, including water budget calculations and provisions for preventing anaerobic conditions, relief structures should be provided to prevent standing water conditions. Depths greater than 12 feet should be equipped with an aerator to prevent thermal stratification.

8.8 Permanent Pool Facilities

MWS encourages the use of permanent pool detention/retention facilities designed for stormwater quality benefit. It also recognizes that wet detention ponds are preferable over dry detention ponds because of the added sediment storage flexibility provided by the permanent pool easing some maintenance activities (namely sediment removal is required less frequently). Provisions for safe slopes, safety benches (grading), access restriction to dangerous areas (fencing), weed control, mosquito control shelf, and aeration for prevention of anaerobic conditions should be considered. MWS may reject facility designs with the potential for becoming nuisances or health hazards.

8.8.1 Water Budget Calculations

Water budget calculations are required (see Volume 1, Section 6.8) for all permanent pool facilities and should consider performance for average annual and wet season conditions. The water budget should consider all significant inflows and outflows including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation, groundwater inflow, and outflow.

Average annual runoff may be computed using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Infiltration and exfiltration should be based on site-specific soils testing data. Evaporation may be approximated using the mean monthly pan evaporation or free water surface evaporation data presented in Table 8-10.

8.8.2 Example Problem

Example 8-3. Water Budget Calculations

A shallow basin with an average surface area of 3 acres and a bottom area of 2 acres is planned for construction at the outlet of a 100-acre watershed. The watershed is estimated to have a post-development runoff coefficient of 0.4. Site-specific soils testing indicates that the average infiltration rate is about 0.1 inch per hour. Determine for average annual conditions if the facility will function as a permanent pool.

1. From NOAA rainfall records, the average annual rainfall is about 50 inches.

2. From Table 8-10, the mean annual evaporation is 35 inches.

3. The average annual runoff is estimated as:

   RO = (0.3) (50 inches) (100 acres).
   RO = 1,500 acre-inches

4. The average annual evaporation is estimated as:

   EVAP = (35 inches) (3 acres)
   EVAP = 105 acre-inches

5. The average annual infiltration is estimated as:

$$INFIL = \left(\frac{0.1 in}{hr}\right)\left(\frac{24\ hr}{day}\right)\left(\frac{365\ days}{yr}\right)(2\ acres)$$

   INFIL = 1,752 acre-inches

6. Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is

   NET BUDGET = 1,500 - 105 - 1752
   NET BUDGET = -357 acre-inches

   and the proposed facility will not function as a permanent pool.

7. Revise pool design as follows;

   Average surface area, 2 acres

Bottom area, 1 acre

8. Recompute the evaporation and infiltration (Steps 4 and 5):

EVAP = (35 inches) (2 acres)

EVAP = 70 acre-inches

$$INFIL = \left(\frac{0.1\ in}{hr}\right)\left(\frac{24\ hr}{day}\right)\left(\frac{365\ days}{yr}\right)(1\ acre)$$

INFIL = 876 acre-inches

9. The revised runoff less evaporation and infiltration losses is

NET BUDGET = 1,500 - 70 - 876
NET BUDGET = 554 acre-inches

The revised facility is assumed to function as a permanent pool. To evaluate actual performance conditions, continuous simulation procedures should be applied.

8.9 Construction and Maintenance Considerations

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed facilities (see Volume 1 Section 6.8). To assure acceptable performance and function, MWS discourages the design of stormwater detention/ retention facilities that may require excessive maintenance. The following maintenance activities should be considered:

1. Weed growth
2. Grass maintenance
3. Sediment removal
4. Slope deterioration
5. Mosquito control

Proper design may eliminate or reduce maintenance requirements by addressing the potential for problems to develop. Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers. Sediment removal may be facilitated by constructing forebays or baffle boxes at the inlets to contain sediment for easy removal. Bank deterioration can be controlled with protective soil bioengineering techniques or lining or by limiting bank slopes. Mosquito control will not be a major problem if the permanent pool is designed with a 12-inch shelf at the edge.

8.10 Access Management

Access management may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences may be required for detention areas where one or more of the following conditions exist:

1. Rapid stage changes (greater than 2-feet over 30 minutes) would make escape practically impossible for small children.

2. Water depths either exceed 2.5 feet for more than 24 hours or are permanently wet and have side slopes steeper than 4:1 (horizontal:vertical). This is for sites where it is impracticable to grade a mosquito bench, safety bench, and 6:1 (horizontal:vertical) littoral zone slope.

3. Side slopes equal or exceed 2:1 (horizontal to vertical).

Table 8-1
Size Categories For Dams In Tennessee

Category	Storage (acre-ft)	Height (ft)
Small	30 to <1,000	20 to <41
Intermediate	1,000 to 50,000	41 to 100
Large	>50,000	>100

Table 8-2
Minimum Freeboard Design Storms For Dams In Tennessee

Hazard Potential Category	Size	Freeboard Design Storm (6-hour)
Category 3 (Low)	Small	100 yr
	Intermediate	α PMP[a]
	Large	½ PMP
Category 2 (Significant)	Small	α PMP
	Intermediate	½ PMP
	Large	PMP
Category 1 (High)	Small	½ PMP
	Intermediate	PMP
	Large	PMP

[a]Probable maximum precipitation, defined as the precipitation resulting from a storm containing the most critical probable conditions.

Reference: Tennessee Department of Health and Environment (1973, 1987).

Table 8-3
Broad-Crested Weir Coefficient C Values
As A Function Of Weir Crest Breadth And Head

Measured Head, H[a]	Weir Crest Breadth (ft)										
(ft)	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[a]Measured at least 2.5H upstream of the weir.

Reference: Brater and King (1976).

Table 8-4
Example Tabulation Of Storage Characteristics Curves

Stage	Storage[a]	Discharge[b]		$S - \frac{0}{2}\Delta t^c$	$S + \frac{0}{2}\Delta t^c$
(ft above NGVD)	(acre-ft)	**(cfs)**	(acre-ft/hr)[d]	(acre-ft)	(acre-ft)
100	0.05	0	0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[a]Obtained from Figure 8-3.

[b]Obtained from Figure 8-4.

[c]Δt = 10 min = 0.167 hour.

[d]1 cfs = 0.0826 acre-ft/hr.

Table 8-5
Storage Indication Method—Example 8-1 Calculations

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time	Inflow	$\frac{(I_1+I_2)\Delta t}{2}$	H_1	$S_1 - \frac{0_1}{2}\Delta t$	$S_2 - \frac{0_2}{2}\Delta t$	H_2	Outflow O
(min)	(cfs)	(acre-ft)	(ft)	(acre-ft)	(acre-ft)	(ft)	(cfs)
0	0						
10	2	0.01	0.0	0.05	0.06	100.10	1
20	27	0.20	100.10	0.06	0.26	101.10	16
30	130	1.08	101.10	0.21	1.29	102.20	41
40	300	2.96	102.20	0.61	3.57	104.10	100
50	360	4.55	104.10	2.20	6.75	105.60	175
60	289	4.47	105.60	4.40	8.87	106.25	217
70	194	3.33	106.25	5.80	9.13	106.30	220
80	133	2.25	106.30	5.90	8.15	106.05	205
90	91	1.54	106.05	5.30	6.84	105.65	177
100	61	1.05	105.65	4.50	5.55	105.10	147
110	37	0.67	105.10	3.60	4.27	104.50	116
120	20	0.39	104.50	2.70	3.09	103.80	87
130	11	0.21	103.80	1.90	2.11	103.05	64
140	5	0.11	103.05	1.18	1.29	102.25	43
150	1	0.04	102.25	0.63	0.67	101.40	22
160	0	0.0	101.40	0.35	0.35	100.70	10

Note: $S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \textit{Equation } 8-12$

(column 6) = (column 5) + (column 3)

Table 8-6
Example 8-2 Runoff Hydrographs

	Pre-Development Runoff		Post-Development Runoff	
Time (Hours)	2-Year (cfs)	10-Year (cfs)	2-Year (cfs)	10-Year (cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190	250
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1
1.3	7	12	0	0

Table 8-7
Example 8-2 Stage-Discharge-Storage Data

Stage (ft)	Q (cfs)	S (acre-ft)	$S_1 - \frac{0}{2}\Delta t$ (acre-ft)	$S_1 - \frac{0}{2}\Delta t$ (acre-ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52
2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.91
4.0	100	1.40	1.81	0.99
4.3	110	1.52	1.97	1.06
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60
6.2	190	2.46	3.25	1.68
6.4	200	2.58	3.41	1.76
6.6	210	2.71	3.57	1.84
6.8	220	2.83	3.74	1.92
7.0	230	2.95	3.90	2.00
7.2	240	3.08	4.07	2.09
7.4	250	3.21	4.24	2.017

Notes:
1. Broad-crested weir length = 4 feet.
2. Discharge coefficient = 3.1 (assumed constant).

Table 8-8
Example 8-2 Reservoir Routing For The 2-Year Storm

Time (hours)	Inflow (cfs)	$\frac{(I_1+I_2)\Delta t}{2}$ (acre-ft)	H_1 (ft)	$S_1-\frac{O_1}{2}\Delta t$ (acre-ft)	$S_2+\frac{O_2}{2}\Delta t$ (acre-ft)	H_2 (ft)	Outflow O (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.0	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.08	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.30	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

Note: $S_2+\frac{O_2}{2}\Delta t=\left[S_1-\frac{O_1}{2}\Delta t\right]+\left[\frac{I_1+I_2}{2}\Delta t\right]$ *Equation* 8 − 12

(column 6) = (column 5) + (column 3)

Allowable outflow = 150 cfs (see Table 8-6).

Table 8-9
Example 8-2 Reservoir Routing For The 10-Year Storm

Time (hours)	Inflow (cfs)	$\frac{(I_1 + I_2)\Delta t}{2}$ (acre-ft)	H_1 (ft)	$S_1 - \frac{O_1}{2}\Delta t$ (acre-ft)	$S_2 + \frac{O_2}{2}\Delta t$ (acre-ft)	H_2 (ft)	Outflow O (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	5.80	173
0.5	90	1.05	5.80	1.30	2.35	4.95	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

Note: $$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \textit{Equation } 8-12$$

(column 6) = (column 5) + (column 3)

Allowable outflow = 200 cfs (see Table 8-6).

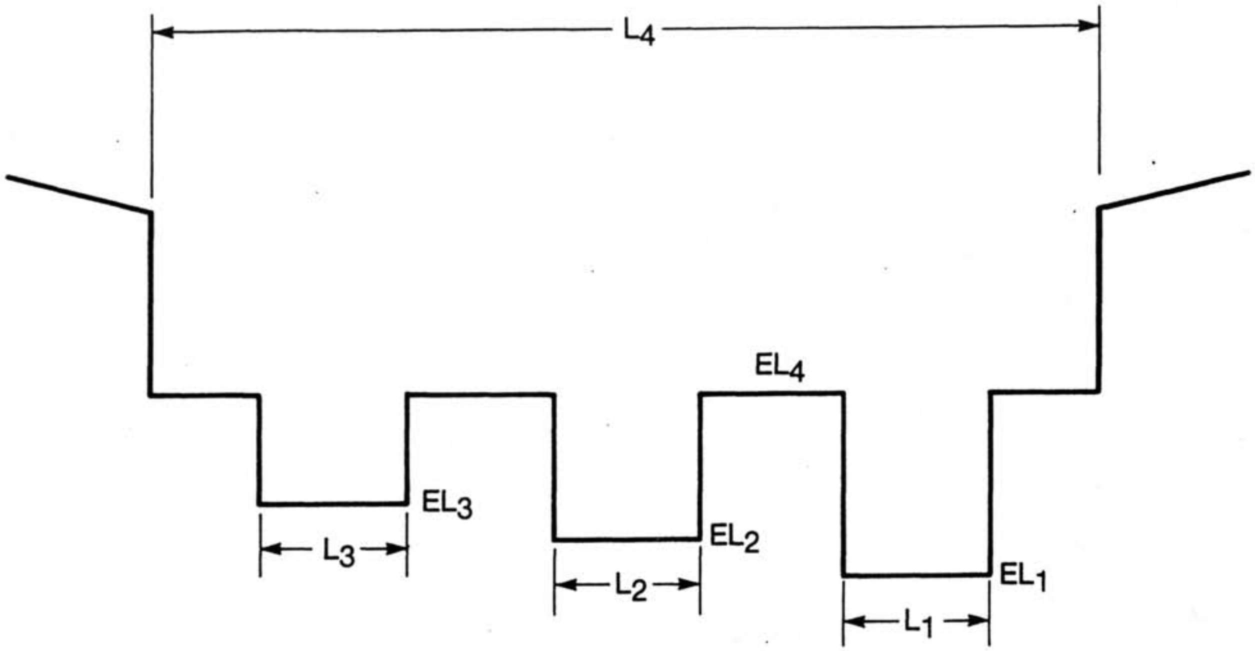
Table 8-10
Mean Monthly Evaporation Data

Month	Pan Evaporation[a] (inches)	Free Water Surface Evaporation[b] (inches)
January	1.10	0.85
February	1.69	1.30
March	3.10	2.39
April	4.79	3.69
May	5.47	4.21
June	6.28	4.84
July	6.38	4.91
August	5.70	4.39
September	4.24	3.26
October	3.24	2.49
November	1.92	1.48
December	1.60	1.23
Mean Annual	45.51	35.04

[a]Based on recorded data from 1977 through 1986 at Cheatham Lock and Dam Station.

[b]Pan coefficient is 0.77.

Reference: U.S. Department of Commerce, NOAA (1977-1988).



TYPICAL ELEVATION

For Water Surface Elevation		
From	**To**	**Discharge Is Computed By**
EL_1	EL_2	$Q_1 = C_1 L_1 (EL_{WS} - EL_1)^{3/2}$
EL_2	EL_3	$Q_2 = C_2 L_2 (EL_{WS} - EL_2)^{3/2} + Q_1$
EL_3	EL_4	$Q_3 = C_3 L_3 (EL_{WS} - EL_3)^{3/2} + Q_2$
EL_4	—	$Q_4 = C4 (L4 - L_3 - L_2 - L_1)(EL_{WS} - EL_4)^{3/2} + Q_3$

where:

L_i = Length of weir opening i, in feet

EL_i = Invert elevation of weir opening i, in feet

Q_i = Cumulative discharge above weir opening i, in cfs

EL_{WS} = Elevation of the water surface, in feet

C_i = Weir discharge coefficient

Figure 8-1
Example Multi-Stage Control Structure

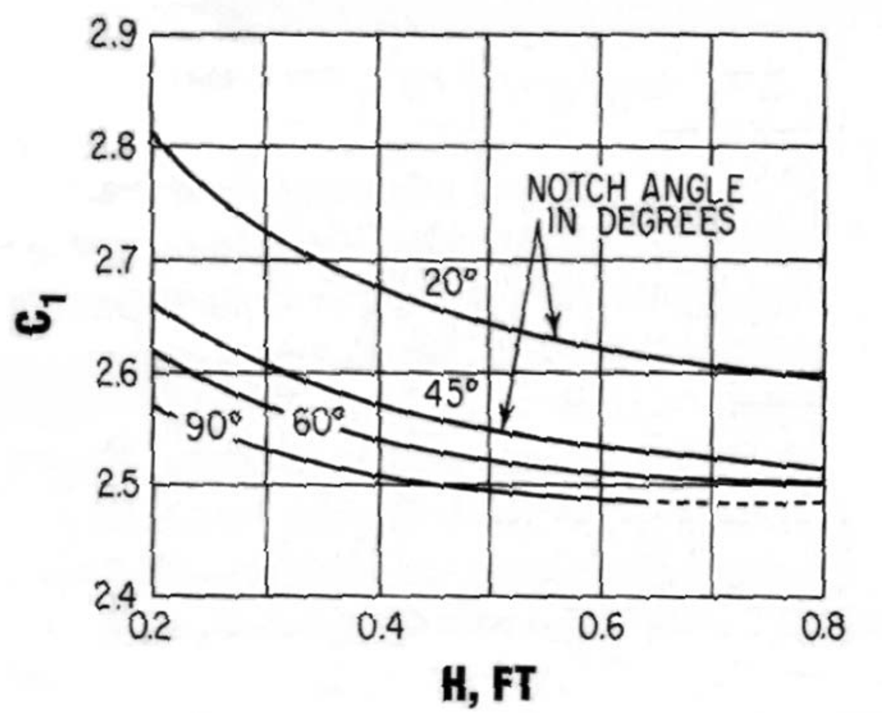


Figure 8-2
Sharp-Crested "V" Notch Weir Discharge Coefficients

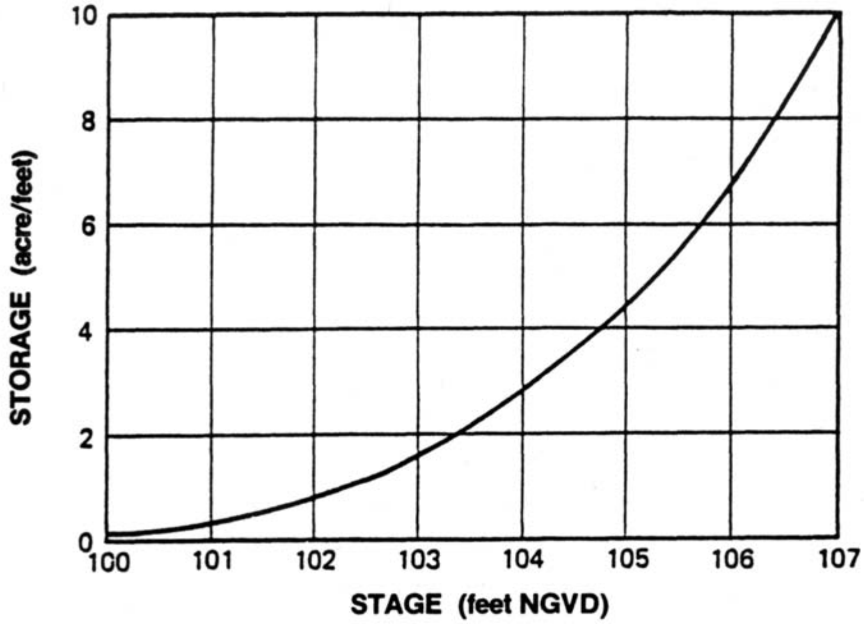


Figure 8-3
Example Stage-Storage Curve

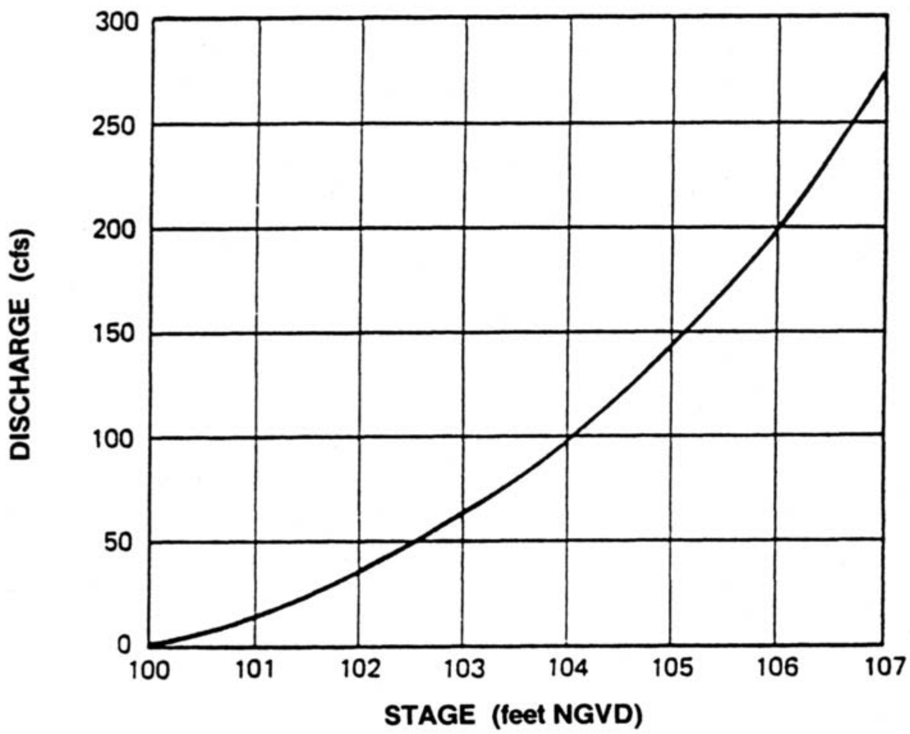


Figure 8-4
Example Stage-Discharge Curve

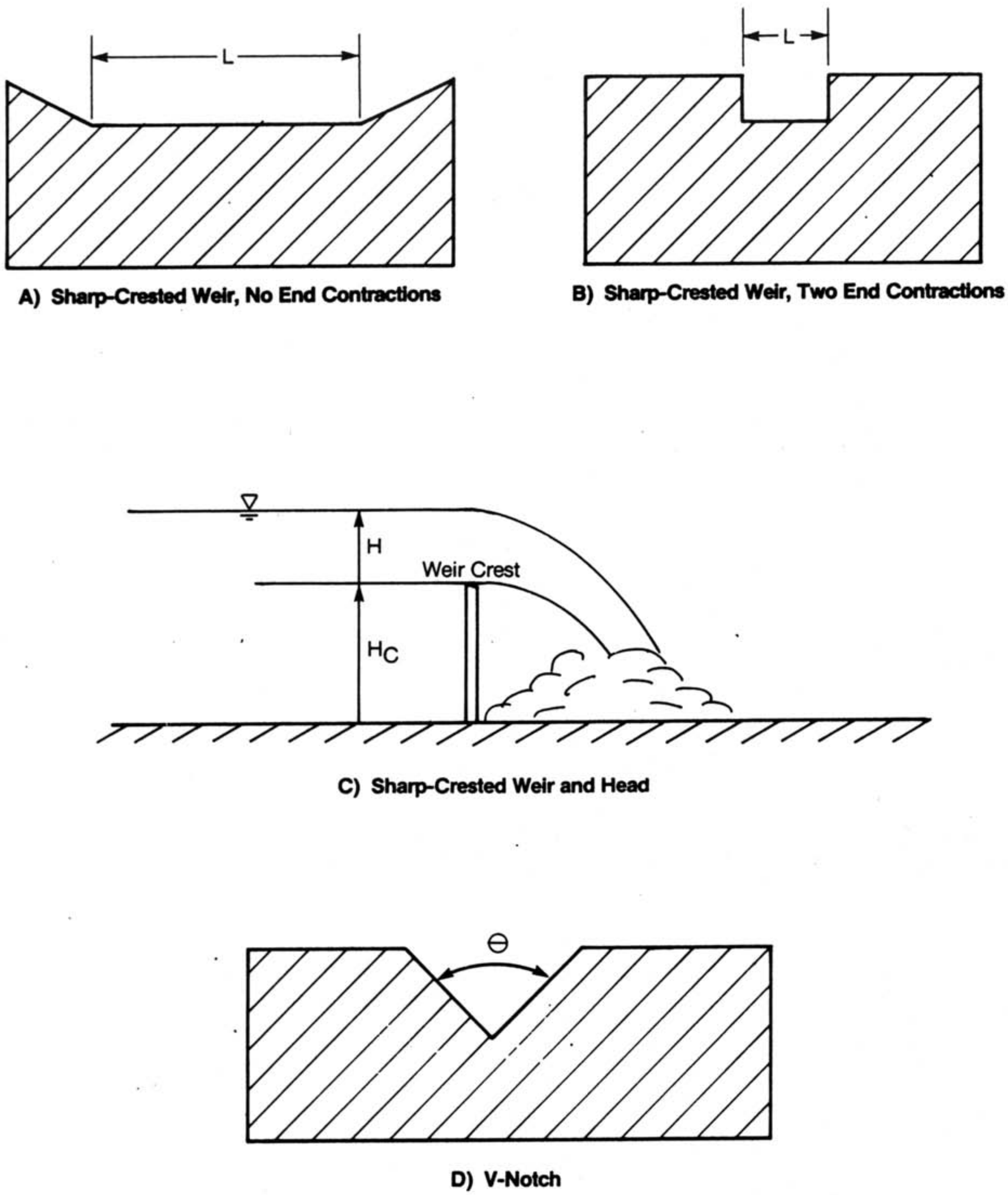
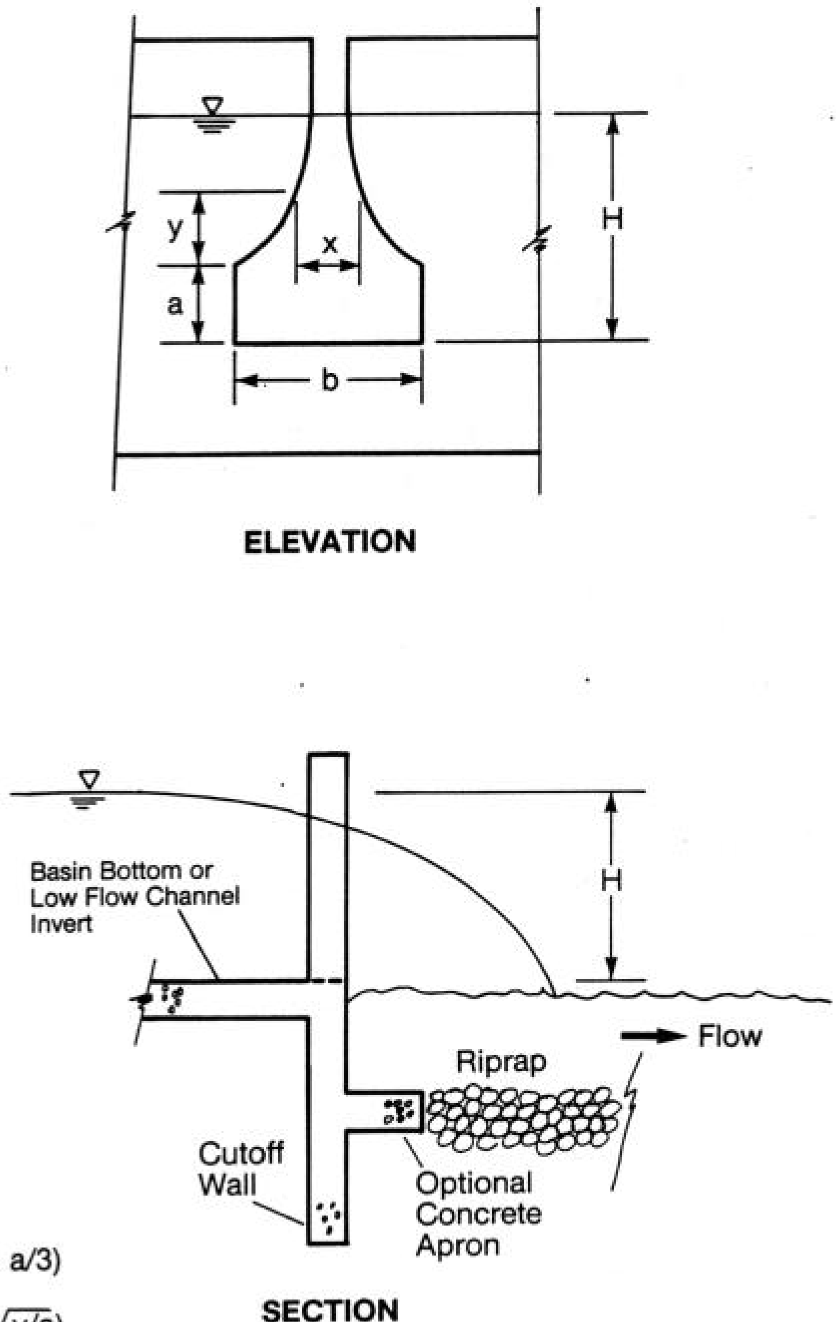


Figure 8-5
Illustrations of Weir Flow Control Structures

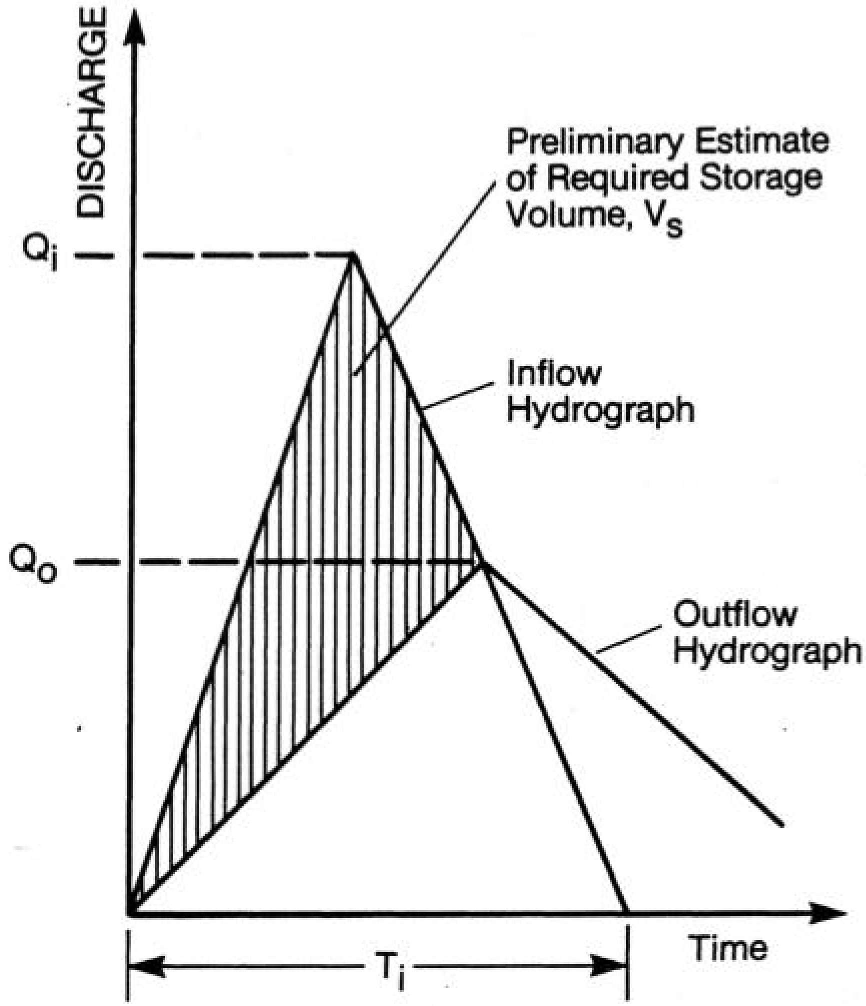


$$Q = 4.97\, a^{1/2} b\, (H - a/3)$$

$$x/b = 1 - 1/\pi\, (\arctan \sqrt{y/a})$$

Reference: Sandvik (1985).

Figure 8-6
Dimensions Used for Design of a Proportional Weir



$$V_s = 0.5\, T_i\, (Q_i - Q_o)$$

where:

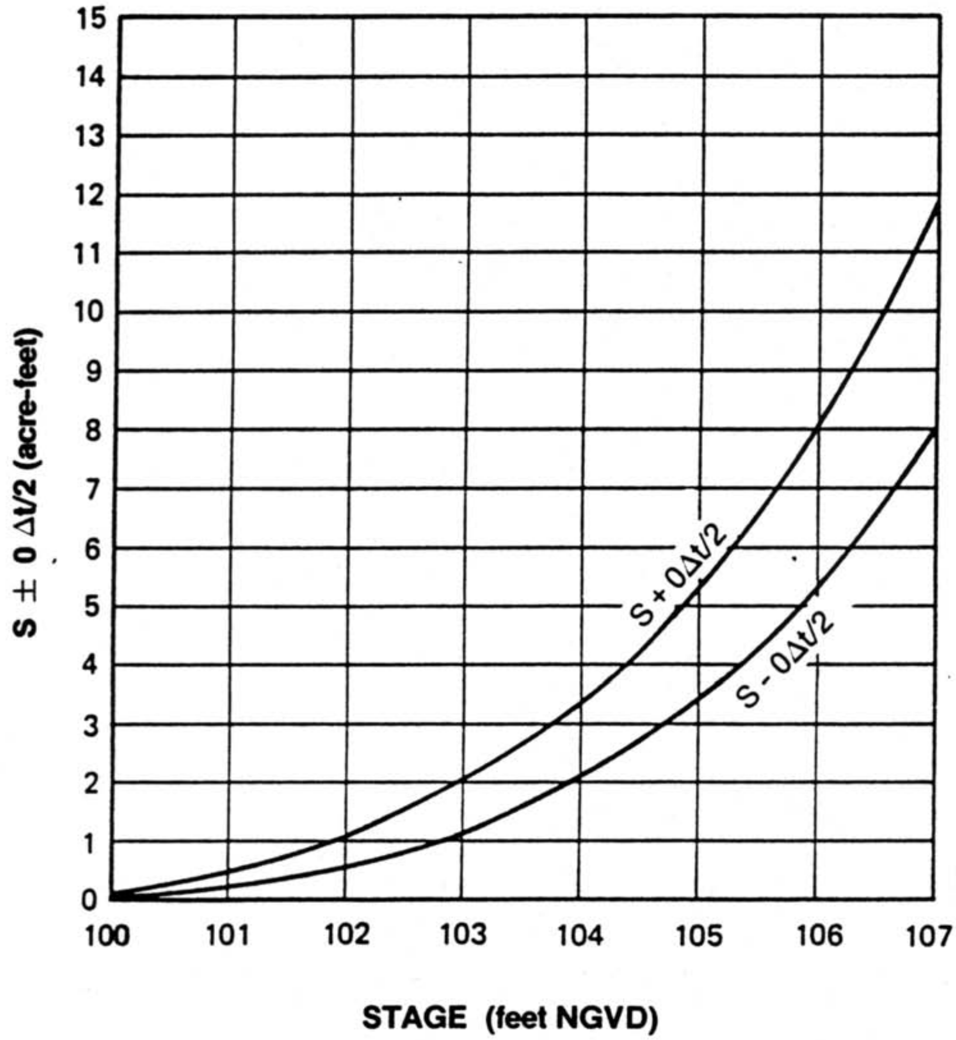
V_s = Storage volume estimate

Q_i = Peak inflow rate

Q_o = Peak outflow rate

T_i = Duration of basin inflow

Figure 8-7
Triangular Shaped Hydrographs for Preliminary Estimate
of Required Storage Volume



Note: Data presented in Table 8-4.

Figure 8-8
Example Storage Characteristics Curves

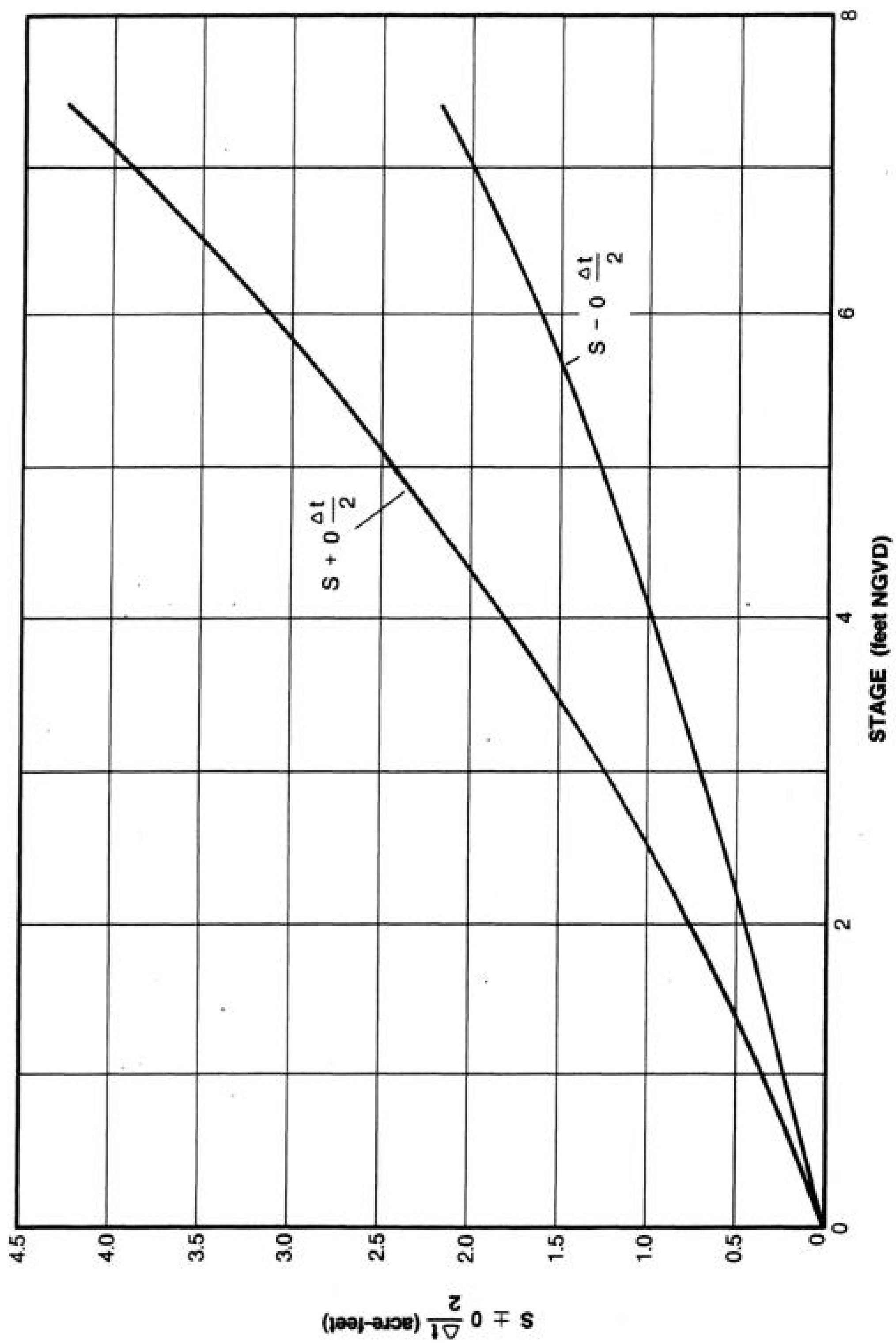


Figure 8-9
Example 8-2 Storage Characteristics Curves

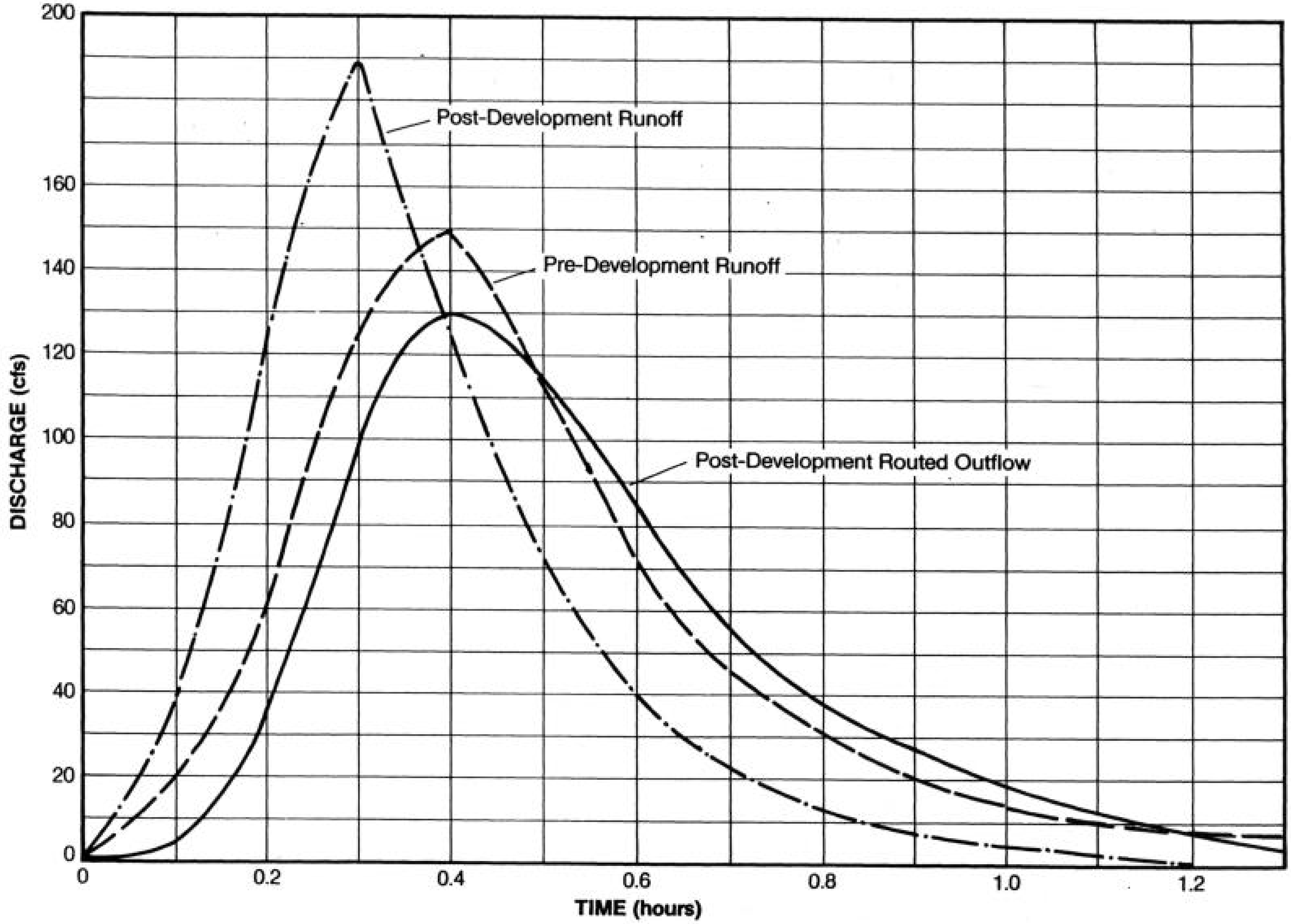


Figure 8-10
Example 8-2 Pre- and Post-Development Runoff Hydrographs
and Routed Outflow Hydrograph for the 2-Year Design Storm

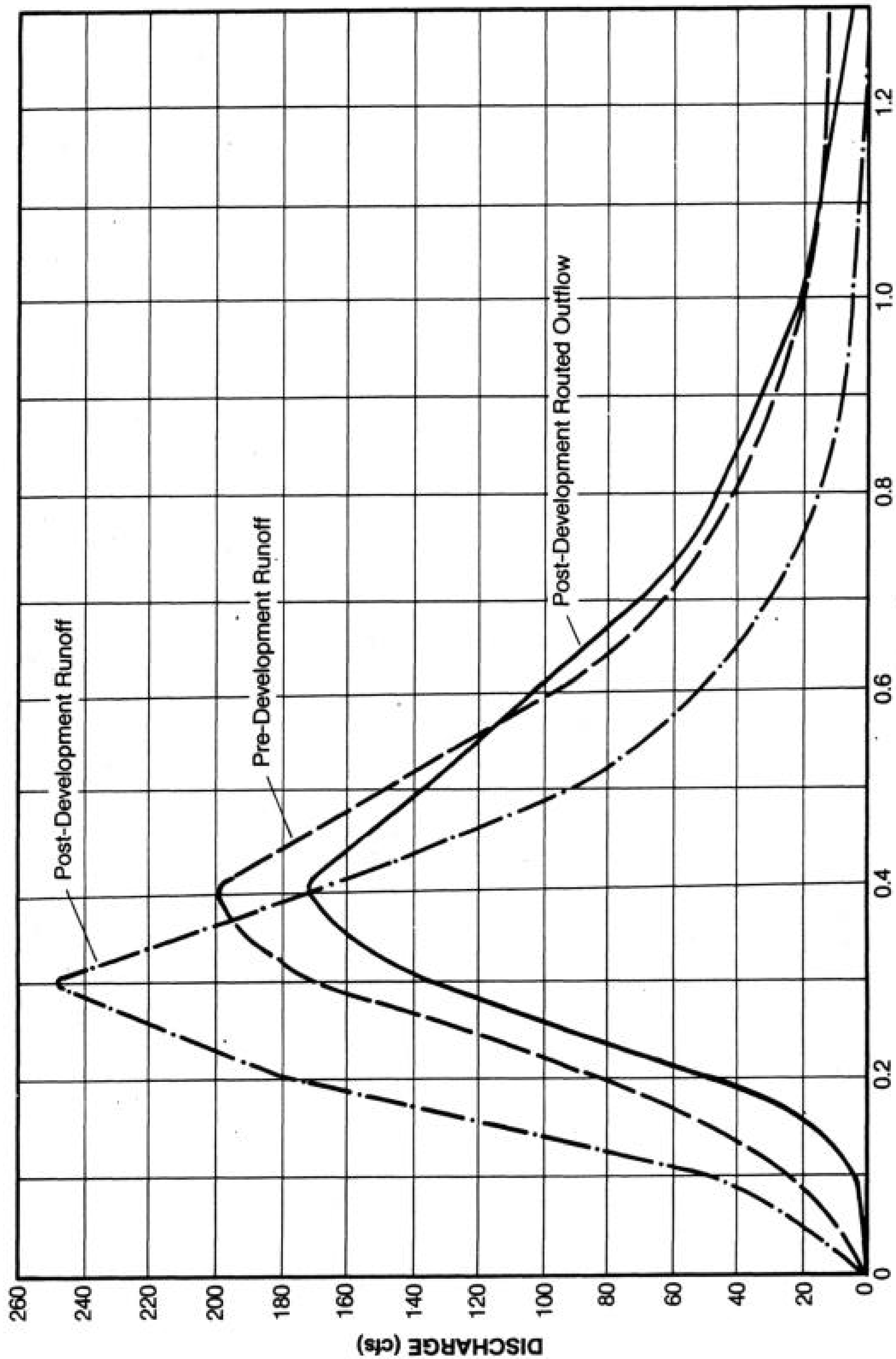


Figure 8-11
Example 8-2 Pre- and Post-Development Runoff Hydrographs
and Routed Outflow Hydrograph for the 10-Year Design Storm

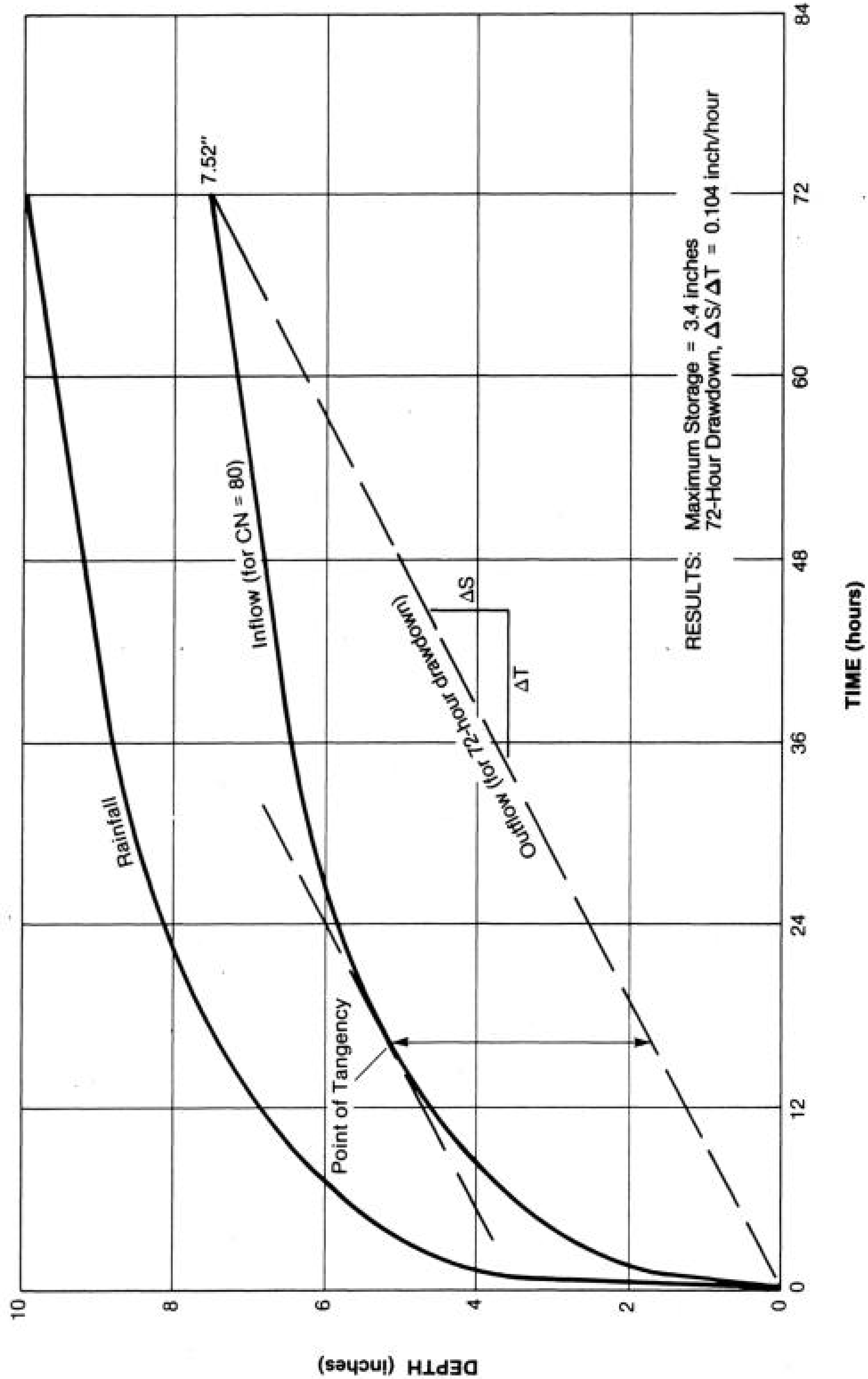


Figure 8-12
Mass Routing Example for Land-Locked Retention Areas

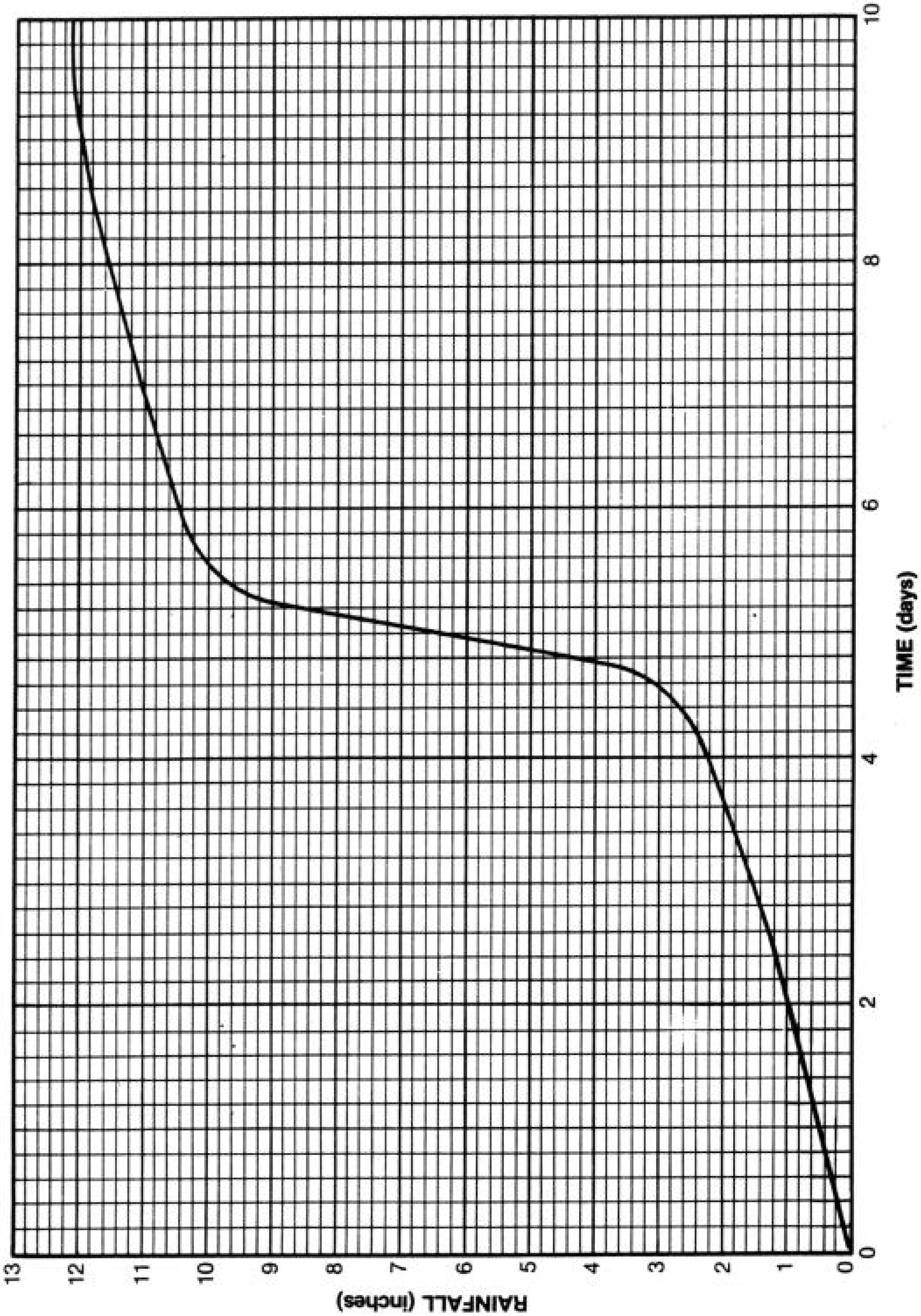


Figure 8-13
Cumulative Rainfall Data for the 100-Year,
10-Day Design Storm

CHAPTER 9
EROSION PREVENTION
AND SEDIMENT CONTROL

Chapter 9
EROSION PREVENTION AND SEDIMENT CONTROL

Synopsis

Construction and land development activities that impact existing topography, vegetative cover, and hydrologic characteristics often increase the potential for soil erosion and sediment transport. Specific control measures to mitigate adverse impacts are required by the regulations contained in Volume 1 Sections 4.2.3 and 6.10 with additional guidance and design criteria presented in Volume 4 Sections TCP and PESC. To address regulatory requirements, an erosion and sediment control plan should be prepared according to Volume 1 Section 4.2.3. The plan should provide information for each of the following items:

1. Existing and proposed contours.
2. A construction activity schedule with a plan for implementing erosion prevention and sediment control measures (EP&SC).
3. Temporary control measures that will be implemented (Volume 4 Section TCP).
4. Removal of temporary measures, when appropriate, and establishment of permanent stabilization.
5. Permanent control measures that will be implemented (Volume 4 Section PESC).
6. Maintenance requirements for temporary and permanent control measures.
7. Measures to protect adjacent areas.
8. Contingency measures in the event that planned controls are not effective.
9. Permanent stormwater conveyance facilities (Volume 4 Section PTP).

9.1 Basic Principles

The design of erosion prevention and sediment controls involve the application of common sense planning, scheduling, and control actions that will minimize the adverse impacts of soil erosion, transport, and deposition. The following five basic principles govern the development and implementation of a sound erosion prevention and sediment control plan:

1. The project should be planned to take advantage of the topography, soils, waterways, buffers, and natural vegetation at the site.

2. The smallest practical area should be exposed for the shortest possible time.

3. Onsite erosion prevention measures should be applied to reduce the suspension of soil particles.

4. Sediment control measures should be used to prevent suspended soil from leaving the site.

5. A thorough inspection and maintenance program should be implemented.

These principles should be tied together in the planning process, which identifies potential erosion and sediment transport problems before construction begins.

Vegetative control measures are required for all disturbed areas and generally include practices such as filter strips, temporary seeding, permanent seeding, sodding, and mulching. Structural control measures are required when runoff leaves a disturbed site and generally include practices such as flow diversions, sediment traps, sediment basins, and permanent detention ponds.

The erosion prevention and sediment control plan must include appropriate construction specifications for all control measures. These specifications must be developed and/or implemented by the design engineer as required for site-specific conditions. Typical design application and design criteria, specifications, inspection recommendations and maintenance requirements are provided in Volume 4 Sections TCP and PESC.

9.2 Applying Best Management Practices

Guidance for applying control measures when developing an erosion and sediment control plan is presented in Volume 4 – Best Management Practices (BMPs). Section 1.3 provides additional detail about the erosion and sedimentation processes while Sections 1.5 and 1.6 discuss types of BMPs and how to select them. Key references for this volume are the, California Stormwater Best Management Practice Handbooks (1993), Caltrans Stormwater Quality Handbooks (1996),Urban Runoff Quality Management (1998) and Tennessee Erosion and Sediment Control Handbook (1992).

The regulations presented in Volume 1 Sections 4.2.3 and 6.10 should be thoroughly reviewed and considered in selecting BMPs to present on the EP&SC plan.

9.3 Universal Soil Loss Equation (USLE)

The USLE provides an empirical approach to estimate soil loss for specific site conditions. Application of the equation to evaluate the performance of proposed erosion and sediment control measures is recommended when the disturbed area exceeds 10 acres. The USLE is expressed as:

$$A = R\ K\ LS\ C\ P \qquad (9\text{-}1)$$

where:

A = Soil loss, in tons/acre, for the time period selected for R

R = Rainfall factor

K = Soil erodibility factor, in tons/acre/R unit

LS = Length-slope factor, dimensionless

C = Cropping management factor, dimensionless

P = Erosion control factor, dimensionless

Numerical values for each of the parameters in the USLE must be determined for each problem considered. Guiding principles and data for determining these parameters in Nashville are discussed in this section.

9.3.1 Rainfall Factor (R)

The average annual R value for Nashville and Davidson County is approximately 200 (Israelson et al., 1980). The monthly distribution, or cumulative percentage, of the average annual R values for a typical year can be determined using the erosion index (EI) distribution curve presented in Figure 9-1 as follows:

1. Use the EI distribution curve to determine the percent of the annual erosion index expected to occur during the time period of concern.

2. Multiply the R value of 200 by the percentage value from Step 1 to obtain the rainfall factor expected for the specified time period.

Annual R values for return period frequencies of 2, 5, and 20 years are reported in Table 9-1. Annual R values range from 198 for a 2-year return period to 339 for a 20-year return period. Expected average R values for single storms are presented in Table 9-2.

To determine the expected average value of soil loss for a specific annual return period or a single storm, the R values reported in Tables 9-1 and 9-2 are used directly in the USLE (Equation 9-1). For example, if the expected average soil loss for a 5-year design storm is desired, an R value of 68 is used in the USLE.

9.3.2 Soil Erodibility Factor (K)

K factors are published in the Soil Survey for Nashville and Davidson County, Tennessee (USDA, SCS, 1981). K factors are generally reported for selected depth intervals of the soil profile. Soil erodibility data published in the soil survey are presented in Table 9-3.

9.3.3 Length-Slope Factor (LS)

The LS factor can be estimated by the following procedure:

1. Identify uniform slope segments and estimate the slope length, in feet, and the slope, in percent.

2. Enter the x-axis of Figure 9-2 with the slope length and move vertically to the appropriate percent-slope curve.

3. Move horizontally from the intersection point in Step 2 to the y-axis and read the LS value.

The procedure is derived from field data for uniform slopes ranging from 3 to 18 percent and from about 30 to 300 feet in length. It should not be used beyond these limits, which are delineated in Figure 9-2. If the actual slope is irregular, special considerations may be required, as discussed below.

Typical concave and convex slopes are illustrated in Figure 9-3. These irregular slopes can be analyzed using Figure 9-2 by dividing the slope into a small number of equal-length and uniform segments. If this is done, two simplifying assumptions must be valid:

1. The changes in gradient are not sufficient to cause upslope deposition.

2. The irregular slope can be divided into a small number of equal-length segments in such a manner that the gradient within each segment is uniform.

After dividing the convex, concave, or complex (composed of both concave and convex components) slope into equal-length segments, the LS factor is determined as follows:

1. List the segment gradients in the order they occur on the slope, beginning at the upper end.

2. Enter the abscissa of Figure 9-2 with the total slope length and read the LS factor for each of the gradients listed in Step 1.

3. Multiply these LS factors by the appropriate factors from Table 9-4.

4. Add the products obtained from Step 3 to obtain the LS factor for the entire slope.

The change in slope required to induce the deposition of eroded soil is somewhat variable. In practice, areas of deposition should be identified by observation. When the slope breaks are sharp enough to cause deposition, the four-step procedure described above can be used to estimate the LS factor for slope segments above and below the point of deposition.

9.3.4 Control Practice Factor (CP)

For construction sites, Chen (1974) proposed that the individual C and P factors of the USLE be evaluated with a single control-practice factor (CP), which is defined as:

$$CP = C_s \, C_r \, C_o \qquad (9\text{-}2)$$

where:

CP = Control-practice factor, or the ratio of soil loss including control practice and soil loss without control practice

C_s = Control due to surface stabilization, such as seeding, mulching, and netting

C_r = Control due to runoff reduction practices, such as diversion berms, interceptor dikes, terraces, sodded ditches, level spreaders, and sectional down drains

C_o = Control due to any erosion control practice not noted above

Detailed information for determining quantitative values of the CP factor for selected erosion control systems for various types of land use and cover conditions is presented in Section 9.4. Tables 9-5 through 9-10 present C_s factors for various site conditions. C_r can be quantified using the expression (Chen, 1974):

$$C_r = \frac{1}{\sqrt{N+1}} \qquad (9\text{-}3)$$

where:

C_r = Runoff control factor, dimensionless

N = Number of diversions placed across a uniform slope

C_o must be established by the designer.

9.3.5 Plan Evaluation

The goal of soil erosion prevention and sediment control measures should be to prevent any sediment from leaving the site. This is not say that treatment practices must be implemented to trap sediment that enters the site, but only sediment that is generated on-site. Realistically, the objective should be to provide between 90 and 95 percent control of the total suspended solids from the disturbed site. Assuming a total gross erosion rate of 200 tons/acre/year and 95 percent control, the target soil loss for an erosion and sediment control plan is approximately 10 or less tons/acre/year. Based on these assumptions, the following procedure is recommended to evaluate the need for erosion prevention and sediment control measures:

1. Estimate the sediment yield from a project site with all erosion control practices in place, A, using the USLE, Equation 9-1.

2. Estimate the sediment trapped onsite, T, using information presented in Section 9.4.

3. Estimate the sediment delivery ratio with controls, D_c, using the equation:

$$D_c = \frac{A - T}{20} \qquad (9\text{-}4)$$

where:

D_c = Sediment delivery ratio with controls

A = Sediment yield from a project site with erosion control, in tons/acre/year (calculated using USLE, Equation 9-1)

T = Sediment trapped onsite, in tons/acre/year (see Section 9.4)

4. If D_c from Equation 9-4 is greater than 1, return to Step 1 and improve the erosion and sediment control plan until D_c is 1 or less.

9.3.6 Example Problem

<u>Example 9-1. Plan Evaluation</u>

A 12-acre site on Beason soils with a 200-foot long, 10 percent slope is to be cleared for construction. No seeding or mulching is planned, and the slope will remain in a rough, irregular tracked condition for about 1 year. Evaluate the acceptability of the proposed activity using the USLE to estimate soil loss for average annual conditions.

1. The average annual rainfall factor is given in Section 9.3.1 as R = 200.

2. The soil erodibility factor from Table 9-3 for Beason soils is K = 0.32.

3. The length-slope factor from Figure 9-2 for a 200-foot, 10 percent slope is LS = 1.93.

4. The control practice factor (Equation 9-2) is determined from a single factor for surface stabilization since no runoff reduction practices are planned. The surface stabilization factor from Table 9-5 for rough, irregular, tracked conditions is $C_s = 0.90$ and, from Equation 9-2, $CP = C_s = 0.90$.

5. The soil loss is estimated as

   $A = (200)\ (0.32)\ (1.93)\ (0.90)$

   $A = 111$ tons/acre/year

6. The sediment delivery ratio is estimated using Equation 9-4:

   $$D_c = \frac{111-0}{20}$$

   $D_c = 5.6$

7. Based on estimated soil loss, with $D_c > 1$, the proposed activity is unacceptable.

8. Improve erosion control by constructing diversions along the slope to reduce the slope length to 100 feet and use mechanically tacked straw or hay mulch at 1.5 tons/acre over the disturbed area.

9. The improved length-slope factor from Figure 9-2 for a 100-foot, 10 percent slope is LS = 1.39.

10. The improved surface stabilization factor from Table 9-8 for straw or hay mulch applied at a rate of 1.5 tons/acre on a 10 percent slope is $C_s = 0.12$.

11. The runoff control factor from Equation 9-3 with one diversion across the slope is $C_r = 0.707$.

12. The improved control practice factor from Equation 9-2 is computed as

    CP = (0.12) (0.707)

    CP = 0.085

13. The improved soil loss is estimated as

    A = (200) (0.32) (1.39) (0.085)

    A = 7.6 tons/acre/year

14. The improved sediment delivery ratio using Equation 9-4 is

$$D_c = \frac{8-0}{20}$$

$D_c = 0.4$

15. Based on estimated soil loss with improvements, with $D_c \leq 1$, the proposed activity is acceptable.

9.4 Erosion Prevention

Erosion prevention is generally the easiest and least costly way to prevent sediment from leaving the site. It is important to note that if erosion is prevented then controlling sediment is not necessary. Volume 4 Section 1.31 discusses the erosion process including water, stream and channel, wind erosion and factors that influence it. Section 1.6.4 discusses selecting erosion prevention activities.

Following a brief description of temporary and permanent considerations, factors for use with the USLE for these classifications are presented below. Remaining erosion prevention topics covered in this section include slope and channel protection, and outlet protection. All of these practices are discussed in more detail in Volume 4 Sections TCP and PESC.

9.4.1 Temporary and Permanent Considerations

To the maximum extent possible, surface stabilization measures should provide permanent protection once construction is complete. In addition, the layout for temporary runoff control measures should be consistent with the layout of permanent drainage facilities. Additional related information is available in Volume 4 Section 1.5.

9.4.2 Surface Stabilization Factors

Soil stabilization factors for natural or unprotected site conditions can be estimated from published data. For various types of bare soil conditions, C_s factors can be estimated from values reported in Table 9-5. For permanent pasture, rangeland, idle land, and grazed woodland, C_s factors can be estimated from values reported in Table 9-6. For undisturbed woodland, C_s factors can be estimated from values reported in Table 9-7.

Soil stabilization factors for mulches, seeding and vegetation, and chemical binders and tacks are discussed below.

Mulches

Table 9-8 presents mulch surface stabilization factors for selected application rates on construction sites. The principal types of mulching material are straw, hay, and wood chips. Data are also presented for crushed stones. Additional detail is provided in Volume 4 TCP-08.

Seeding and Vegetation

C_s factors for temporary and permanent seedings are presented in Table 9-9. Mechanically disturbed woodland sites with 0 to 80 percent of the site covered with residue and various levels of weed cover can be evaluated using C_s factors from Table 9-10. Suitable vegetative cover plants and plant mixtures are listed in Table 9-11 along with appropriate planting dates and application rates. Additional detail is provided in Volume 4 TCP-05 and PESC-01.

Chemical Binders and Tacks

If construction occurs at a time when conventional vegetative measures are not feasible, or immediate protection is required under adverse conditions, chemical binders and tacks may be suitable. C_s factors for selected forms of these treatments are presented in Table 9-9. Additional detail is provided in Volume 4 CP-17, TCP-08 and 10.

Other Stabilization Practices

Other stabilization practices including buffer zones, filter strips, top soil management, surface roughening, nets, mats, geotextiles, soil bioengineering, and terracing are discussed in Volume 4 TCP-04, 06, 07, 09, 10, 11, 23, PESC-02, 03, 04, and 05.

9.4.3 Exposure Scheduling Factors

The impact of exposure scheduling on the gross soil loss from a site can be determined using the monthly distribution of the rainfall erosion index, which is presented in Figure 9-1. The anticipated exposure schedule can be evaluated by the following procedure:

1. Establish the anticipated sequence of time periods with consistent surface cover conditions.

2. Determine appropriate surface stabilization cover factors (C_s) using data presented in Tables 9-5 through 9-10.

3. Determine the fraction of the annual R value for each time period, using the EI factors from Figure 9-1 (see Section 9.3.1).

4. Multiply the C_s values from Step 2 by the fractions from Step 3.

5. Sum the results from Step 4 for each time period to obtain a composite C_s value for the anticipated construction schedule.

This procedure is demonstrated in Table 9-12. Since a construction schedule is subject to unplanned changes, a worst-case scenario should be considered.

9.4.4 Runoff Control Factors

Quantitative information related to the runoff control (C_r) factor presented in Equation 9-2 is currently available only for diversion structures, since they are the principal means of reducing slope lengths and, thus, erosion. However, this should not limit the usefulness of the USLE as a planning tool for runoff control. Any structure that slows runoff or diverts it away from down-slope areas can benefit erosion prevention. The impact of diversions on gross erosion can be quantified using Equation 9-3, as proposed by Chen (1974).

9.4.5 Slope and Channel Protection

Steep slopes, both natural and cut and fill, have the potential for severe erosion. As a result, slope protection is often required to safely convey upland stormwater runoff to the toe of slopes. Slope and channel protection practices intended to reduce the potential for slope and gully erosion include temporary seeding, surface roughening, mulching, nets, mats, geotextiles, terracing, check dams, diversion: drains, swales and berms and bank stabilization. Appropriate construction specifications should be developed by the design engineer as guided by Volume 4 TCP-19, 20, 21, 22, PESC-06.

9.4.6 Outlet Protection

Design procedures for outlet protection should be consistent with the erosion prevention information for open channels presented in Chapter 3, energy dissipation methods presented in Chapter 10 and additional information provided in Volume 4 TCP-21, 24, 25, PESC-07 and 08. The design should include a plan view, profile, and cross section for each unique channel reach between the storm sewer outlet and the existing publicly maintained system or natural stream channel. The velocity should be indicated for the outlet (pipe, structure, or reinforced channel), riprap or paved apron section, and each successive channel reach from the end of the apron to the point of entry into the existing drainage system or natural stream channel. The plan should indicate the proposed method of stabilizing each channel reach, consistent with computed velocities. The velocity at the end of a structure or channel reach must not exceed the allowable velocity of the next downstream reach.

9.5 Sediment Control

Sediment control measures that can prevent the transport of detached soil from a site include sediment barriers, sediment traps, sediment basins, construction entrance stabilization and related activities. Additional information about these and other related practices for sediment control are presented in Volume 4 Sections TCP, PESC and PTP.

9.5.1 Temporary and Permanent Considerations

To the maximum extent possible, permanent facilities should be phased/scheduled to be used as temporary (construction phase) sediment control facilities. This is a more cost-effective approach than implementing many more small sediment control devices site-wide as the generally larger permanent facilities must be graded and eventually constructed. It must be noted that it may still be necessary to implement some sediment controls in other areas of the site to prevent the permanent facility from being overloaded with sediment. Furthermore, the permanent facility will generally need to be over-excavated to account for the trapped sediment. The outlet structure will need to be reconfigured to perform under the construction phase runoff sediment loadings that generally are significantly higher than post-construction (stabilized site) runoff. Additional related information is available in Volume 4 Section 1.5.

9.5.2 Sediment Barriers

Sediment barriers are intended to intercept and/or filter small volumes of sediment resulting mainly from sheet flow and rill erosion. Typical sediment barrier applications include continuous berms, brush barriers, sand bag barriers, silt fences, straw bale barriers, and inlet barriers. Check dams are similar to sediment barriers in that they slow water in small channels to the point that sediment can settle out of runoff.

In general, sediment barriers have a useful life expectancy of 3 to 6 months, depending on the construction technique. Continuous berms are strongly encouraged because of their installation ease, and minimal maintenance requirements. Straw bales are the least preferred because of the inconsistent materials qualities and very high maintenance considerations. Extreme care should be used when locating sediment barriers and application limitations must be carefully considered. Improper location and installation may result in failure of the barrier, which can cause more damage than the erosion the barrier was intended to prevent. Additional detail is provided in Volume 4 TCP-12, 13, 14, 15, 16 and 24.

9.5.3 Sediment Traps and Basins

Temporary sediment traps are generally formed by constructing a small ponding area behind an embankment and/or gravel outlet. The tributary drainage area and required service life will dictate the sizing of a small trap or temporary basin. However, it should be noted that MWS strongly encourages using permanent facilities with outlet structures configured to manage temporary (construction phase) sediment control (see section 9.5.1). Temporary sediment traps and basins are often constructed in combination with temporary diversion berms or barriers. Additional details about temporary sediment traps and basins are provided in Volume 4 TCP-17 and 18 while information about permanent detention facilities are provided in Volume 4 PTP-01 and 06.

9.5.4 Construction Road and Entrance Management

Soil tracked off the construction site by delivery and other vehicles is a significant problem. Road and entrance management is required to reduce the amount of soil transported from a construction site. At a minimum stone-stabilized entrance pads should be constructed at vehicular traffic entrances and exits to a public road or paved area. When a stabilized pad proves inadequate, a wash rack or additional road stabilization will be required. Wash water runoff should be conveyed to a sediment basin or trap. Additional information is available in Volume 4 TCP-01, 02 and 03.

Table 9-1
ANNUAL RAINFALL FACTOR (R) VALUES FOR 2-, 5-, AND 20-YEAR RETURN PERIODS FOR NASHVILLE AND DAVIDSON COUNTY

Observed R Value Annual Range (22 Years)	Annual R Value for Various Return Period Frequencies		
	2-Year	5-Year	20-Year
116-381	198	262	339

Reference: Wischmeier and Smith (1978).

Table 9-2
EXPECTED SINGLE STORM RAINFALL FACTOR (R) VALUES FOR NASHVILLE AND DAVIDSON COUNTY

Expected Single Storm R Value for Various Return Period Frequencies				
1-Year	2-Year	5-Year	10-Year	20-Year
35	49	68	83	99

Reference: Wischmeier and Smith (1978).

Table 9-3
SCS SOIL ERODIBILITY DATA

Soil Name and Map Symbol	Depth Interval (inches)	Soil Erodibility Factor, K (tons/acre/R unit)
AmB, AmC, AmC3	0-16	0.43
Armour	16-41	0.37
	41-66	0.37
Ar	0-35	0.37
Arrington	35-65	0.32
BbD,* BbE*	0-8	0.17
Barfield	8-15	0.17
	15	--
Rock Outcrop		
BcC, BcD	0-8	0.32
Baxter	8-14	0.24
	14-72	0.24
Be	0-18	0.32
Beason	18-65	0.32
BoD	0-5	0.28
Bodine	5-20	0.28
	20-65	0.28
BsE*	0-5	0.28
Bodine	5-20	0.28
	20-65	0.28
Sulphura	0-5	0.20
	5-26	0.20
BvB	0-7	0.43
Bradyville	7-18	0.28
	18-55	0.27
	55	--
ByB	0-9	0.43
Byler	9-24	0.43
	24-44	0.43
	44-65	0.24

Table 9-3
SCS SOIL ERODIBILITY DATA (continued)

Soil Name and Map Symbol	Depth Interval (inches)	Soil Erodibility Factor, K (tons/acre/R unit)
DeD, DeE	0-6	0.17
Dellrose	6-61	0.24
	61-74	0.24
DkB	0-8	0.43
Dickson	8-25	0.43
	25-44	0.43
	44-65	0.28
Eg	0-22	0.32
Egam	22-56	0.32
	56-75	0.37
GdC	0-10	0.17
Gladeville	10	--
HmC, HmD	0-5	0.37
Hampshire	5-45	0.28
	45-53	0.24
HuB	0-8	0.20
Humphreys	8-55	0.24
	55-62	0.24
Ld	0-11	0.28
Lindell	11-62	0.28
Ln*	0-11	0.28
Lindell	11-62	0.28
Urban Land		
LoB	0-9	0.43
Lomond	9-16	0.37
	16-46	0.32
	46-65	0.28

Table 9-3
SCS SOIL ERODIBILITY DATA (continued)

Soil Name and Map Symbol	Depth Interval (inches)	Soil Erodibility Factor, K (tons/acre/R unit)
McB*	0-7	0.32
Maury	7-24	0.28
	24-48	0.28
	48-65	0.28
Urban Land		
MmC, MmD	0-7	0.20
Mimosa	7-14	0.20
	14-55	0.20
	55	--
MoE3	0-6	0.20
Mimosa	6-55	0.20
	55	--
MrD,* MrE*	0-7	0.20
Mimosa	7-14	0.20
	14-55	0.20
	55	--
Rock Outcrop		
MsD*	0-7	0.20
Mimosa	7-14	0.20
	14-55	0.20
	55	--
Urban Land		
MvC		
Mountview		
Ne	0-6	0.43
Newark	6-43	0.43
	43-60	0.43

Table 9-3
SCS SOIL ERODIBILITY DATA (continued)

Soil Name and Map Symbol	Depth Interval (inches)	Soil Erodibility Factor, K (tons/acre/R unit)
Pt*		
Pits		
RtC*		
Rock Outcrop		
Talbott	0-5	0.37
	5-32	0.24
	32	--
Se	0-7	0.24
Sequatchie	7-32	0.24
	32-65	0.24
SmC	0-6	0.24
Stemley	6-20	0.28
	20-46	0.24
	46-65	0.28
StC, StD	0-8	0.24
Stiversville	8-53	0.24
	53-60	--
	60	--
SvD*	0-8	0.24
Stiversville	8-53	0.24
	53-60	--
	60	--
Urban Land		
Ta	0-7	0.43
Taft	7-22	0.43
	22-61	0.43

Table 9-3
SCS SOIL ERODIBILITY DATA (continued)

Soil Name and Map Symbol	Depth Interval (inches)	Soil Erodibility Factor, K (tons/acre/R unit)
TbC	0-5	0.37
Talbott	5-32	0.24
	32	--
TcC3	0-6	0.37
Talbott	6-32	0.24
	32	--
TrC*	0-5	0.37
Talbott	5-32	0.24
	32	--
Rock Outcrop		
TuC*	0-5	0.37
Talbott	5-32	0.24
	32	--
Wo	0-6	0.37
Wolftever	6-24	0.37
	24-55	0.37
	55-65	0.32

*See description of the map unit for composition and behavior characteristics of the map unit.
Reference: USDA, SCS (1974).

Table 9-4
ESTIMATED RELATIVE SOIL LOSSES FROM SUCCESSIVE EQUAL-LENGTH SEGMENTS OF A UNIFORM SLOPE

Number of Segments, N	Sequence Number of Segment	Fraction of Soil Loss[a] m = 0.5	m = 0.4	m = 0.3
2	1	0.35	0.38	0.41
	2	0.65	0.62	0.59
3	1	0.19	0.22	0.24
	2	0.35	0.35	0.35
	3	0.46	0.43	0.41
4	1	0.12	0.14	0.17
	2	0.23	0.24	0.24
	3	0.30	0.29	0.28
	4	0.35	0.33	0.31
5	1	0.09	0.11	0.12
	2	0.16	0.17	0.18
	3	0.21	0.21	0.21
	4	0.25	0.24	0.23
	5	0.28	0.27	0.25

[a]Derived from the equation:

$$\text{Soil loss fraction} = \frac{i^{m+1} - (i-1)^{m+1}}{N^{m+1}}$$

where:

i = Segment sequence number
m = Slope-length exponent (0.5 for slopes $\geq$ 5 percent, 0.4 for 4 percent slopes, and 0.3 for 3 percent or less)
N = Number of equal-length segments into which the slope was divided.

Reference: Wischmeier and Smith (1978).

Table 9-5
SURFACE STABILIZATION (C_s) FACTORS
FOR BARE SOIL CONDITIONS

Bare Soil Conditions	C_s Factor
Freshly disked to 6-8 inches	1.00
After one rain	0.89
Loose to 12 inches smooth	0.90
Loose to 12 inches rough	0.80
Compacted bulldozer scraped up and down	1.30
Same, except root raked	1.20
Compacted bulldozer scraped across slope	1.20
Same, except root raked across	0.90
Rough irregular tracked all directions	0.90
Seed and fertilizer, fresh	0.64
Same, after 6 months	0.54
Seed, fertilizer, and 12 months chemical	0.38
Not tilled algae crusted	0.01
Tilled algae crusted	0.02
Compacted fill	1.24-1.71
Undisturbed, except scraped	0.66-1.30
Scarified only	0.76-1.31
Sawdust 2 inches deep, disked in	0.61

Reference: Transportation Research Board (1980).

Table 9-6
SURFACE STABILIZATON (C_s) FACTORS FOR PERMANENT PASTURE, RANGELAND, IDLE LAND, AND GRAZED WOODLAND[a]

Vegetable Canopy Type and Height of Raised Canopy[b]	Canopy *Cover*[c] %	*Cover That Contacts the Surface* Type[d]	*Percent Ground Cover* 0	20	40	60	80	95-100
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.091	.043	.011
Canopy of tall weeds or short brush (20-inch fall height)	25	G	.36	.17	.09	.038	.013	.033
		W	.36	.20	.13	.083	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.076	.039	.011
	75	G	.17	.10	.06	.032	.011	.003
		W	.17	.12	.09	.068	.038	.011
Appreciable brush (6.5-ft fall height)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.087	.042	.011
	50	G	.34	.16	.08	.038	.012	.003
		W	.34	.19	.13	.082	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.078	.040	.011
Trees but no Appreciable low brush (13-ft fall height)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.089	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.087	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.084	.041	.011

[a]All values shown assume: (1) random distribution of mulch or vegetation and (2) mulch of appreciable depth where it exists. Idle land refers to land with undisturbed profiles for a period of at least 3 consecutive years. Also to be used for burned forestland and forestland that was harvested less than 3 years before.

[b]Average fall height of water drops from canopy to soil surface.

[c]Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[d]G: Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 inches deep. W: Cover at surface is mostly broadleaf herbaceous plants (such as weeds with little lateral root network near the surface) and/or undecayed residue.
Reference: Wischmeier and Smith (1978).

Table 9-7
SURFACE STABILIZATION (C_s) FACTORS FOR UNDISTURBED WOODLAND

Effective Canopy[a] (% of Area)	Forest Liter [b] (% of Area)	C_s Factor [c]
100-75	100-90	.0001-.001
70-40	85-75	.002-.004
35-20	70-40	.003-.009

[a]Where effective litter cover is less than 40 percent or canopy cover is less than 20 percent, the area should be considered as grassland or idle land, with C_s selected from Table 9-6. Where woodlands are being harvested, grazed, or burned, also use Table 9-6.

[b]Forest litter is assumed to be at least 2 inches deep over the percent ground surface area covered.

[c]The range C_s values is due in part to the range in the percent area covered. In addition, the percent of effective canopy and its height has an effect. Low canopy is effective in reducing raindrop impact and in lowering the C_s factor. High canopy, over 13 meters, is not effective in reducing raindrop impact and will have no effect on the C_s value.

Reference: Wischmeier and Smith (1978).

Table 9-8
MULCH SURFACE STABILIZATION (C_s) FACTORS AND LENGTH LIMITS FOR CONSTRUCTION SLOPES[a]

Type of Mulch	Mulch Rate (tons/acre)	Land Slope (%)	C_s Factor	Length Limit [b] (ft)
None	0	All	1.0	--
Straw or hay, tied down by anchoring and tacking equipment[c]	1.0	1-5	0.20	200
	1.0	6-10	.20	100
	1.5	1-5	.12	300
	1.5	6-10	.12	150
	2.0	1-5	.06	400
	2.0	6-10	.06	200
	2.0	11-15	.07	150
	2.0	16-20	.11	100
	2.0	21-25	.14	75
	2.0	26-33	.17	50
	2.0	34-50	.20	35
Crushed stone, ¼ to 1½ in	135	<16	.05	200
	135	16-20	.05	150
	135	21-33	.05	100
	135	34-50	.05	75
	240	<21	.02	300
	240	21-33	.02	200
	240	34-50	.02	150
Wood chips	7	<16	.08	75
	7	16-20	.08	50
	12	<16	.05	150
	12	16-20	.05	100
	12	21-33	.05	75
	25	<16	.02	200
	25	16-20	.02	150
	25	21-33	.02	100
	25	34-50	.02	75

[a]Developed by interagency workshop group on the basis of field experience and limited research data.

[b]Maximum slope length for which the specified mulch rate is considered effective. When this limit is exceeded, either a higher application rate or mechanical shortening of the effective slope length is required.

[c]When the straw or hay mulch is not anchored to the soil, C_s values on moderate or steep slopes or on soils having K values greater than 0.30 should be taken at double the values given in this table.

Reference: Wischmeier and Smith (1978).

Table 9-9
SURFACE STABILIZATION (C_s) FACTORS FOR SELECTED METHODS OF SURFACE STABILIZATION

Surface Stabilization Method	C_s Factor
Asphalt Emulsion	
1,250 gallons/acre	0.02
1,210 gallons/acre	0.01-0.019
605 gallons/acre	0.14-0.57
302 gallons/acre	0.28-0.60
151 gallons/acre	0.65-0.70
Dust Binder	
605 gallons/acre	1.05
1,210 gallons/acre	0.29-0.78
Other Chemicals	
1,000-lb fiberglass roving with 60-150 gallons/acre	0.01-0.05
Aquatain	0.68
Aerospray 70, 10 percent cover	0.94
Curasol AE	0.30-0.48
Petroset SB	0.40-0.66
PVA	0.71-0.90
Terra-Tack	0.66
Wood fiber slurry[a], 1,000 lb/acre fresh	0.05
Wood fiber slurry[a], 1,400 lb/acre fresh	0.01-0.02
Wood fiber slurry[a], 3,500 lb/acre fresh	0.10
Seedings[b]	
Temporary, 0 to 60 days[c]	0.40
Temporary, after 60 days	0.05
Permanent, 0 to 60 days[c]	0.40
Permanent, 2 to 12 months	0.05
Permanent, after 12 months	0.01
Brush	0.35
Excelsior Blanket With Plastic Net	0.04-0.10

[a]Wood fiber slurry is commonly referred to as hydromulch.

[b]Use minimum C_s values if plantings are performed with mulches.

[c]If dry weather occurs at planting and emergence is delayed, extend the 0-60 days to a period when rainfall normally occurs.

Reference: Transportation Research Board (1980).

Table 9-10
SURFACE STABILIZATION (C_S) FACTORS FOR
MECHANICALLY DISTURBED WOODLAND SITES

Percent of Soil Covered With Residue in Contact With Soil Surface	Soil Condition[a] and Weed Cover[b]							
	Excellent		*Good*		*Fair*		*Poor*	
	NC	WC	NC	WC	NC	WC	NC	WC
None								
Disked, raked, or bedded[c,d]	.52	.20	.72	.27	.85	.32	.94	.36
Burned[e]	.25	.10	.26	.10	.31	.12	.45	.17
Drum chopped[e]	.16	.07	.17	.07	.20	.08	.29	.11
10% Cover								
Disked, raked, or bedded[c,d]	.33	.15	.46	.20	.54	.24	.60	.26
Burned[e]	.23	.10	.24	.10	.26	.11	.36	.16
Drum chopped[e]	.15	.07	.16	.07	.17	.08	.23	.10
20% Cover								
Disked, raked, or bedded[c,d]	.24	.12	.34	.17	.40	.20	.44	.22
Burned[e]	.19	.10	.19	.10	.21	.11	.27	.14
Drum chopped[e]	.12	.06	.12	.06	.14	.07	.18	.09
40% Cover								
Disked, raked, or bedded[c,d]	.17	.11	.23	.14	.27	.17	.30	.19
Burned[e]	.14	.09	.14	.09	.15	.09	.17	.11
Drum chopped[e]	.09	.06	.09	.06	.10	.10	.11	.07
60% Cover								
Disked, raked, or bedded[c,d]	.11	.08	.15	.11	.18	.14	.20	.15
Burned[e]	.08	.06	.09	.07	.10	.08	.11	.08
Drum chopped[e]	.06	.05	.06	.05	.07	.05	.07	.05
80% Cover								
Disked, raked, or bedded[c,d]	.05	.04	.07	.06	.09	.08	.10	.09
Burned[e]	.04	.04	.05	.04	.05	.04	.06	.05
Drum chopped[e]	.03	.03	.03	.03	.03	.03	.04	.04

Notes for Table 9-10

[a]Excellent: Highly stable soil aggregates in topsoil with fine tree roots and litter mixed in.

Good: Moderately stable soil aggregates in topsoil or highly stable aggregates in subsoil (topsoil removed during raking), with only traces of litter mixed in.

Fair: Highly unstable soil aggregates in topsoil or moderately stable aggregates in subsoil, with no litter mixed in.

Poor: No topsoil, highly erodible soil aggregates in subsoil, with no litter mixed in.

[b]NC—No live vegetation.
WC—75 percent cover of grass and weeds, having an average drop fall height of 20 inches. For intermediate percentages of cover, interpolate between columns.

[c]Multiply Item A values by the following values to account for surface roughness:

Very rough, major effect on runoff and sediment storage, depressions greater than 6 inches	.40
Moderate	.65
Smooth, less than 2 inches	.90

[d]The C_s values for Item A are for the first year following treatment. For A-type sites 1 to 4 years old, multiply C_s value by .7 to account for aging. For sites 4 to 8 years old, use Table 9-6. For sites more than 8 years old, use Table 9-7.

[e]The C_s values for B and C areas are for the first 3 years following treatment. For sites treated 3 to 8 years ago, use Table 9-6. For sites treated more than 8 years ago, use Table 9-7.

Reference: Wischmeier and Smith (1978).

Table 9-11
GUIDELINES FOR SELECTING VEGETATIVE COVER

Plant or Plant Mixture	Application Rate Per Acre[a]	Plant Dates[b]
Temporary Plants		
1. Rye	3 bushels	Aug. 15 – Nov. 1
2. Wheat	2-3 bushels	Sept. 1 – Nov. 1
3. Annual Ryegrass	30 pounds	Aug. 15 – Nov. 1
4. Browntop or Pearl Millet	20 pounds	Apr. 15 – Jul. 15
5. Sudangrass	40 pounds	Apr. 1 – Jul. 15
Permanent Plant Mixtures		
1. Tall Fescue (Ky 31)	45 pounds	Feb. 15 – Apr. 15
White Clover[c]	3 pounds	Jul. 15 – Oct. 15
2. Crownvetch[d]	20 pounds	Feb. 15 – Apr. 15
Tall Fescue (Ky 31)	30 pounds	Aug. 15 – Oct. 15
3. Sericea Lespedeza (Scarified)	45 pounds	Mar 1. – Jul. 15
Tall Fescue (Ky 31)	20 pounds	
Annual Lespedeza (Kobe)	8 pounds	
4. Sericea Lespedeza (Scarified)	45 pounds	Apr. 15 – Jul. 15
Weeping Lovegrass	3 pounds	
5. Common Bermudagrass (Hulled)	14 pounds	Apr. 15 – Jul. 15
Annual Lespedeza (Kobe)	8 pounds	
Permanent Sprig Plants		
1. Midland or Tifton 44 Bermudagrass	30 cubic feet, machine set; 50 cubic feet, broadcast & disked	Acceptable Dates Should Be Confirmed with Local Extension Office

[a]Soil testing should be performed and evaluated by an agronomist to determine soil treatment requirements for parameters such as pH, nitrogen, phosphorus, potassium, and other factors

[b]Seed should be irrigated during dry periods.

[c]Inoculate clover.

[d]Inoculate crownvetch with special inoculant. When seeded with hydroseeder, use 10 times the amount of inoculant stated on the package for non-hydroseeder application.

Reference: USDA, SCS (1978).

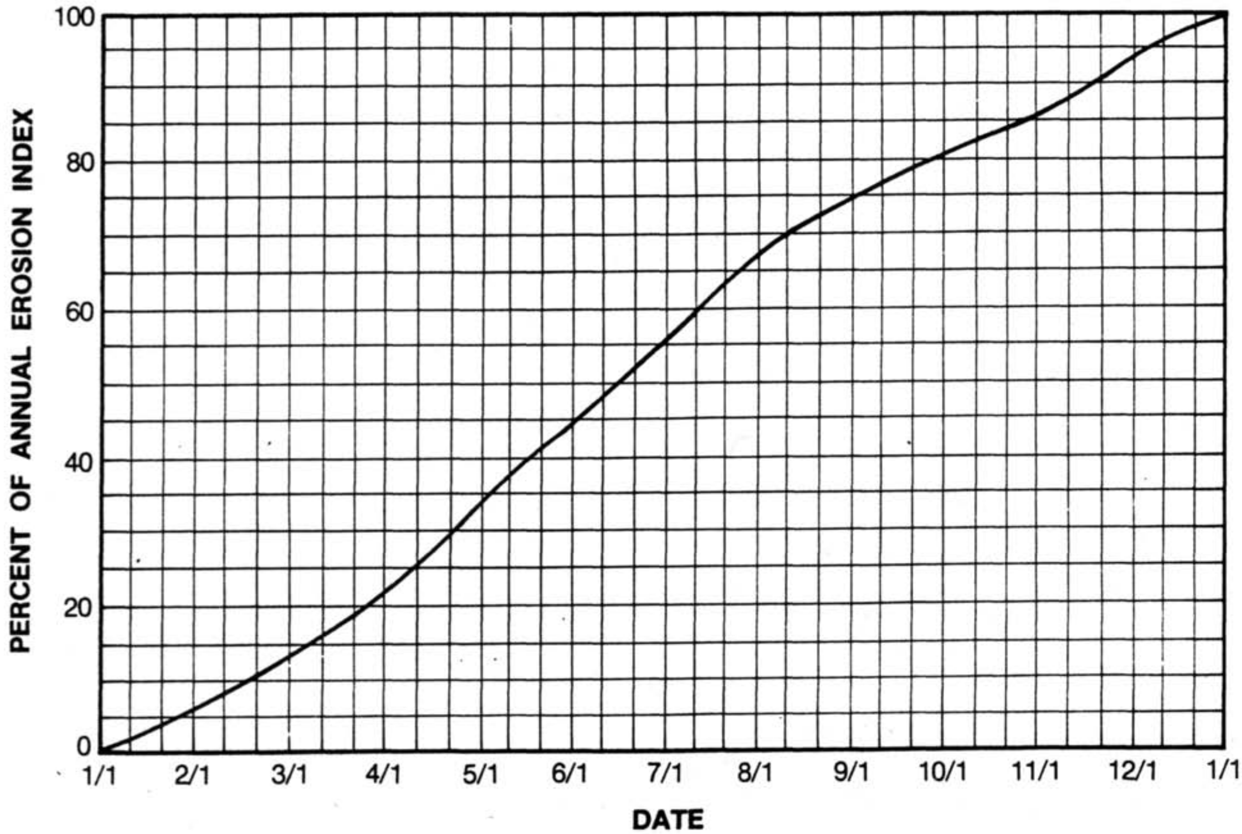
Table 9-12
EXAMPLE CALCULATION OF THE SURFACE STABILIZATION (C_s)
FACTOR FOR EXPOSURE SCHEDULING

Time Period	Surface Cover	C_s Factor	Fraction of Annual R During Time Period[a]	Weighted C_s Factor [b]
1/1 - 4/1	Undisturbed Woodland	0.003	0.22	0.0007
4/1 – 6/1	Cleared Site	1.00	0.23	0.23
6/1 – 8/1	Temporary Seeding	0.40	0.22	0.088
8/1 – 12/31	Permanent Seeding	0.05	0.33	0.017

Note: Composite C_s for exposure scheduling is the sum of each weighted C_s factor or 0.336.

[a]Obtained from Figure 9-1.

[b]Product of the C_s factor and the fraction of annual R during the specified time period.



Reference: Transportation Research Board (1980).

Figure 9-1
Erosion Index (EI) Distribution Curve
Applicable to Nashville and Davidson County

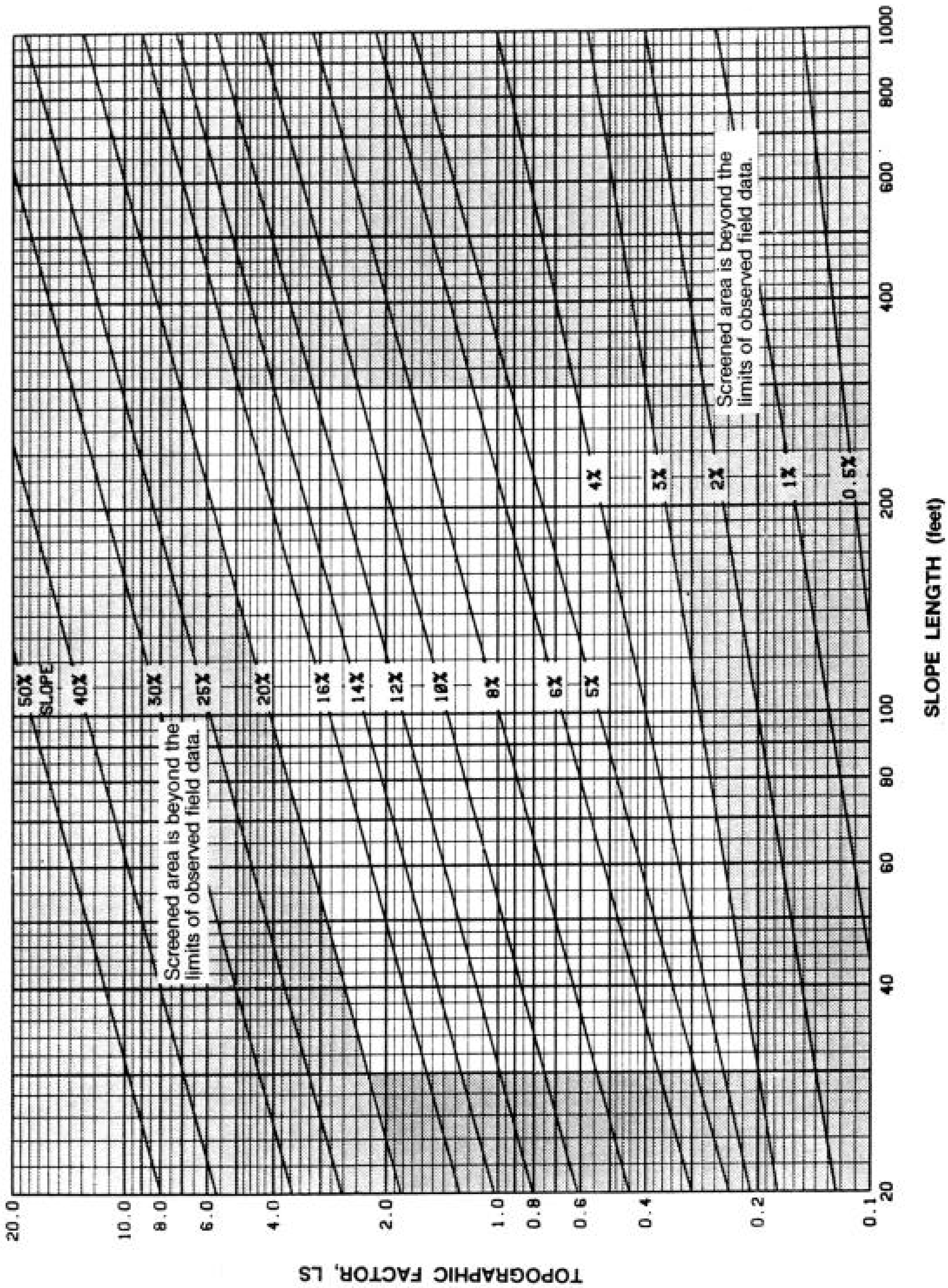


Figure 9-2
Length-Slope Factor Chart for USLE

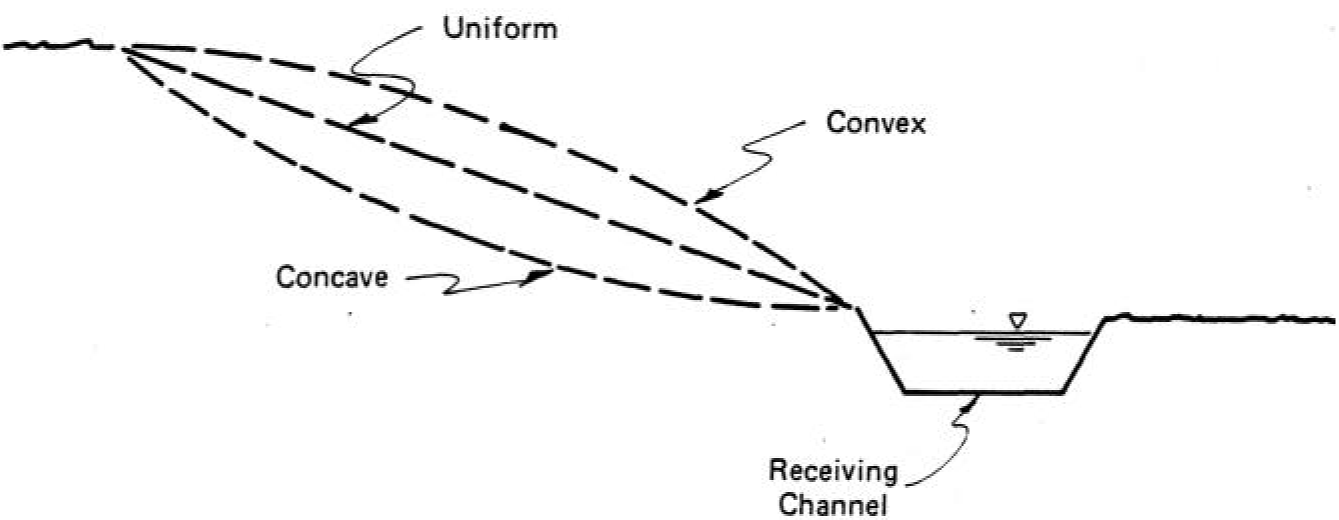


Figure 9-3
Conceptual Sketch of Typical Concave and Convex Slopes

CHAPTER 10
OUTLET PROTECTION

Chapter 10
OUTLET PROTECTION

Synopsis

Transitions from closed conduit or other flow concentrating facilities to natural channel systems often create high velocities and erosive flow conditions, which must generally be mitigated with facilities that prevent excessive erosion and scour. This chapter provides a general procedure to identify cases when outlet protection may be required, as well as selection criteria and design details for protection facilities. Key references for the information presented in this chapter are USDOT, FHWA, HEC-14 (1983), U.S. Department of the Interior (1978), and USDA, SCS (1975).

While not presented in this chapter FHWA-IP-89-016 (HEC-11) (Brown and Clyde, 1989) should be reviewed for discussions on recognizing erosion potential, erosion mechanisms and rip rap failure modes, riprap types including rock riprap, gabions, preformed blocks, grouted rock, and paved linings, design discharge, flow types, channel geometry, flow resistance, extent of protection and toe depth.

Only outlet protection is addressed in this chapter. Additional temporary and permanent erosion prevention measures may be important to provide stability for other parts of the overall drainage system. Methods for reducing erosion and channel lining or stabilization are discussed in Chapters 3, 9 and Volume 4 Section TCP and PESC.

The general procedure for outlet protection selection and design is presented in Section 10.1. Recommended methods for estimating outlet erosion and scour potential are included in Section 10.2. Design details for riprap aprons, riprap outlet basins, baffled outlets, and U.S. Bureau of Reclamation (USBR) Type II basins are presented in Sections 10.3, 10.4, 10.5, and 10.6, respectively.

10.1 General Procedure

The following procedure is generally applicable for outlet protection facilities:

1. Prepare appropriate input data.

    a. Culvert and other terminal outlet structures
        (1) Design capacity
        (2) Type of control
        (3) Barrel slope
        (4) Outlet depth

(5) Outlet velocity
(6) Length
(7) Tailwater
(8) Froude number

b. Channel
(1) Capacity
(2) Bottom slope
(3) Cross section dimensions
(4) Normal depth
(5) Average velocity
(6) Allowable velocity
(7) Debris and bedload
(8) Soil plasticity index
(9) Saturated shear strength

c. Allowable scourhole dimensions, based on site conditions
(1) Depth, h_s
(2) Width, W_s
(3) Length, L_s
(4) Volume, V_s

2. Compute local scourhole dimensions with the procedure in Section 10.2. A nonerodible layer (e.g., bedrock) may limit scourhole depth but only slightly affect scourhole width and length.

3. Compare the local scourhole dimensions from Step 2 to the allowable scourhole dimensions from Step 1. If the allowable dimensions are exceeded, outlet protection is required.

4. If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are presented in Table 10-1. When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered. Applicable conditions for each outlet protection measure are briefly summarized below.

   a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Froude number between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

d. USBR Type II basins may prove economical when the theoretical dissipation velocity is 50 feet per second or greater. These basins rely upon flow expansion to create an efficient hydraulic jump for energy dissipation. A USBR Type II basin may be desirable when the structural requirements of a baffled outlet become prohibitive.

5. If outlet protection is not required, dissipate energy through formation of a local scourhole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scourhole depth, h_s.

6. Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur.

10.2 Local Scourhole Estimation

Estimates of erosion at culvert outlets must consider factors such as discharge, culvert diameter, soil type, duration of flow, and tailwater depth. In addition, the magnitude of the total erosion can consist of local scour and channel degradation.

Empirical equations for estimating the maximum dimensions of a local scourhole are presented in Table 10-2. These equations are based on test data obtained as part of a study conducted at Colorado State University (USDOT, FHWA, HEC-14, 1983). A form for recording the following local scourhole computations is presented in Table 10-3:

1. Prepare input data, including:

   Q = Design discharge, in cfs

   For circular culvert, D = diameter, in inches

For other shapes, use the equivalent depth

$y_e = (A/2)^{1/2}$

t = Time of scour, in minutes

v_o = Outlet mean velocity, in feet/second

$\tau_c = 0.0001\ (S_v + 180) \tan (30 + 1.73\ PI)$ (10-1)

where:

τ_c = Critical tractive shear stress, in pounds/square inch

S_v = Saturated shear strength, in pounds/square inch

PI = Plasticity index from Atterburg limits

The time of scour should be based on a knowledge of peak flow duration. As a guideline, a time of 30 minutes is recommended. Tests indicate that approximately 2/3 to 3/4 of the maximum scour occurs in the first 30 minutes of the flow duration.

2. Based on the channel material, select the proper scour equations and coefficients from Table 10-2.

3. Using the results from the equations selected in Step 2, compute the following scourhole dimensions:

Depth, h_s

Width, W_s

Length, L_s

Volume, V_s

Observations indicate that a nonerodible layer at a depth less than h_s below the pipe outlet affects only scourhole depth. The width, W_s, and the length, L_s, may still be computed using the equations in Table 10-2.

10.3 Riprap Aprons

A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5.

10.3.1 Design Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d_{50}. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts.

The procedure consists of the following steps:

1. If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 10-1 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 10-2 should be used.

2. Determine the correct apron length and median riprap diameter, d_{50} using the appropriate curves from Figures 10-1 and 10-2. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 10-3.

    a. For pipes flowing full:

       Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, L_a, and median riprap diameter, d_{50} from the appropriate curves.

    b. For pipes flowing partially full:

       Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, L_a, from the scale on the left.

    c. For box culverts:

Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, L_a, using the scale on the left.

3. If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 10-3. This will provide protection under either of the tailwater
4. conditions.

10.3.2 Design Considerations

The following items should be considered during riprap apron design:

1. The maximum stone diameter should be 1.5 times the median riprap diameter. The riprap depth should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater.

2. The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, d_w. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height and should taper to the flat surface at the end of the apron.

3. If there is a well-defined channel, the apron length should be extended as necessary so that the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.

4. If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.

5. The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

10.3.3 Example Problems

<u>Example 10-1. Riprap Apron Design for Minimum Tailwater Conditions</u>

A flow of 280 cfs discharges from a 66-inch pipe with a tailwater of 2 feet above the pipe invert. Find the required design dimensions for a riprap apron.

1. Compute 0.5 d_o = 2.75 feet.

2. Since TW = 2 feet, use Figure 10-1 for minimum tailwater conditions.

3. By Figure 10-1, the apron length, L_a, and median stone size, d_{50} are 38 feet and 1.2 feet, respectively.

4. The downstream apron width equals the apron length plus the pipe diameter:

   $W = d + L_a = 5.5 + 38 = 43.5$ feet

5. Maximum riprap diameter is 1.5 times the median stone size;

   $1.5\ (d_{50}) = 1.5\ (1.2) = 1.8$ feet

6. Riprap depth = $1.5\ (d_{max}) = 1.5\ (1.8) = 2.7$ feet.

<u>Example 10-2. Riprap Apron Design for Maximum Tailwater Conditions</u>

A concrete box culvert 5.5 feet high and 10 feet wide conveys a flow of 600 cfs at a depth of 5.0 feet. Tailwater depth is 5.0 feet above the culvert outlet invert. Find the design dimensions for a riprap apron.

1. Compute $0.5\ (d_o) = 0.5\ (5.0) = 2.5$ feet.

2. Since TW = 5.0 feet is greater than 2.5 feet, use Figure 10-2 for maximum tailwater conditions.

$$v = Q / A = \frac{600}{(5)(10)} = 12 \text{ feet/second}$$

3. On Figure 10-2, at the intersection of the curve, d_o = 60 inches and v = 12 feet/second, d_{50} = 0.4 foot.

   Reading up to the intersection with d = 60 inches, find L_a = 40 feet.

5. Apron width downstream = $d_w + 0.4\ L_a = 10 + 0.4\ (40) = 26$ feet.

5. Maximum stone diameter = $1.5\ d_{50} = 1.5\ (0.4) = 0.6$ feet.

6. Riprap depth = 1.5 d_{max} = 1.5 (0.6) = 0.9 feet.

10.4 Riprap Outlet Basins

A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump. The discussion is based on data obtained from a study conducted at Colorado State University (USDOT, FHWA, HEC-14, 1983). A detailed schematic diagram of a riprap outlet basin is presented in Figure 10-4.

10.4.1 Design Procedure

A form for recording the following riprap outlet basin computations is presented in Table 10-4:

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that $TW/y_o \leq 0.75$ for design discharge.

2. For subcritical flow conditions (culvert set on mild or horizontal slope), use Figure 10-5 or 10-6 to obtain y_o/D, then obtain v_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert. If the culvert is on a steep slope, v_o will be the normal velocity obtained by using Manning's Equation for appropriate slope, section, and discharge (see Chapter 3).

3. Compute Fr for brink conditions ($y_e = (A/2)^{1/2}$). Select the d_{50}/y_e value appropriate for available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 10-7, and check to see that $2 \leq h_s/d_{50} \leq 4$. Repeat computations if h_s/d_{50} falls out of this range.

4. Size basin using details shown in Figure 10-4.

5. Where the allowable exit velocity for the riprap basin is exceeded:

    a. Determine the average normal flow depth in the natural channel for the design discharge.

    b. Extend the length of the riprap basin (if necessary) so that the width of the basin at section A-A of Figure 10-4 times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

6. In the exit region of the basin, warp (or transition) the walls and apron of the basin so that the cross section of the basin at the exit conforms to the cross section of the natural

channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

7. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

   a. Design a conventional basin for low tailwater conditions in accordance with the instructions above.

   b. Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 10-8.

   c. Shape downstream channel and size riprap using Figure 10-9 and the stream velocities obtained from Figure 10-8.

10.4.2 Design Considerations

Riprap outlet basin design should include a consideration of the following additional items:

1. The dimensions of a scourhole in a basin constructed with angular rock can be approximately the same as the dimensions of a scourhole in a basin constructed of rounded material when rock size and other variables are similar.

2. When the ratio of tailwater depth to brink depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed as it leaves the basin.

3. The surface of the riprapped floor of the energy dissipating pool is constructed at an elevation, h_s, below the culvert invert. This elevation is the approximate depth of scour that would occur in a thick pad of riprap constructed at the outfall of the culvert, if subjected to the design discharge. The ratio of h_s to d_{50} of the material should range from 2 to 4.

4. The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole is enlarged.

5. For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jetlike character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the

scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

6. The length of the energy dissipating pool is 10 (h_s) or $3W_o$, whichever is larger. The overall length of the basin is 15 (h_s) or $4W_o$, whichever is larger.

7. It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.

8. Filter material should be considered to prevent the migration of streambed material through the riprap. Bank material adjacent to a culvert is not subjected to flow for long continuous periods. Also, the streambed material may be sufficiently well graded and not require a filter. If some siltation of the basin accompanied by plant growth is anticipated, a filter may not be required. If required, a filter cloth or filter material should be specified.

9. Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow in Chapter 3. If required, riprap lined transitions should be designed as outlined in Chapter 3.

10.4.3 Example Problems

<u>Example 10-3. Riprap Outlet Basin Design for Supercritical Flow and Minimum Tailwater Conditions</u>

An 8-foot by 6-foot box culvert conveys a supercritical flow of 800 cfs. The normal flow depth and the equivalent brink depth (y_e) both equal 4 feet. Tailwater depth is estimated to be 2.8 feet. Find the dimensions of a riprap outlet basin for these conditions.

1. For a rectangular section, $y_o = y_e = 4$ feet.

2. Compute the outlet velocity.

   $v_o = Q/A = 800/(4)\ (8) = 25$ feet/second

3. Use the outlet velocity to compute the Froude number,

   $Fr = v_o/[(32.2)\ (y_e)]^{1/2}$

   $Fr = 25/[(32.2)\ (4)]^{1/2} = 2.20$

4. Determine the ratio of the tailwater depth and equivalent brink depth.

   $TW/y_e = 2.8/4.0 = 0.7$

   $TW/y_e < 0.75$ OK

5. Try $d_{50}/y_e = 0.45$, $d_{50} = (0.45)\ (4) = 1.80$ feet

   From Figure 10-7, $h_s/y_e = 1.6$

   $h_s = (4)\ (1.6) = 6.4$ feet

   $h_s/d_{50} = 6.4/1.8 = 3.6$ feet

   $2 \leq h_s/d_{50} \leq 4$ OK

6. Determine the required pool length as the larger of the following:

   a. $L_s = (10)\ (6.4) = 64$ feet

   b. $L_s = (3)\ (W_o) = (3)\ (8) = 24$ feet
      Use L = 64 feet.

7. Determine the required overall apron length as the larger of the following:

   a. $L_B = (15)\ (6.4) = 96$ feet

   b. $L_B = (4)\ (W_o) = (4)\ (8) = 32$ feet

   c. Use B = 96 feet.

8. Other basin dimensions are designed in accordance with details shown in Figure 10-4.

Example 10-4. Riprap Outlet Basin Design for Supercritical Flow and Maximum Tailwater Conditions with Excessive Outlet Velocity

An 8-foot by 6-foot box culvert conveys a supercritical flow of 800 cfs. The normal depth and the equivalent brink depth (y_e) are both equal to 4 feet. The tailwater depth is 4.2 feet and the downstream channel can tolerate a maximum velocity of 7 feet per second. Find the dimensions of a riprap outlet basin for these conditions.

Note—High tailwater depth, $TW/y_o = 1.05 > 0.75$.

1. Design riprap basin using Steps 1-7 in Example 10-3.

   d_{50} = 1.8 feet

   h_s = 6.4 feet

   L_s = 64 feet

   L_B = 96 feet

2. Design riprap for downstream channel. Use Figure 10-8 for estimating average velocity along the channel. Compute equivalent circular diameter, D_e, for brink area.

   $A = \pi D_e{}^2/4 = (y_o)(W_o) = (4)(8) = 32$ square feet

   $D_e = [32 (4) / \pi]^{1/2}$

   D_e = 6.4 feet
   v_o = 25 feet/second (Example 10-3)

L/D_e	L (ft)	v_L/v_o (from Figure 10-8)	v_L ft/sec	Rock Size d_{50} (ft) (from Figure 10-9)
10	64	0.59	14.7	1.4
15	96	0.36	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

3. Riprap should be at least the size shown. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 feet downstream from the culvert brink.

Example 10-5. Riprap Outlet Basin Design for Subcritical Flow Conditions

A 6-foot diameter CMP culvert conveys a subcritical flow of 135 cfs with a normal depth of 4.5 feet and a normal velocity of 5.9 feet per second. The associated slope is 0.004 and Manning's n is 0.024. For a tailwater depth of 2.0 feet, find the dimensions of a riprap outlet basin.

1. Determine the outlet depth, y_o, and the outlet velocity, v_o.

   $Q/D^{2.5} = 135/(6)^{2.5} = 1.53$

$TW/D = 2.0/6 = 0.33$

From Figure 10-6, $y_o/D = 0.45$

$y_o = (0.45)\ (6) = 2.7$ feet

$TW/y_o = 2.0/2.70 = 0.74$

$TW/y_o < 0.75$ OK

Find the brink area, A, for $y_o/D = 0.45$.

$A = (0.343)\ (36) = 12.3$ square feet (0.343 is from Chapter 3)

$v_o = Q/A = 135/12.3 = 11.0$ feet/second

2. Compute the equivalent brink depth.

   $y_e = (A/2)^{0.5} = (12.3/2)^{0.5} = 2.48$ feet

3. Compute the outlet Froude number.

   $Fr_o = v_o/[(32.2)\ (y_e)]^{0.5}$

   $Fr_o = 11/[(32.2)\ (2.48)]^{0.5} = 1.23$

4. Try $d_{50}/y_e = 0.25$.

   $d_{50} = (0.25)\ (2.48) = 0.62$ feet

   From Figure 10-7,

   $h_s/y_e = 0.75$

   $h_s = (0.75)\ (2.48) = 1.86$ feet

   Check: $h_s/d_{50} = 1.86/0.62 = 3$, $2 \leq h_s/d_{50} \leq 4$ OK

5. Compute the pool length as the larger of the following:

   a. $L_s = (10)\ (h_s) = (10)\ (1.86) = 18.6$ feet

   b. $L_s = (3)\ (W_o) = (3)\ (6) = 18$ feet

Use L_s = 18.6 feet.

6. Compute the overall apron length as the larger of the following:

   a. $L_B = (15)\,(h_s) = (15)\,(1.86) = 27.9$ feet

   b. $L_B = (4)\,(W_o) = (4)\,(6) = 24$ feet

   Use L_B = 27.9 feet.

7. d_{50} = 0.62 feet; use d_{50} = 8 inches.

8. Other basin dimensions are assigned in accordance with details shown in Figure 10-4.

10.5 Baffled Outlets

The baffled outlet is a boxlike structure with a vertical hanging baffle and an end sill, as shown in **Figure 10-10**. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

10.5.1 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U.S. Department of the Interior (1978). The dimensions of a baffled outlet as shown in Figure 10-10 should be calculated as follows:

1. Determine input parameters, including:

   h = Energy head to be dissipated, in feet (can be approximated as the difference between channel invert elevations at the inlet and outlet)

   Q = Design discharge, in cfs

   v = Theoretical velocity, in feet/second = $\sqrt{2gh}$

   A = Q/v = Flow area, in square feet

$d = \sqrt{A}$ = Representative flow depth entering the basin, in feet (assumes square jet)

$F_r = v / \sqrt{gd}$ = Froude number, dimensionless

2. Calculate the minimum basin width, W, in feet, using the following equation, which is shown graphically in Figure 10-11:

$$W/d = 2.88Fr^{0.566} \qquad (10\text{-}2)$$

or

$$W = 2.88dFr^{0.566} \qquad (10\text{-}3)$$

where

W = Minimum basin width, in feet

d = Depth of incoming flow, in feet

$F_r = v / \sqrt{gd}$ = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

3. Calculate other basin dimensions as shown in Figure 10-10, as a function of W. Standard construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

4. Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum) , and depth f (W/6). The side slopes should be 1.5:1, and median rock diameter should be at least W/20.

5. Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

6. Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 feet per second flowing full.

7. If the entrance pipe slopes downward, the outlet pipe should be turned horizontal for at least 3 feet before entering the baffled outlet.

8. If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent of diameter approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

10.5.2 Example Problem

Example 10-6. Baffled Outlet Basin Design

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 30 feet, and a tailwater depth, TW, of 3 feet above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

1. Compute the theoretical velocity from

$$v = \sqrt{2gh} = \sqrt{2(32.2 \text{ ft/sec}^2)(30 \text{ ft})}$$

$v = 43.95$ feet/second

This is less than 50 feet/second, so a baffled outlet is suitable.

2. Determine the flow area using the theoretical velocity as follows:

$$A = \frac{Q}{v} = \frac{150 \text{ cfs}}{43.95 \text{ ft/sec}} = 3.41 \text{ squarefeet}$$

3. Compute the representative flow depth using the area from Step 2.

$$d = \sqrt{A} = \sqrt{3.41 \text{ ft}^2} = 1.85 \text{ feet}$$

4. Compute the Froude number using the results from Steps 2 and 3.

$$F_r = \frac{v}{\sqrt{gd}} = \frac{43.95 \text{ ft/sec}}{\sqrt{(32.2 \text{ ft/sec}^2)(1.85 \text{ ft})}} = 5.7$$

5. Determine the basin width using Equation 10-3 with the Froude number from Step 4.

$W = 2.88\ dFr^{0.566}$

$W = 2.88\ (1.85)\ \ (5.7)^{0.566}$

W = 14.27 feet (minimum)

Use 14 feet, 4 inches as design width.

6. Compute the remaining basin dimensions (as shown in Figure 10-10):

L = 4/3 (W) = 19.1 feet
Use L = 19 feet, 2 inches

f = 1/6 (W) = 2.39 feet
Use f = 2 feet, 5 inches

e = 1/12 (W) = 1.19 feet
Use e = 1 foot, 3 inches

H = 3/4 (W) = 10.75 feet
Use H = 10 feet, 9 inches

a = 1/2 (W) = 7.17 feet
Use a = 7 feet, 2 inches

b = 3/8 (W) = 5.38 feet
Use b = 5 feet, 5 inches

c = 1/2 (W) = 7.17 feet
Use c = 7 feet, 2 inches

Baffle opening dimensions would be calculated from f as shown in Figure 10-10.

7. Basin invert should be at

$\frac{b}{2}$ + f below tailwater, or

$\frac{5\ feet, 5 inches}{2}$ + 2 feet, 5 inches = 5.125 feet

Use 5 feet 2 inches; therefore, invert should be 2 feet, 2 inches below ground surface.

8. The riprap transition from the baffled outlet to the natural channel should be 14 feet, 4 inches long by 14 feet, 4 inches wide by 2 feet, 5 inches deep (W x Wx f). Median rock diameter should be of diameter W/20, or about 9 inches.

9. Inlet pipe diameter should be sized for an inlet velocity of about 12 feet/second.

$$\frac{pd^2}{4} = \frac{Q}{v} \quad ; \qquad d = \sqrt{\frac{4Q}{pv}} = \sqrt{\frac{4(150 \text{ cfs})}{p\,(12 \text{ ft/sec})}} = 3.99 \text{ feet}$$

Use 48-inch pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 inches.

10.6 U.S. Bureau of Reclamation Type II Outlet Basin

The Type II Basin was developed by the USBR based on model studies and evaluation of existing basins. The basin elements are shown in Figure 10-12. Chute blocks and a dentated sill are used, but because the useful range of the basin involves relatively high velocities entering the jump, baffle blocks are not employed.

10.6.1 Supercritical Flow Expansion

For expansions where the exit Fr is greater than 3, the location of the section being considered is greater than three culvert diameters away from the outlet, and S_o is less than 10 percent, the energy equation can be used to determine flow conditions leaving the transition. Normally, these parameters would be used as the input values for a basin design. For conditions outside these limits, more appropriate values must be used.

The expansion shown in Figure 10-13 is used to convert depth or potential energy at the culvert outlet to kinetic energy by allowing the flow to expand, drop, or both. The results are that the depth decreases, the velocity increases, and Fr increases. The higher Fr results in a more efficient jump and a shorter basin is required. All design dimensions are defined graphically in Figure 10-13.

The energy balance is written from the culvert outlet, section 0, to the basin, section 1 (see Figure 10-13). Substituting $Q/y_1 W_B$ for v_1 and solving for Q results in:

$$Q = y_1 W_B \left[2g(z_o - z_1 + y_o - y_1) + v_o^{\,2}\right]^{0.5} \qquad (10\text{-}4)$$

where:

Q = Design discharge, in cfs

v_o = Outlet velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

and the remaining dimensions are defined in Figure 10-13

This expression has three unknowns: y_1, W_B,and z_1. The depth, y_1, can be determined by trial and error if W_B and z_1 are assumed. The width, W_B, should be limited to the width that a jet would flare naturally in the slope distance, L, as expressed below:

$$W_B < W_o + 2L_T\left(\sqrt{S_T{}^2 + 1}\right) / 3(Fr_o) \qquad (10\text{-}5)$$

where:

Fr = Outlet Froude number

and the remaining terms are as defined in Figure 10-13

Since the flow is supercritical, the trial y_1 value should start near zero and increase until the design Q is reached. This depth, y_1, is used to find the sequent depth, y_2, using the hydraulic jump equation:

$$y_2 = C_1 y_1 \left[\sqrt{1 + 8Fr^2} - 1\right] / 2 \qquad (10\text{-}6)$$

where:

$C_1 = TW/y_2$

For USBR basins, C_1 is found using the procedure in Section 10.6.2. The above value of $y_2 + z_2$ must be equal to or less than TW + z_3 for the jump to occur. To perform this check, z_3 is obtained graphically or by using the following expressions:

$$L_T = (z_o - z_1)/S_T \qquad (10\text{-}7)$$

$$L_s = (z_3 - z_2)/S_s \qquad (10\text{-}8)$$

$$L_B = f(y_1, Fr_1) \qquad (10\text{-}9)$$

$$L = L_T + L_B + L_S = (z_o - z_3)/S_o \qquad (10\text{-}10)$$

Solving for z_3 yields

$$z_3 = z_o - (L_T + L_B - z_2/S_s)\, S_o/(S_o/S_s + 1) \qquad (10\text{-}11)$$

This expression is valid only if z_2 is less than or equal to z_3.

If $z_2 + y_2$ is greater than z_3 + TW, the basin must be lowered and the trial and error process repeated until sufficient tailwater exists to force the jump. Perform the following steps to calculate design parameters:

1. Calculate culvert brink depth, y_o, using Figure 10-5 or 10-6, velocity v_o, and Fr = $v_o / \sqrt{g y_o}$.

2. Determine y_n (tailwater, TW) in downstream channel using procedures in Chapter 3.

3. Find y_2 using Equation 10-6.

4. Compare y_2 and TW. If $y_2 <$ TW, the jump will form. If $y_2 >$ TW, lower the basin to provide additional tailwater.

5. Determine the elevation of the basin by trial and error.

   a. Choose trial basin elevation, z_1.

   b. Choose basin width, W_B, and basin slopes, S_T and S_s. A slope of 0.5 (2:1) or 0.33(3:1) is satisfactory for either S_T or S_s.

   c. Check W_B using Equation 10-5.

   d. Calculate y_1 by trial and error using Equation 10-3 and calculate v_1.

   e. Calculate
   $Fr_1 = v_1 / \sqrt{g y_1}$.

   f. Determine y_2 using Equation 10-6 with C_1 corresponding to basin type.

   g. Find z_3 using Equation 10-11.

   h. Calculate $y_2 + z_2$ and z_3 + TW. If $y_2 = z_2$ is greater than z_3 + TW, choose another z_1 and repeat steps 5a through 5h until balance is reached.

6. Calculate L_T, L_s, and L_B using Equations 10-7, 10-8, and 10-9 and compute the horizontal distance downstream to the sill crest, L, using Equation 10-10. L_B can also be found using Figure 10-14.

7. Determine radius to use between culvert and transition from Figure 10-15.

10.6.2 Design Procedure

A form for recording the following USBR Type II design computations is presented in Table 10-5;

1. Determine basin width, W_B, elevation, z_1, length, L_B, total length, L, incoming depth, y_1, incoming Froude number, Fr_1, and jump height, y_2, by using the procedure in Section 10.6.1. For step 5f of Section 10.6.1, use C = 1.1 to find y_2. For Step 6 of Section 10.6.1, use Figure 10-14 to find L_B.

2. The required tailwater depth is as indicated in **Figure 10-16**.

3. The chute block height, h_1, width, W_1, and spacing, W_2, are all equal to the incoming depth.

   $W_1 \;\; W_2 \;\; h_1 = y_1$

   The number of blocks, N_c, is equal to

   $N_c = W_B/2y_1$, rounded to a whole number

   Adjusted $W_1 = W_2 = W_B/2N_c$

   Side wall spacing = $W_1/2$

4. The dentated sill height, h_2, the block width, W_3, and the spacing width, W_4, are determined as follows:

   $h_2 = 0.2y_2$

   $W_3 \;\; W_4 \;\; 0.15y_2$

   where y_2 = jump height

The number of blocks, N_s, plus spaces approximately equals W_B/W_3. Round this to the next lowest odd whole number and adjust $W_3 = W_4$ to fit W_B. The downstream sill slope is 2:1.

10.6.3 Design Considerations

The following factors should be considered during basin design:

1. The Type II basin may be used for Fr from 4 to 14.

2. The chute blocks and end sill do not need to be staggered relative to each other. The width and spacing of the sill blocks may be reduced, but should remain proportional.

3. This design procedure will result in a conservative stilling basin for flows up to 500 cfs per foot of basin width.

4. Chute blocks tend to lift part of the incoming jet from the floor, creating a large number of energy dissipating eddies. The blocks also reduce the tendency of the jump to sweep off the apron. Test data and evaluation of existing structures indicate that a chute block height, width, and spacing equal to the depth of incoming flow, y_1, are satisfactory.

5. As long as the velocity distribution of the incoming jet is fairly uniform, the effect of the chute slope on jump performance is insignificant. For steep chutes or short flat chutes, the velocity distribution can be considered uniform. Difficulty will be experienced with long flat chutes where frictional resistance results in center velocities substantially exceeding those on the sides. This causes an asymmetrical jump with strong side eddies. The same effect will result from sidewall divergent angles too large for the water to follow.

6. The design information for the Type II basin is considered valid for rectangular sections only. If trapezoidal or other sections are proposed, a model study is recommended to determine design parameters.

7. A margin of safety for tailwater is recommended for inclusion in the design. The basin should always be designed with a tailwater 10 percent greater than the conjugate depth. This safety factor is included in the design curves used.

10.6.4 Example Problem

<u>Example 10-7. USBR Type II Outlet Basin Design</u>

Given a 10-foot by 6-foot RCB, Q = 417 cfs, S_o = 6.5%, elevation outlet invert z_o = 100 feet, and v_o = 27.8 feet/second, y_o = 1.5 feet. The downstream channel is a 10-foot bottom trapezoidal channel with 2:1 side slopes and n = 0.03. Find the dimensions for a USBR Type II basin.

1. Determine basin elevation using procedures outlined in Section 10.6.1:

a. Compute the outlet Froude number for $v_o = 27.8$ feet/second and $y_o = 1.5$ feet, $Fr_o = 4$.

b. Estimate the tailwater depth using the normal depth in the channel, TW = $y_n = 1.9$ ft. The resulting normal velocity is = 15.9 feet/second.

c. Determine the depth at Section 2,

$$y_2 = C_1 y_1 \left[\sqrt{1+8Fr^2} - 1\right]/2 =$$

$$1.1(1.5)\left[\sqrt{1+8(4)^2} - 1\right]/2 = 8.6\,feet$$

d. Since $y_2 >$ TW (8.6 > 1.9), drop the basin.

e. (1) Use $z_1 = 84.5$ feet $= z_2$

(2) $W_B = 10$ feet, $S_T = S_s = 0.5$

(3) W_B OK, no flare

(4) From Equation 10-4,

$Q = 10y_1\,[2g(100 - 84.5 + 1.5 - y_1) + 27.8^2]^{1/2}$

$Q = 10y_1\,[64.4(17 - y_1) + 772.8]^{1/2}$

Solving for the depth at section 1, $y_1 = 0.98$ OK

The resulting velocity at section 1 is $v_1 = 417/.98\,(10) = 42.6$ feet/second

(5) Compute the Froude number for section 1,

$$Fr_1 = 42.6/\sqrt{g(0.98)} = 7.58$$

(6) For $C_1 = 1.1$, $y_2 = 1.1(.98)$

$$\left[\sqrt{1+8(6.93)^2} - 1\right]/2 = 11\,feet$$

(7) Determine the basin dimensions. From Figure 10-14,

$L_B/y_2 = 4.3$

$L_B = 47.5$ feet

$L_T = (z_o - z_1)/S_T = (100 - 84.5)/.5 = 31$ feet

$z_3 = [100 - (47.5 + 31 - 84.5/0.5)\ 0.065]/1.13$

$z_3 = 93.7$ feet

(8) $y_2 + z_2 = 95.5$ feet

$z_3 + TW = 95.6$ feet OK

f. $L_T = 31$ feet, $L_B = 47.5$ feet

$L_S = (z_3 - z_2)/S_s = (93.7 - 84.5)/0.5 = 18.4$ feet

$L = 31 + 47.5 + 18.4 = 97$ feet

g. $Fr_o = 4$ from Figure 10-16, $y_o/r = 0.1$

$r = 1.5/0.1 = 15$ feet

Basin width, $W_B = 10$ feet

Basin elevation, $z_1 = 84.5$ feet

Basin length, $L_B = 47.5$ feet

Total length, $L = 97$ feet

Incoming depth, $y_1 \cong 1$ foot

Incoming Froude number, $Fr_1 = 7.6$

Jump height, $y_2 \cong 11$ feet

h. Determine the chute block dimensions

$h_1 = W_1 = W_2 = y_1 = 1.0$ foot

$N_c = 10/2(1) = 5$ OK, whole number

$W_1 = W_2 = 10/2(5) = 1$

Sidewall spacing = $W_1/2 = 0.5$ foot

i. Determine the dentated sill dimensions:

$h_2 = 0.2y_2 = 0.2(11) = 2.2$ feet

$W_3 = W_4 = 0.15y_2 = 1.65$ feet

$N_s = W_B/W_3 = 10/1.65 \cong 6$

Use 5, which makes 3 blocks and 2 spaces each 2 feet.

Table 10-1
SUGGESTED OUTLET PROTECTION TYPE
BASED ON FROUDE NUMBER AND VELOCITY

Type of Outlet Protection	$Fr \le 2.5$	Fr Between 2.5 and 4.5	$Fr \ge 4.5$ and $V < 50$ [a]	$Fr \ge 4.5$ and $V \ge 50$ [a]
Riprap Apron	X			
Riprap Outlet Basin	X			
Baffled Outlet	X [b]	X [b]	X [b]	
USBR Type II Basin				X

[a] Velocity is based on the energy to be dissipated. Theoretically, the dissipation velocity can be calculated using the equation:

$$v = \sqrt{2gh}$$

Where: v = Theoretical dissipation velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second ^{2}

h = Energy head to be dissipated, in feet (can be approximated as the difference between channel invert elevations at the inlet and outlet)

[b] Practical application requires that $1 \le Fr \le 9$.

Table 10-2
EXPERIMENTAL COEFFICIENTS FOR CULVERT OUTLET SCOUR

Material	Nominal Grain Size d_{50} (mm)	Scour Equation	Depth h_s				Width W_s				Length L_s				Volume V_s			
			α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e	α	β	θ	α_e
Uniform Sand	0.20	V-1 or V-2	2.72	.375	0.10	2.79	11.73	0.92	.15	6.44	16.82	0.71	0.125	11.75	203.36	2.0	0.375	80.71
Uniform Sand	2.0	V-1 or V-2	1.86	0.45	0.09	1.76	8.44	0.57	0.06	6.94	18.28	0.51	0.17	16.10	101.48	1.41	0.34	79.62
Graded Sand	2.0	V-1 or V-2	1.22	0.85	0.07	.75	7.25	0.76	0.06	4.78	12.77	0.41	0.04	12.62	36.17	2.09	0.19	12.94
Uniform Gravel	8.0	V-1 or V-2	1.78	0.45	0.04	1.68	9.13	0.62	0.08	7.08	14.36	0.95	0.12	7.61	65.91	1.86	0.19	12.15
Graded Gravel	8.0	V-1 or V-2	1.49	0.50	0.03	1.33	8.76	0.89	0.10	4.97	13.09	0.62	0.07	10.15	42.31	2.28	0.17	32.82
Cohesive Sandy Clay																		
60% Sand PI 15	0.15	V-1 or V-2	1.86	0.57	0.10	1.53	8.63	0.35	0.07	9.14	15.30	0.43	0.09	14.78	79.73	1.42	0.23	61.84
Clay PI 5-16	Various	V-3 or V-4	0.86	0.18	0.10	1.37	3.55	0.17	0.07	5.63	2.82	0.33	0.09	4.48	0.62	0.93	0.23	2.48

EQUATIONS

V-1. FOR CIRCULAR CULVERTS. Cohesionless material or the 0.15mm cohesive sandy clay.

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, or\ \frac{V_s}{D^3}\right] = \propto \left(\frac{Q}{\sqrt{g}\,D^{5/2}}\right)^b \left(\frac{t}{t_o}\right)^q$$

where t_o = 316 min.

V-2. FOR OTHER CULVERT SHAPES. Same material as above.

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, or\ \frac{V_s}{y^3{}_e}\right] = \propto_e \left(\frac{Q}{\sqrt{g}\,y_e{}^{5/2}}\right)^b \left(\frac{t}{t_o}\right)^q$$

where t_o = 316 min.
Reference: USDOT, FHWA, HEC-14 (1983).

V-3. FOR CIRCULAR CULVERTS. Cohesive sandy clay with PI = 5-16.

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, or\ \frac{V_s}{D^3}\right] = \propto \left(\frac{rV^2}{t_c}\right)^b \left(\frac{t}{t_o}\right)^q$$

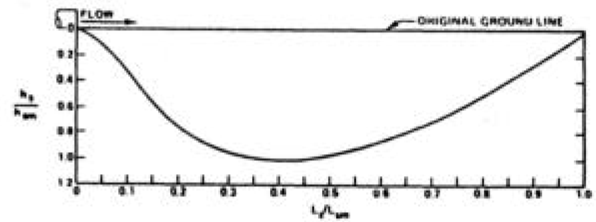
where t_o = 316 min.

V-4. FOR OTHER CULVERT SHAPES. Cohesive sandy clay with PI = 5-16.

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, or\ \frac{V_s}{y_e{}^3}\right] = \propto_e \left(\frac{rV^2}{t_c}\right)^b \left(\frac{t}{t_o}\right)^q$$

where t_o = 316 min.

Project ______________
Station ______________
Designer ______________
Date ______________



DIMENSIONLESS CENTER-LINE PROFILE

(1) $y_e = (A/2)^{1/2}$

CULVERT	TYPE / CONTROL	SIZE	"n"	LENGTH	SLOPE S_o	DISCHARGE	DEPTH y_o	VELOCITY V_o	FLOW AREA	EQUI(1) DEPTH y_e	Fr	END TREATMENT Entrance	END TREATMENT Exit

CHANNEL	TYPE	SIDE SLOPE	"n"	BOTTOM WIDTH	SLOPE	DISCHARGE Q	VELOCITY	FLOW AREA	TW	FREE BOARD	FR	TYPE OF MATERIAL

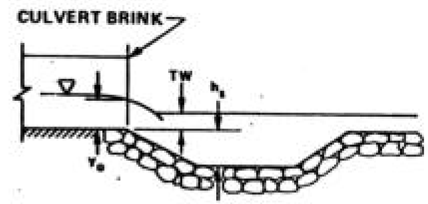
SCOUR COMPUTATIONS	EQUIVALENT DEPTH y_e	SCOUR DEPTH h_s	SCOUR WIDTH W_s	SCOUR LENGTH L_s	SCOUR VOLUME V_s

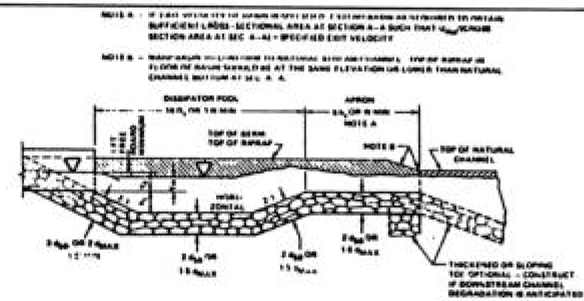
ALLOWABLE CONDITIONS	VELOCITY	SCOUR DEPTH	SCOUR WIDTH	SCOUR LENGTH	OTHER RESTRICTIONS

OTHER SITE CONSTRAINTS	

Reference: *USDOT, FHWA, HEC-14 (1983).*

Table 10-3
Culvert, Channel, Scour, and Other Site Data Computation Form





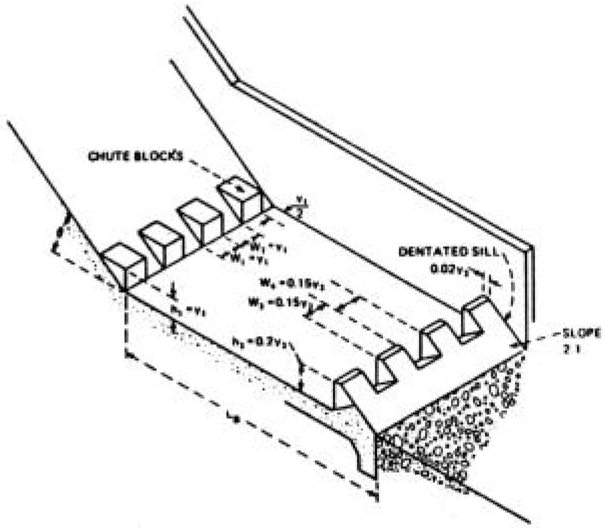
	TW	y_e	(1) TW/y_e	d_{50}/y_e	d_{50}	h_s/y_e	h_s	(2) h_s/d_{50}	
LOW TW $TW/y_e \le 0.75$									
	V ALLOWABLE	L/D_e (3)	L	V_{ave}/V_L	V_L				
HIGH TW $TW/y_e > 0.75$									

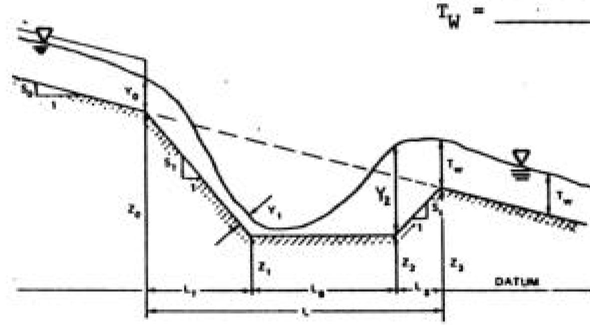
	Larger of		
Length of Pool	$= 10\ h_s$ or $3W_o$	= ______	ft.
Length of Apron	$= 5\ h_s$ or W_o	= ______	ft.
Thickness of Approach	$= 3d_{50}$ or $2d_{max}$	= ______	ft.
Thickness of Remainder of Basin	$= 2d_{50}$ or $1.5d_{max}$	= ______	ft.

(1) $TW/y_e \le 0.75$ for Low TW Design
(2) $2 < hs/d_{50} < 4$
(3) $D_e = [4A/\pi]^{1/2}$

Reference: USDOT, FHWA, HEC-14 (1983).

Table 10-4
Riprap Outlet Basin Computation Form





C_1 = ______

S_s = ______

S_T = ______

W_B = ______

y_2 = ______

T_W = ______

If $y_2 > T_W$, Depress Basin

$Z_1 = Z_2$	y_1	V_1	Fr_1	y_2	L_B	L_T	Z_3	$y_2 + Z_2$ (1)	$Z_3 + T_W$ (1)
L_S	L	W_1	h_1	N_c	h_2	W_3	W_4	N_s	

(1) $Z_3 + T_W \geq y_2 + Z_2$

Reference: *USDOT, FHWA, HEC-14 (1983).*

Table 10-5
USBR Type II Basin Computation Form

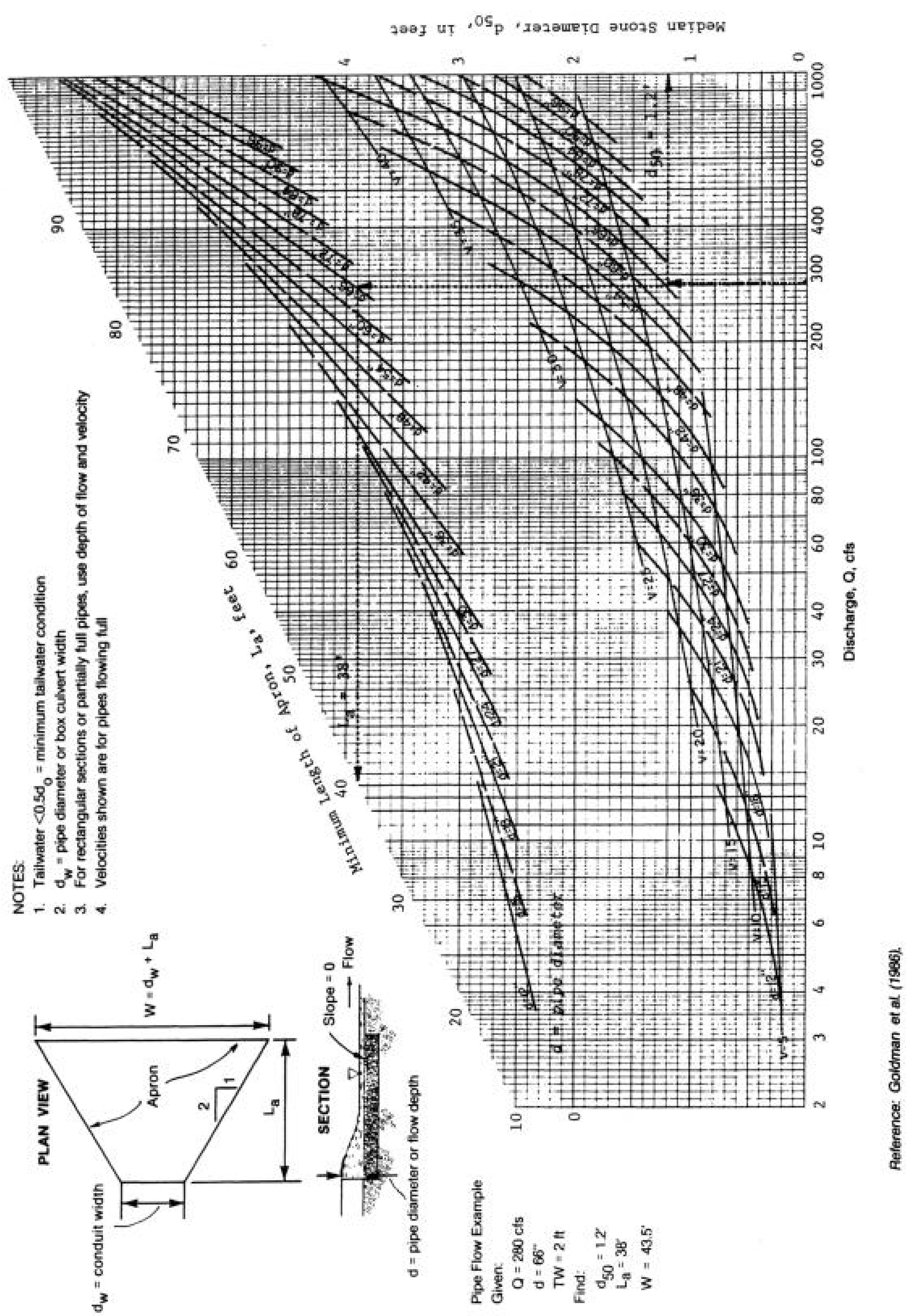


Figure 10-1
Design of Riprap Apron under Minimum Tailwater Conditions

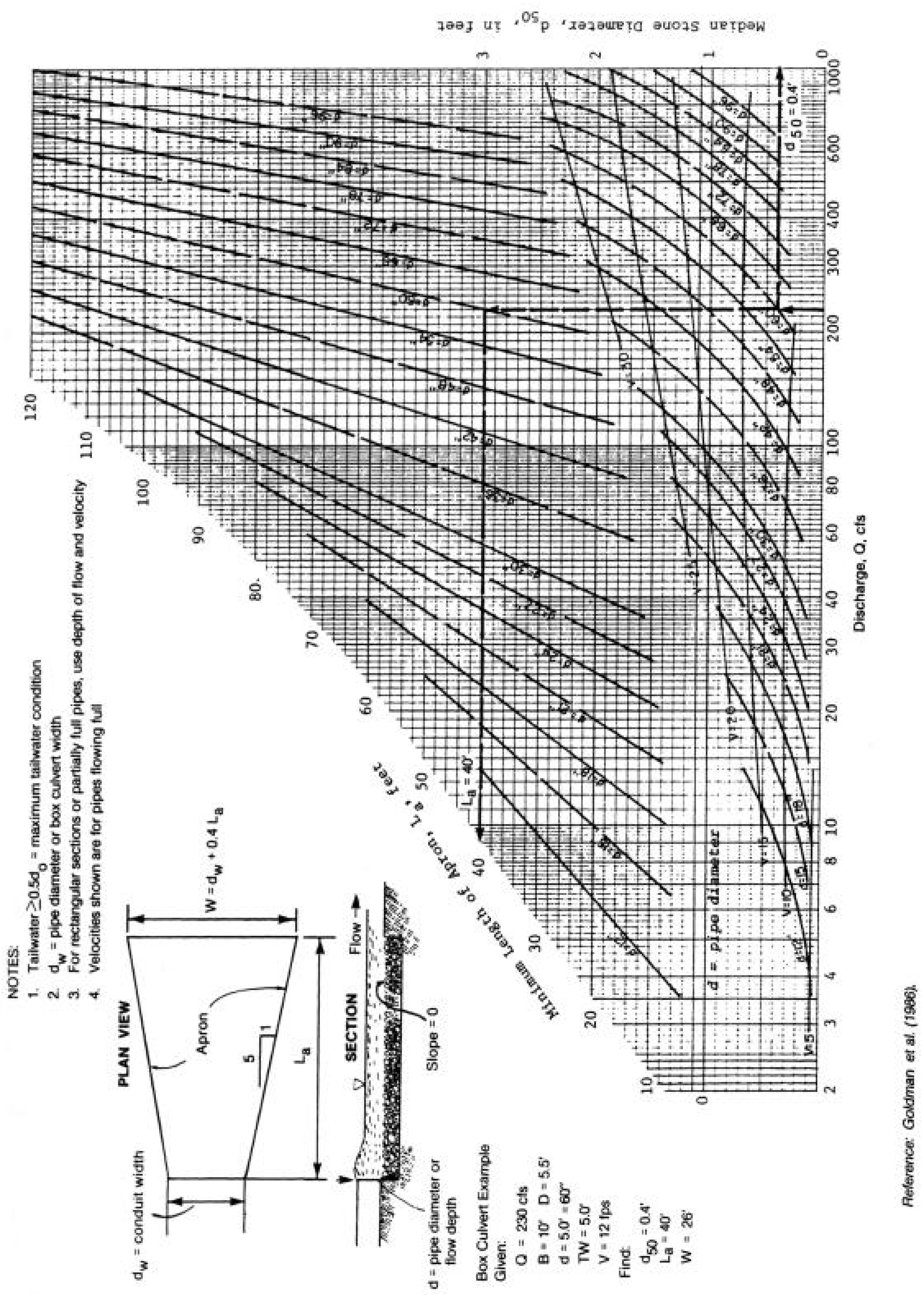


Figure 10-2
Design of Riprap Apron under Maximum Tailwater Conditions

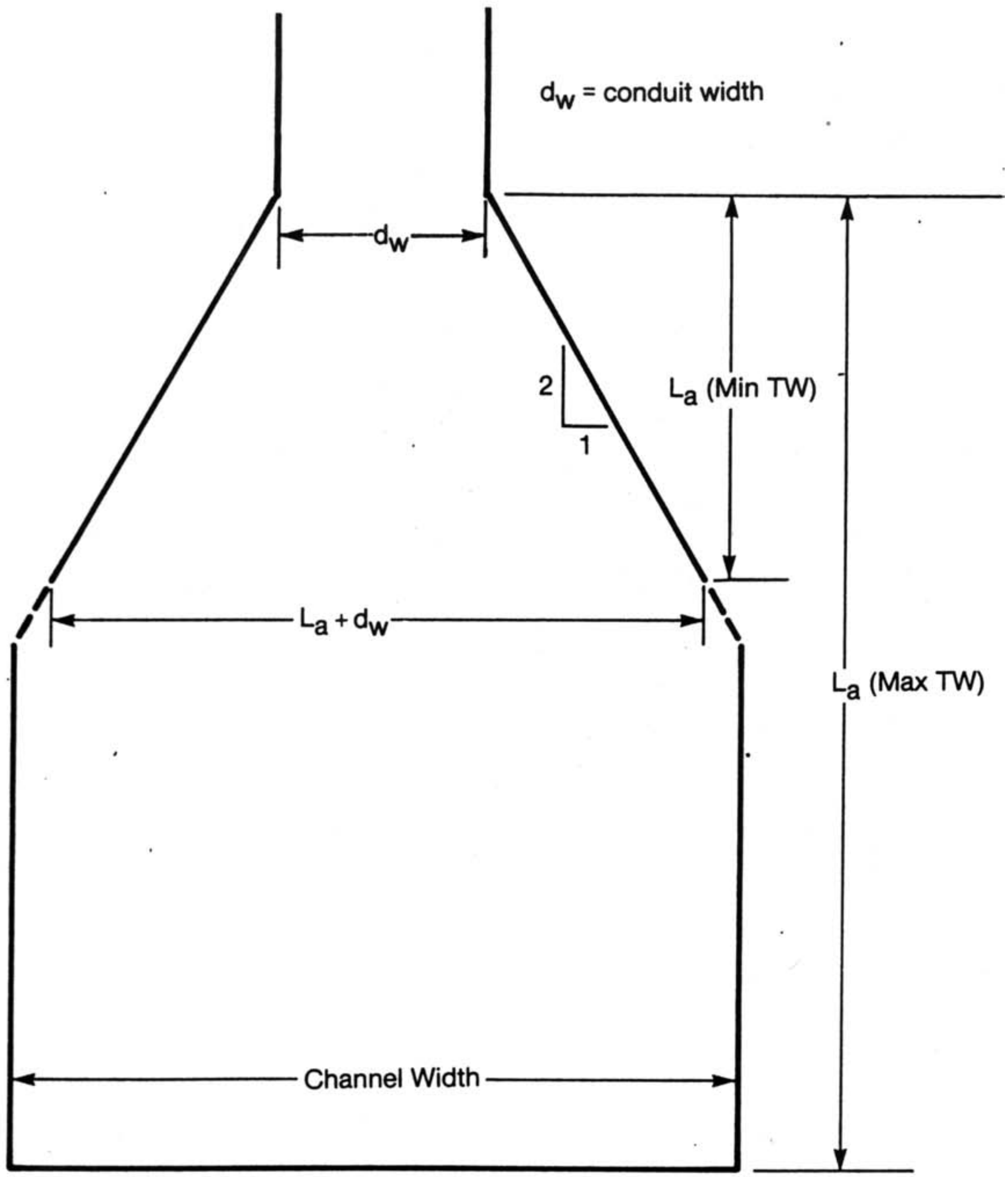
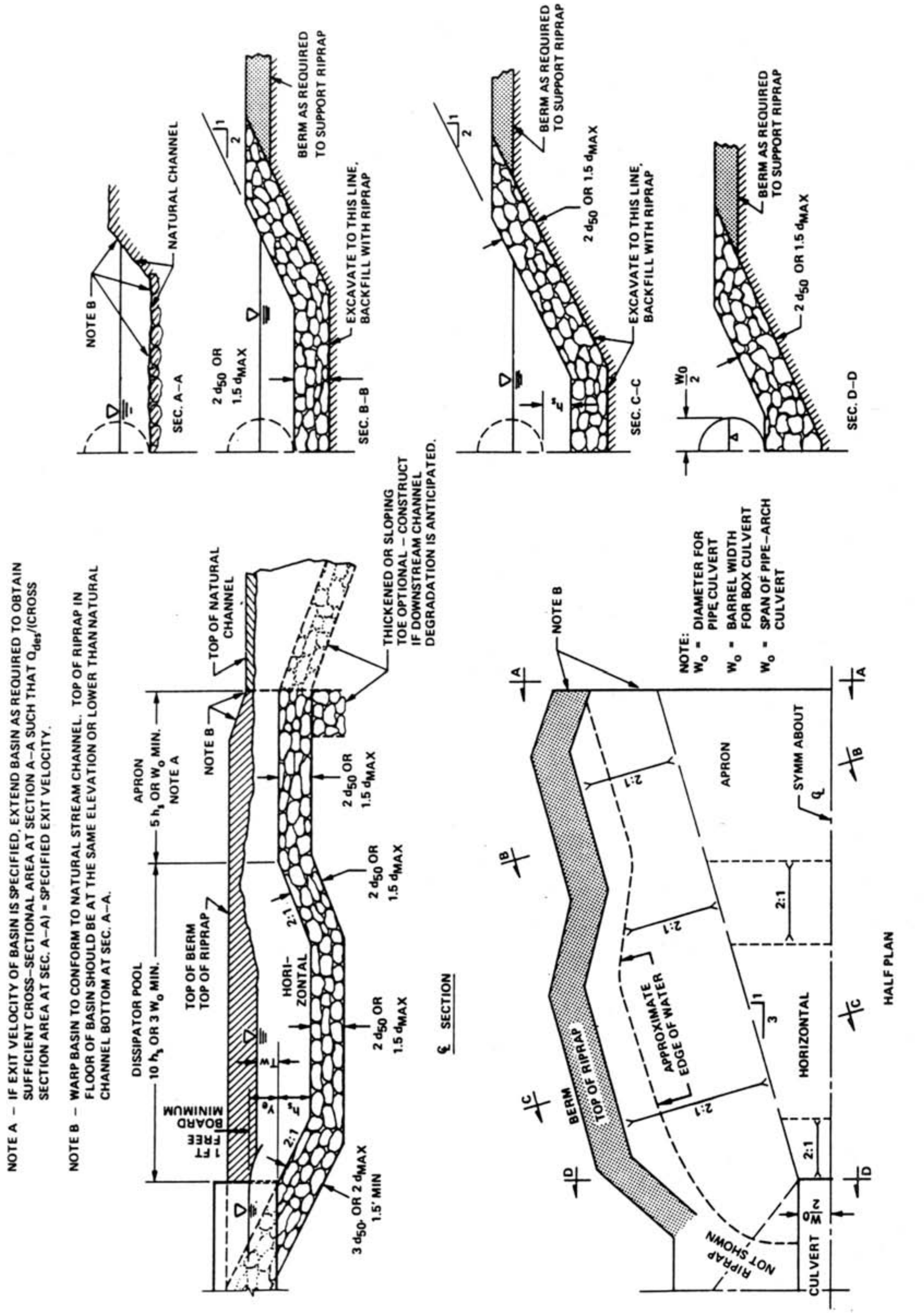
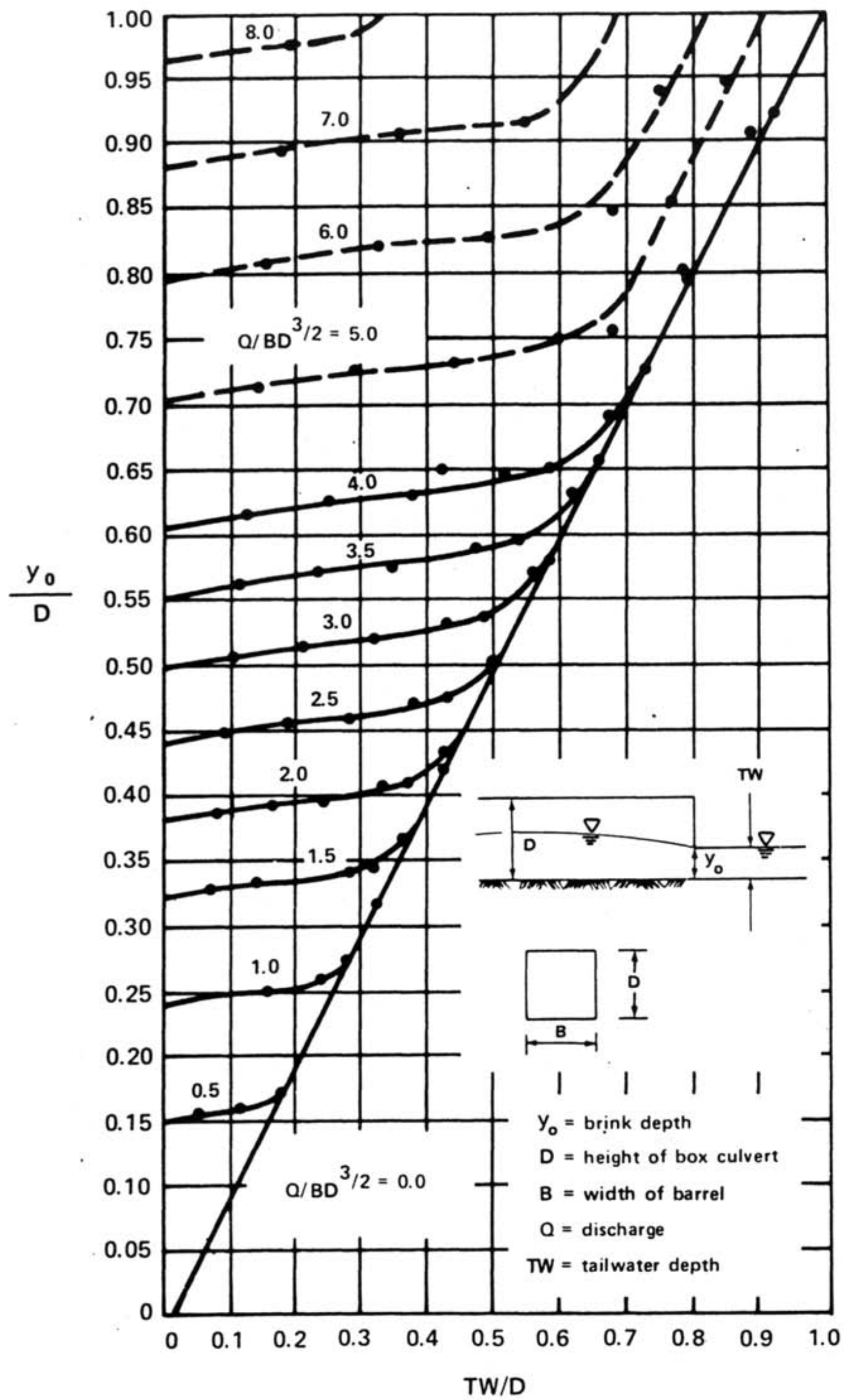


Figure 10-3
Riprap Apron Schematic for Uncertain Tailwater Conditions



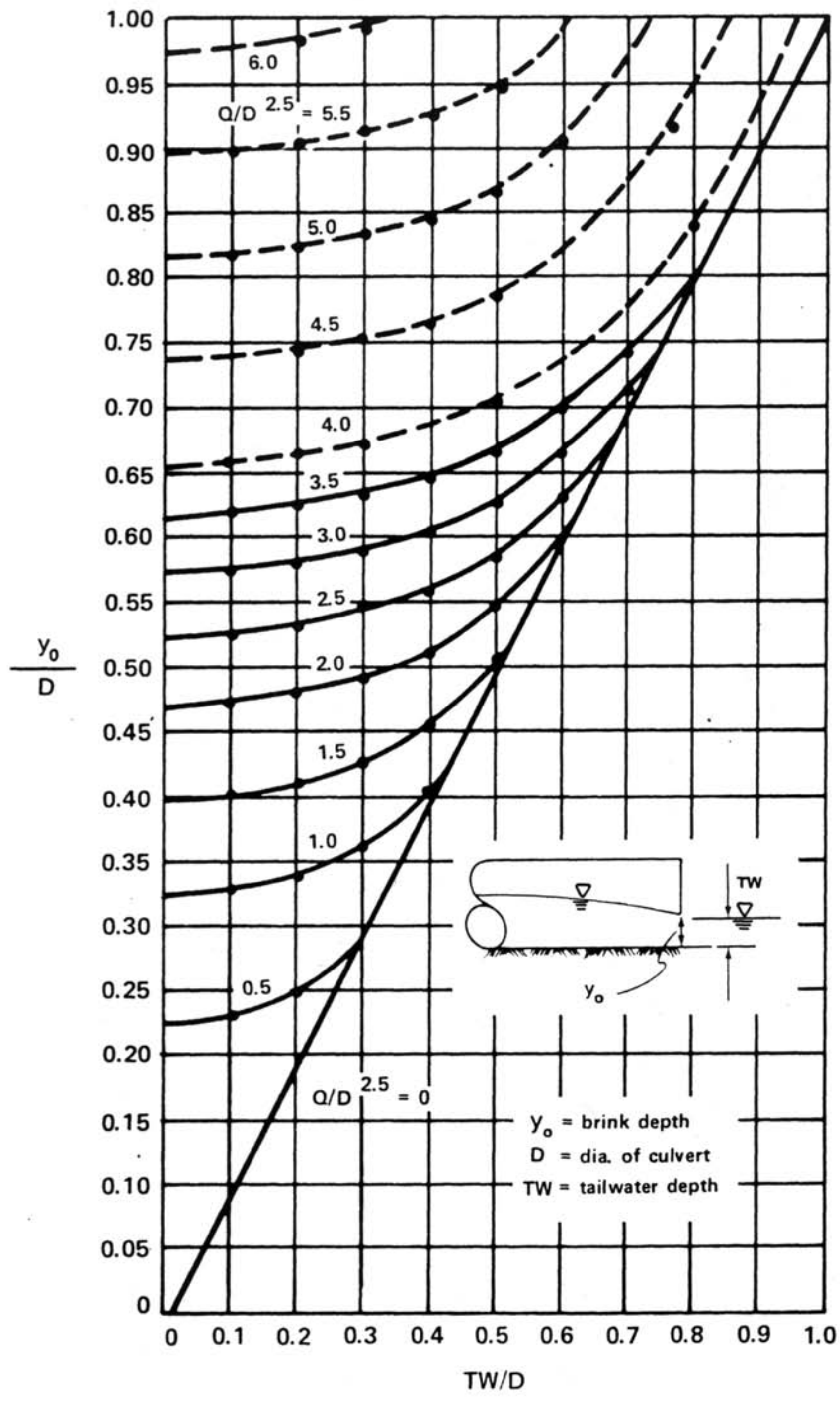
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-4
Details of Riprap Outlet Basin



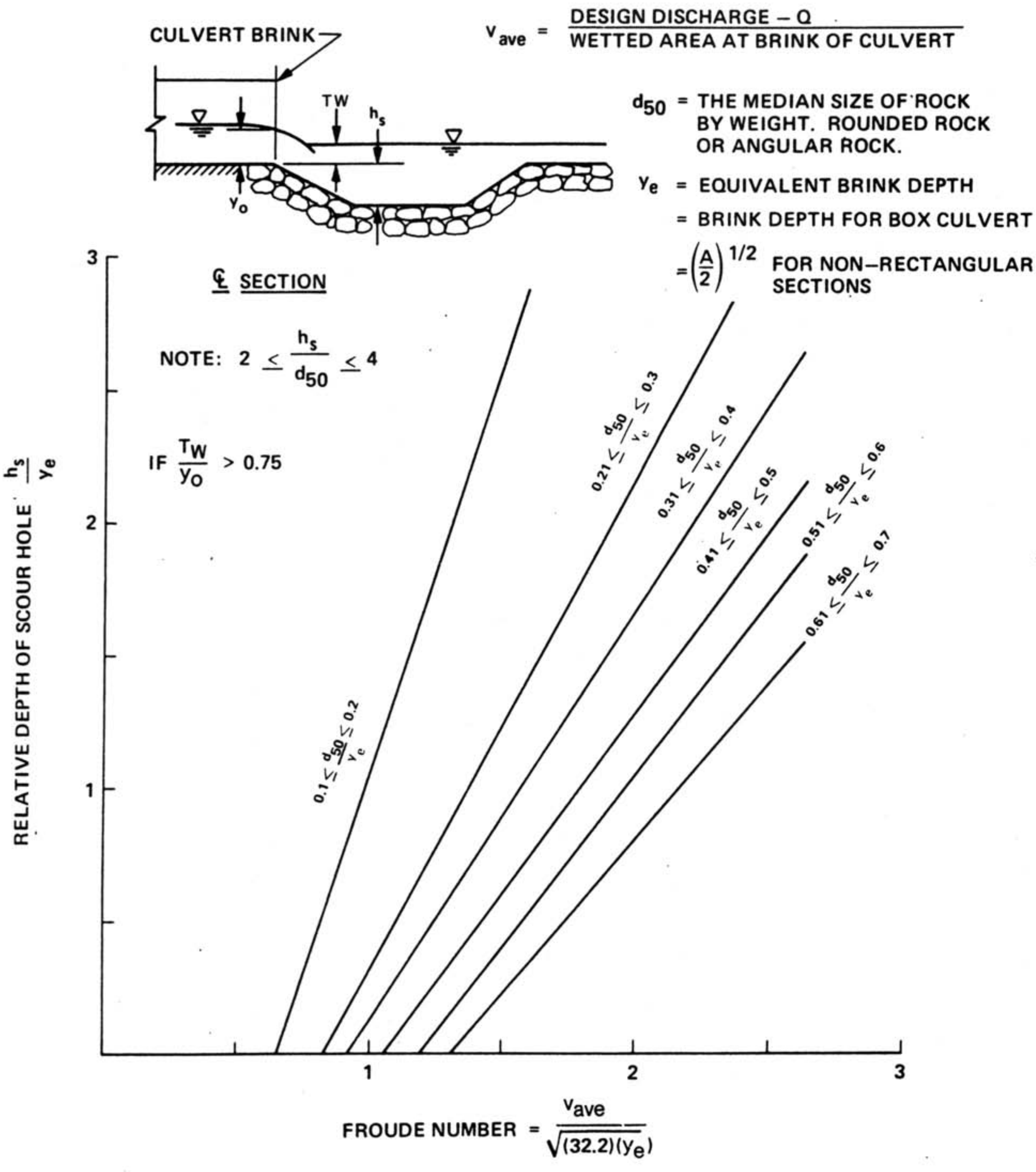
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-5
Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes



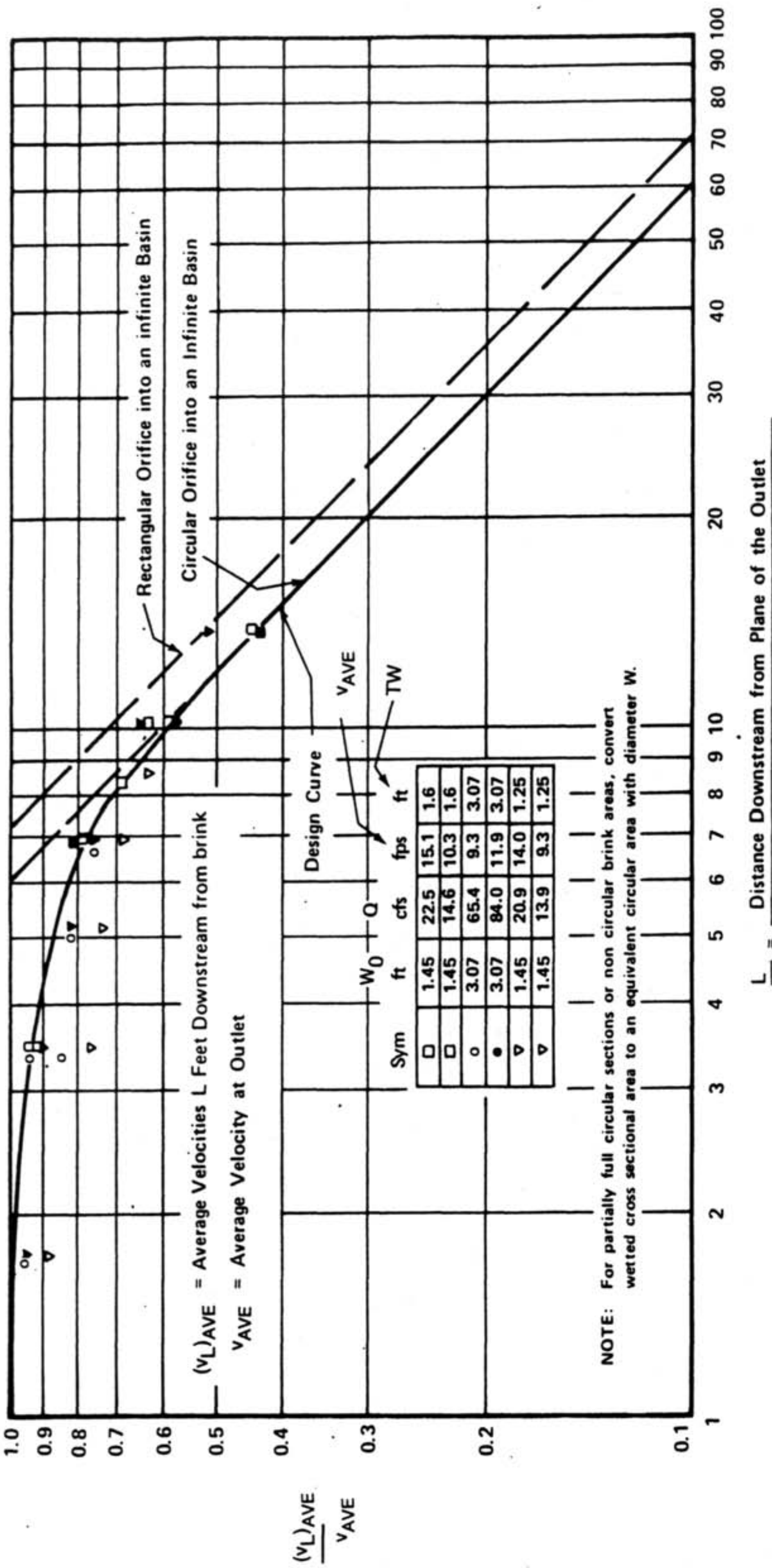
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-6
Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes



Reference: USDOT, FHWA, HEC-14 (1983).

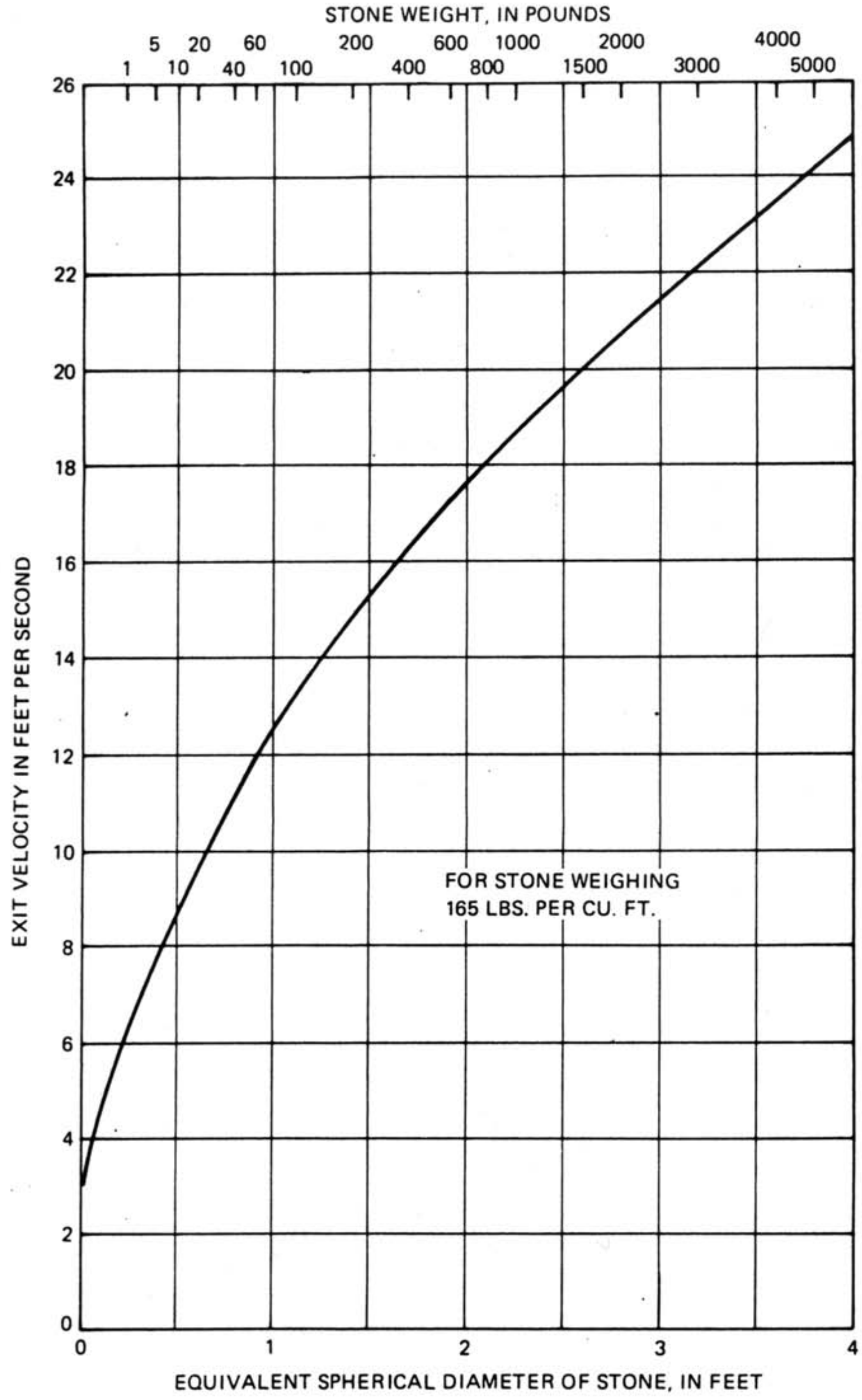
Figure 10-7
Relative Depth of Scourhole vs. Froude Number at
Brink of Culvert with Relative Size of Riprap as a Third Variable



NOTE: Chart is used to predict channel velocities downstream from culvert outlet where high tailwater prevails. Velocities obtained from this chart can be used with Figure 10-9 to size riprap.

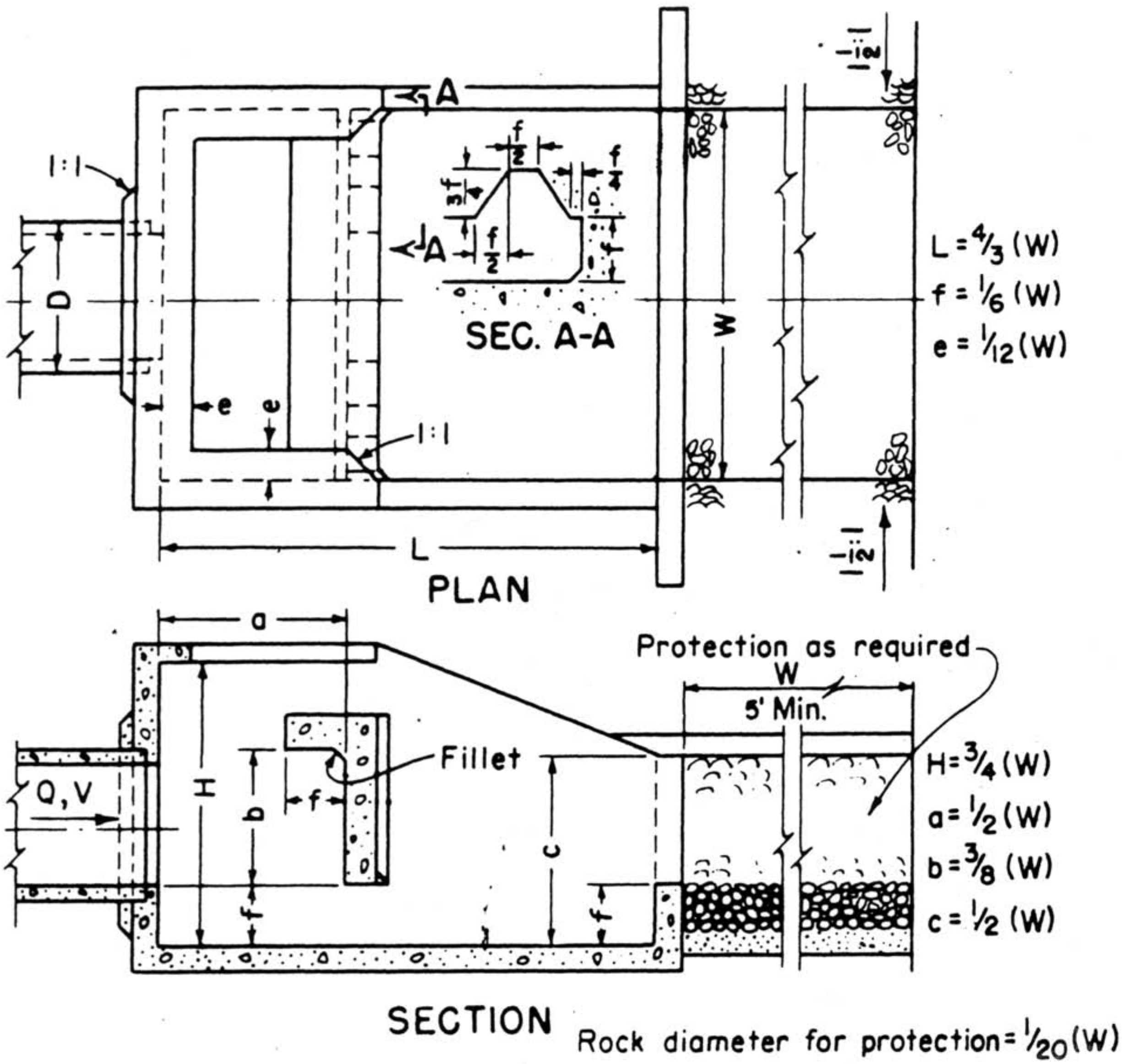
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-8
Distribution of Centerline Velocity for Flow from Submerged Outlets



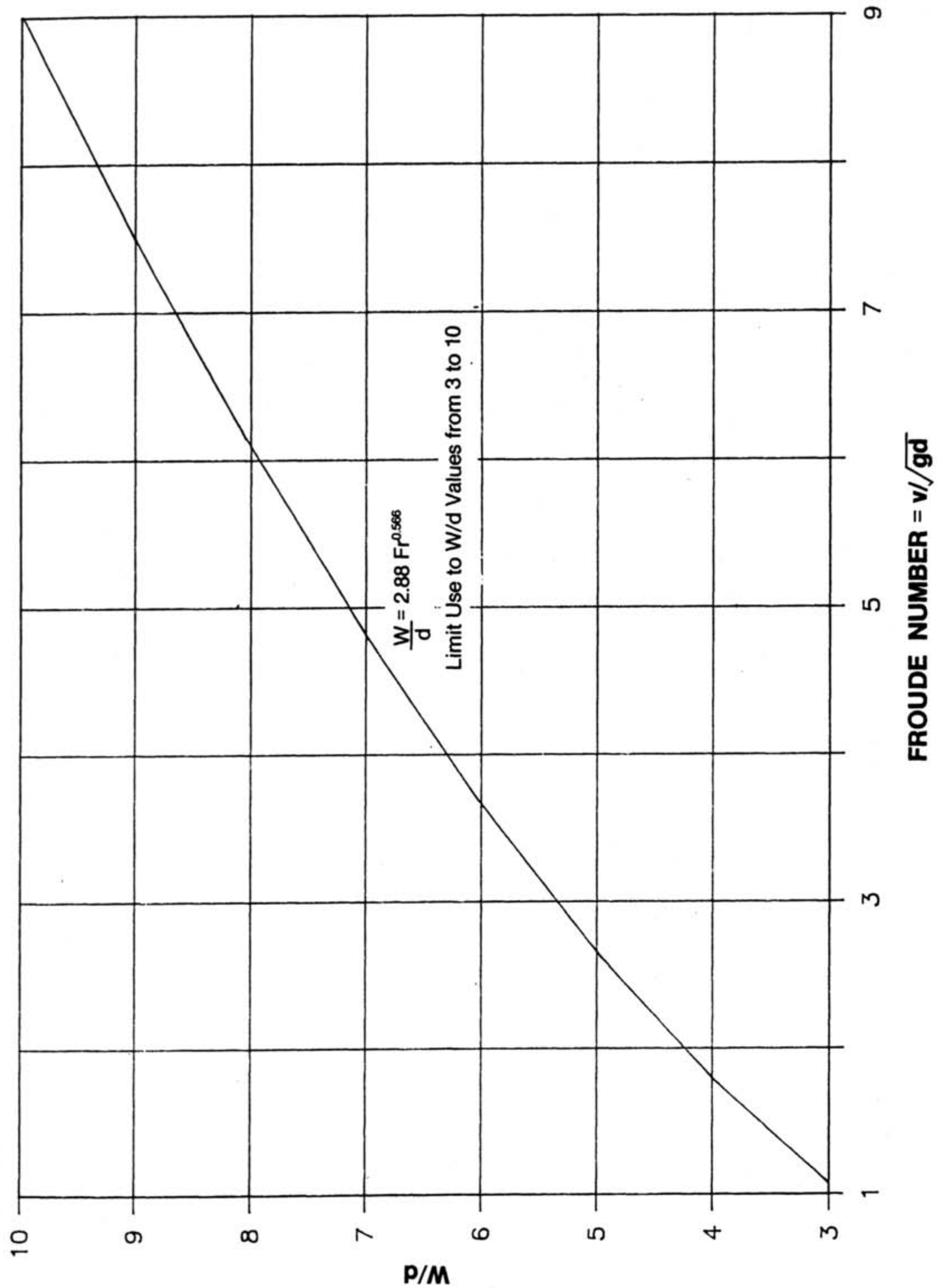
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-9
Riprap Size for Protection Downstream of Outlet Basins



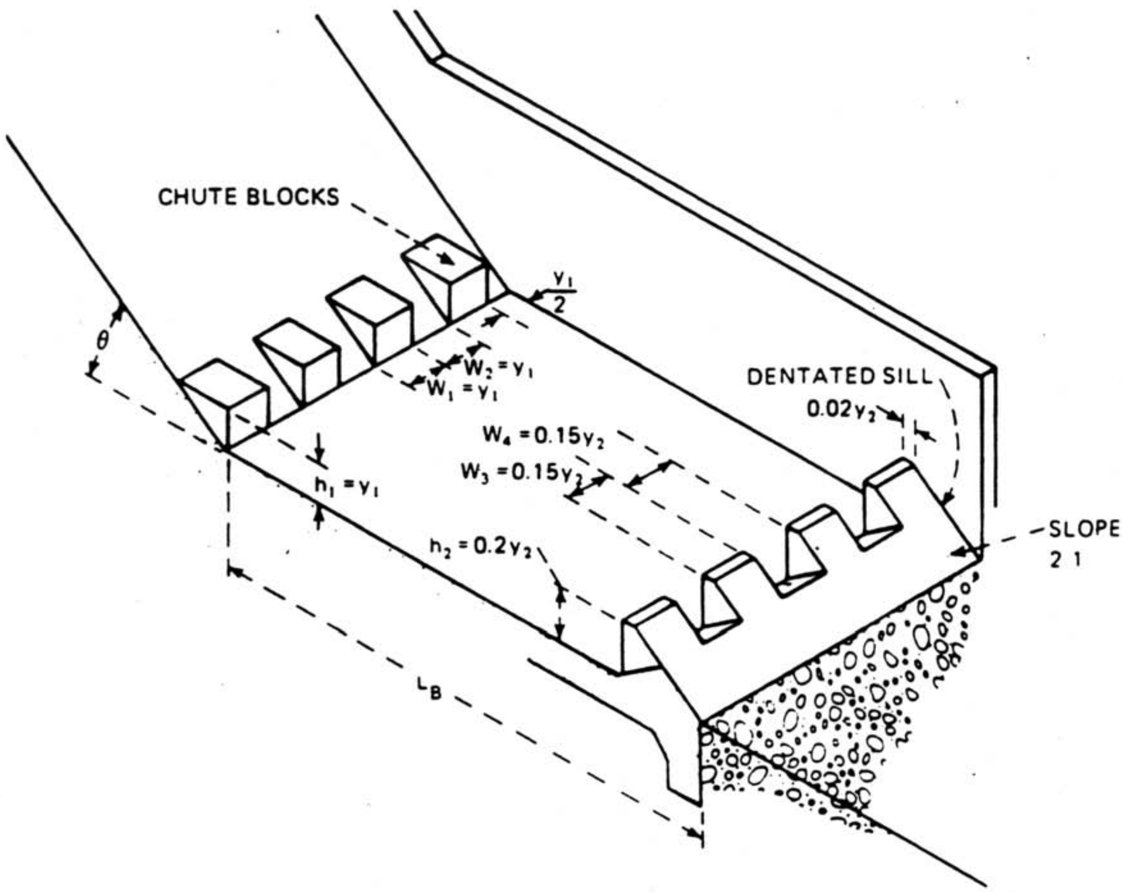
Reference: U.S. Department of the Interior (1978).

Figure 10-10
Schematic of Baffled Outlet



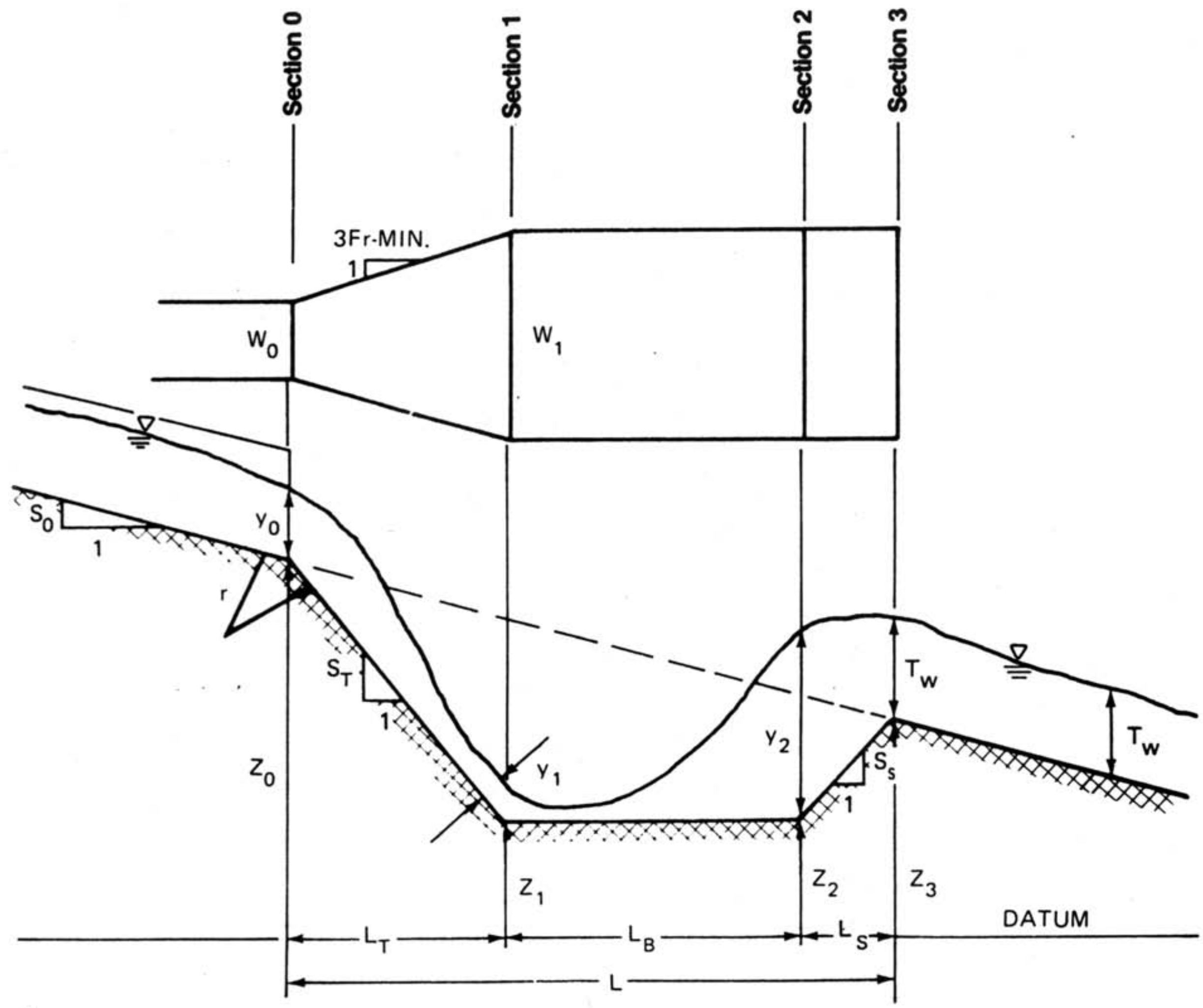
Reference: U.S. Department of the Interior (1978).

Figure 10-11
W/d vs. Froude Number for Baffled Outlet Basins



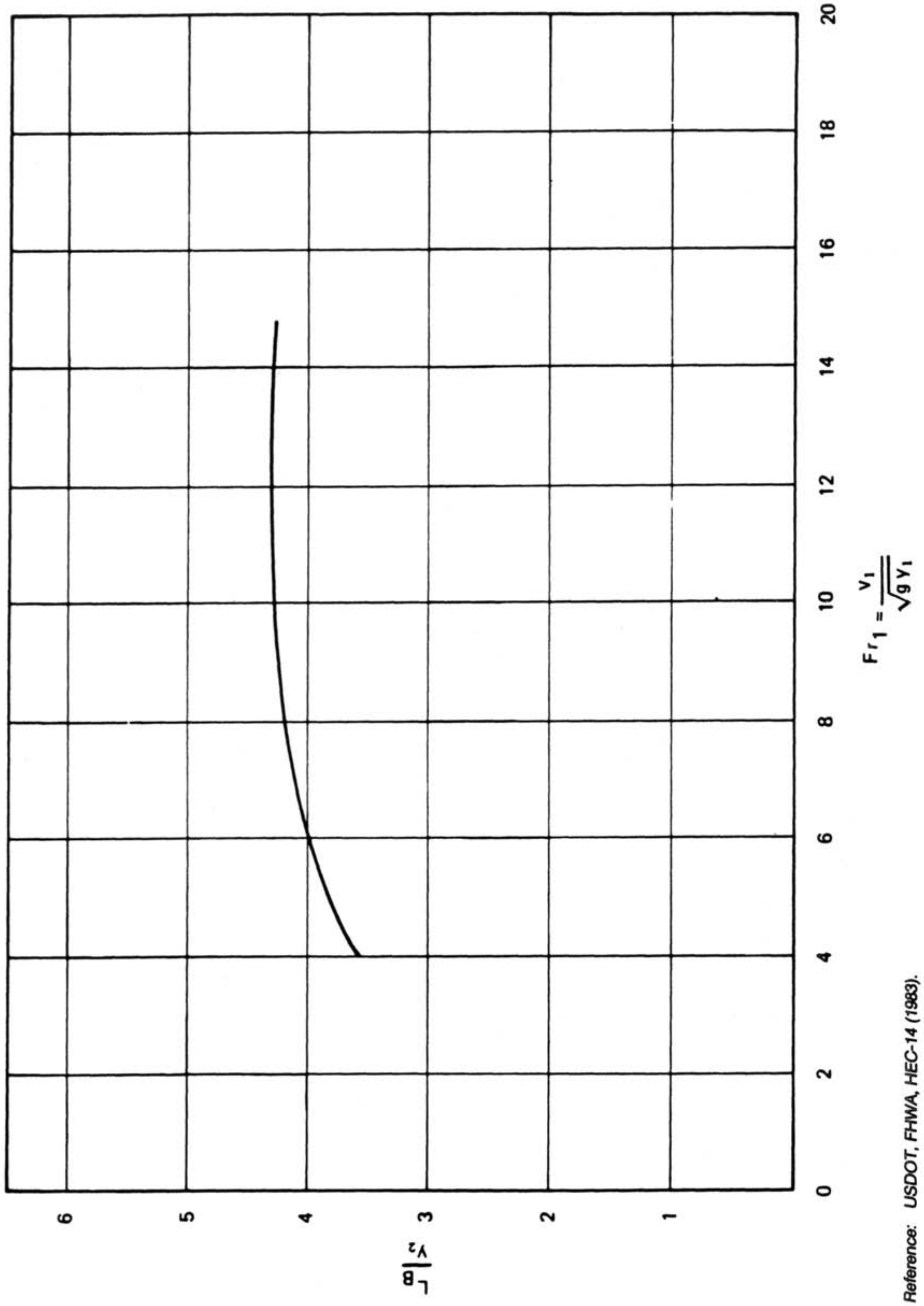
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-12
USBR Type II Outlet Basin



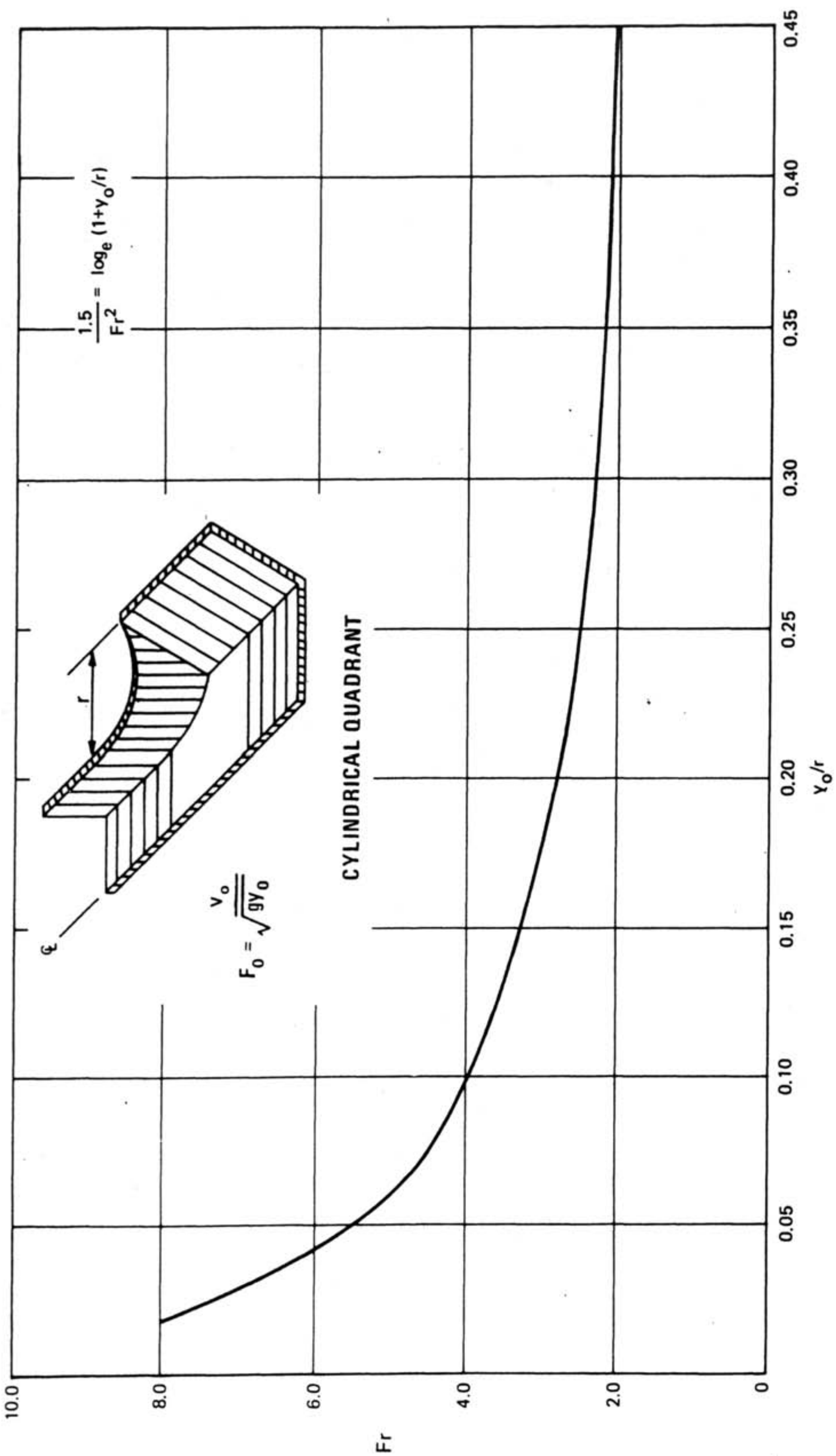
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-13
Definition Sketch of Supercritical Flow Expansions



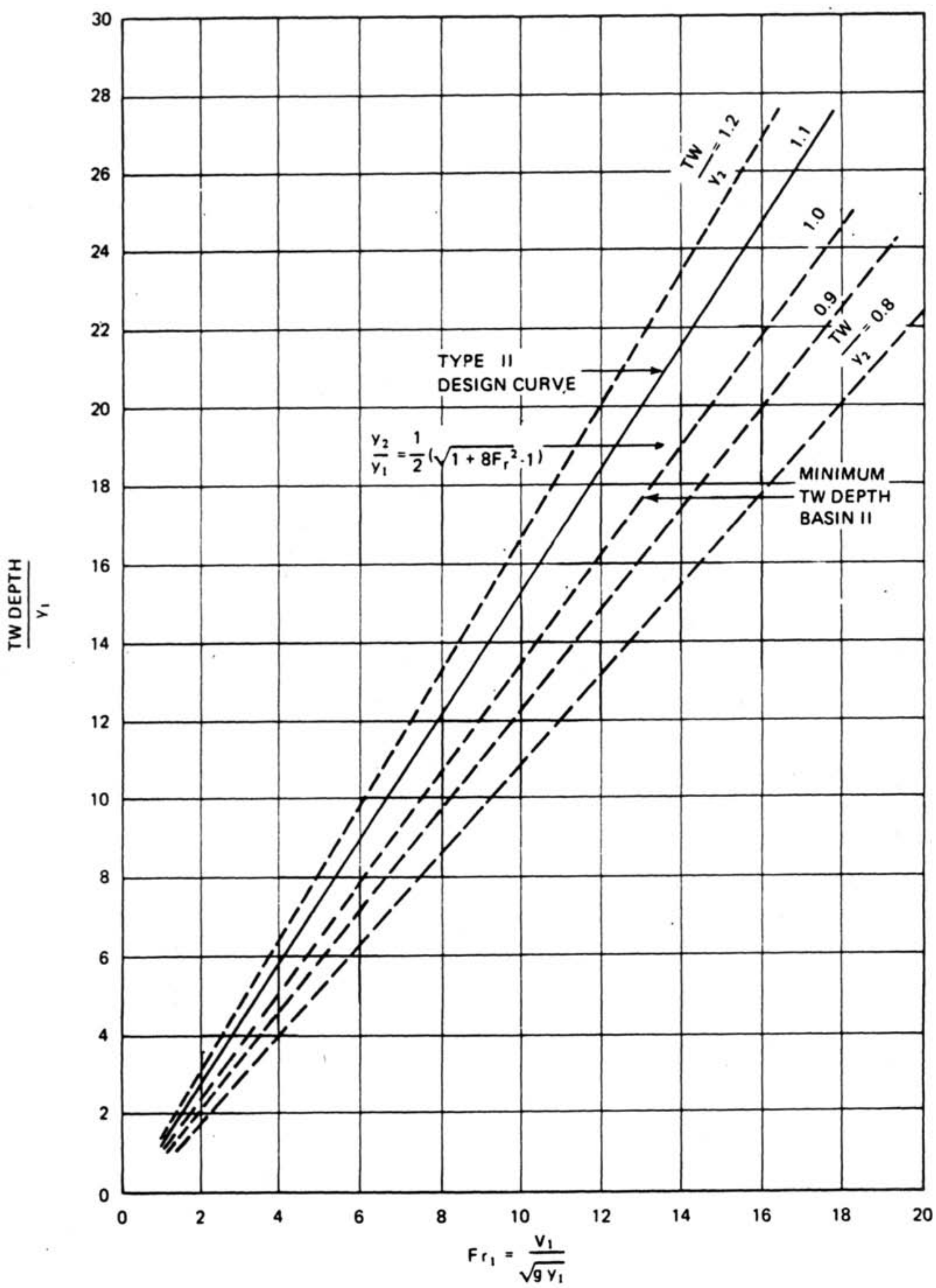
Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-14
Length of Jump on Horizontal Floor
for USBR Type II Outlet Basin



Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-15
Fr vs. y_o/r for Flow Transitions



Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-16
Tailwater Depth for USBR Type II Outlet Basin

CHAPTER 11
DATA COLLECTION

Chapter 11
DATA COLLECTION

Synopsis

Identification of data needs should be a part of the early planning phase of a project, when preparing to develop a site and/or appropriate procedures for performing hydrologic and hydraulic calculations are selected. Several categories of data may be relevant to a particular project, including published data on precipitation, soils, land use, topography, streamflow, and flood history (including floodway data defining buffers zones of non-disturbance). Field investigations are generally necessary to determine drainage areas, identify pertinent features, obtain high water information, and survey channel sections and bridge and culvert crossings.

11.1 General Information

Useful data usually fall into one of the following four categories:

1. Previous hydrologic studies
2. Natural resources data
3. Manmade features
4. Field investigations

Not all the information presented in this chapter may be required to address the needs of each project.

There are many potential sources of the data typically required for development projects. Identifying these sources can be difficult, and making the subsequent necessary contacts can be time-consuming. To assist with identification, typical data sources are presented in this chapter. In general, the watershed master plans developed by MWS, provide the best source of data for the watersheds studied and provide a consistent starting point for watersheds not studied.

The principal use of stormwater data is to establish the hydrologic and hydraulic characteristics of a watershed in order to evaluate stormwater runoff quantity and quality conditions. Both existing and future watershed conditions should be considered. Stormwater data should be collected before calculations are initiated, using the following general guidelines;

1. Identify data needs, sources, and uses.

2. Collect published data, based on sources identified in Step 1 and information presented in Sections 11.2, 11.3, 11.4, and 11.5.

3. Compile and document the results of Step 2, and compare data needs and uses with published data availability. Identify any additional field data needs.

4. Collect field data based on needs identified in Steps 1 and 3, using information presented in Section 11.6.

5. Compile and document the results of Step 4.

11.2 Previous Studies

11.2.1 MWS

MWS can provide master plans for selected watersheds in Nashville and Davidson County. These plans, available from MWS, should be consulted at the beginning of a drainage study or beginning of site planning. MWS has also performed hydrologic studies for specific project areas and should be contacted for availability. These studies are the basis for a stream buffer defining no-disturbance areas as required in Volume 1 Sections 2.2.14 and 5.9.

11.2.2 Metro Planning

Metro Planning maintains a series of 14 Subarea Plans that among other things define the intended long-term use and character for various areas of the Metro area. The plans also present existing land use, steep slope areas, restrictive soils areas, general information about the transportation system, utilities, local area policies, and other planning information. While these studies may not provide much specific information for stormwater management design in a site development it does provide insight in overall site development planning. These plans are available in hard copy from the Planning Commission.

11.2.3 U.S. Geological Survey

The USGS has published several papers, studies via hard copy or electronically via the internet on the hydrologic aspects of Tennessee, among the most pertinent of which are the following:

1. Yearly Water Resources Data for Tennessee.

2. Historical daily stream flows at http://water.usgs.gov/osw/

3. Randolph, W.J. and C.R. Gamble. Technique for Estimating Magnitude and Frequency of Floods in Tennessee. 1976.

4. Wibben, H.C. Effects of Urbanization on Flood Characteristics in Nashville-Davidson County, Tennessee. 1976.

5. Gamble, C.R. Technique for Estimating Depth of Floods in Tennessee. 1983.

6. Robbins, C.H. Basic Data Report on Effects of Urbanization on the Magnitude and Frequency of Floods on Small Streams in Tennessee. 1984a.

7. Robbins, C.H. Synthesized Flood Frequency for Small Urban Streams in Tennessee. 1984b.

8. Robbins, C.H. Techniques for Simulating Flood Hydrographs and Estimating Flood Volumes for Ungaged Basins in Central Tennessee. 1986.

9. Water Resources Investigations in Tennessee: Programs and Activities of the U.S. Geological Survey. 1992-94:OFR-94-498. 1995-96:OFR-97-113. and subsequent reports.

More information on USGS activities can be obtained by contacting the Nashville District Office at the following address:

U.S. Geological Survey
Water Resources Division
A-413
Federal Building, U.S. Courthouse
Nashville, Tennessee 37203
(615) 736-5424

A wide variety of other data on geology, mapping, water resources can be purchased and/or downloaded from the USGS on the world wide web at http://www.usgs.gov/pubprod/.

11.2.4 U.S. Army Corps of Engineers

The Corps of Engineers may be a source for the following information:

1. Event of record high water marks

2. Local flood control studies

3. A report from 1979 entitled Managing Our Urban Water Resources that provides extensive background information for the Metro area on the following topics:

   a. Environmental resources
   b. Population
   c. Economic resources
   d. Flood damage abatement

e. Water supply
f. Wastewater management
g. Navigation
h. Stream bank stabilization
i. Water-related recreation
j. Fish and wildlife

11.2.5 U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS – Formerly SCS)

The USDA NRCS develops and maintains a variety of natural resources data, studies, reports, and technical tools. They are particularly know for good soils data, soil management techniques, and urban hydrology. Information about the NRCS technical resources can be reviewed at http://www.nrcs.usda.gov/techres.html.

11.2.6 Other Studies

The Greater Nashville Area 208 Waste Treatment Plan completed by the Mid-Cumberland Council of Governments and Developments districts in April 1978 provides environmental information on the Metro area in the following categories:

1. Population economic activity and land use

2. Water quality sampling and modeling

3. Development of technical subplans

4. Management planning

5. Environmental assessment

6. Public participation and area-wide planned recommendations

11.3 Metropolitan Planning Commission

The Metropolitan Planning Commission has some information on a variety of natural resource data and manmade feature data that is discussed in the next section. They maintain Geographic Information System (GIS) data layers for Metro Nashville and Davidson county. Data layers including property mapping, topography (5-foot contours), edge of pavement, road centerline, building footprints, approximate stream and other hydrologic feature locations, major TVA utility data, railroads, zoning, and other similar mapping information. This data is available in hard copy or electronically. The Planning Commission can be contacted by an individual or company to set up an internet user agreement that allows them to download electronic data in DXF format. It can also be purchased directly from the Planning Commission in Arc/Info, DXF or Arc/Info Export formats. Electronic GIS data can currently (April 2000) be purchased electronically for $71.00 per map tile which corresponds to an area of 12,000 by 8,000 feet on the ground. Currently (April 2000) a standard topography plot will generally cost $20.00 and a standard zoning plot will cost $6.00.

For more information on data sales or for obtaining an internet user agreement for professional access contact the Metro Planning Commission Mapping Services at (615) 862-7181.

11.4 Natural Resources Data

The major categories of pertinent natural resources data include precipitation, soils, topography, streamflow and flood history, and groundwater.

11.4.1 Precipitation

Published precipitation data should be collected by the following procedure:

1. Select an appropriate procedure for hydrologic calculations using information presented in Chapter 2.
2. Determine the type of precipitation data that are needed. Generally, either IDF curves or hyetographs for historic or design storm conditions are used (see Chapter 2). However, hourly electronic precipitation data is also available electronically. The National Climatic Data Center (NCDC) for National Oceanic and Atmospheric Administration (NOAA) has fee based historical and near real-time on-line climate data available for Tennessee at http://www.srcc.lsu.edu/srcc.html.

MWS compared rainfall records from the Nashville Metropolitan Airport with published data from the National Weather Service for durations of 1 to 24 hours. The 1-hour rainfall depths determined from a frequency analysis of the airport data agreed well with HYDRO 35 (Frederick et al., 1977), which was consequently adopted for durations of 1 hour or less. It was determined that TP-40 (Hershfield, 1961) underestimates rainfall depths compared to recorded data for durations of 1 to 24 hours, and that the results of the frequency analysis should therefore be used for those durations. For applications requiring larger durations (such as some storage facilities), the National Weather Service publication by Miller (1964) is appropriate for determining rainfall depths. All such duration data are compiled for ready use in Chapter 2 and replace data published in the previous Metro Nashville Drainage Manual.

11.4.2 Soils

Information from the SCS Soil Survey of Nashville and Davidson County completed in 1981 is suitable for use in hydrologic modeling, but additional site-specific data from more detailed or later surveys may be appropriate as a supplement to the published data. The NRCS publication Erosion and Sediment Control Handbook for Urban Areas and Construction Sites in Tennessee (1974) contains valuable information for quantifying erosion and specifying erosion control practices, with graphs and charts tailored for Tennessee. Selected information from the NRCS Soil Survey and Handbook can be found in Chapter 9. However, it should be noted that erosion prevention and sediment control practices presented in Volume 4, Sections 3 (TCP) and 5 (PESC) are to be applied in the Metropolitan Nashville and Davidson County area.

11.4.3 Topography

USGS 7.5-minute topographic quadrant maps of the area were prepared in 1968 and photorevised in 1983 for planimetric features only. The maps, which are at a scale of 1 inch to 2,000 feet with a contour interval of 10 feet, are suitable for general information on larger basins. There are several commercially available products that have the USGS quadrant map data in an electronic format. This and a wide variety of other maps, databases, reports and publications may also be purchased from the USGS and/or downloaded from the world wide web site (http://www.usgs.gov/pubprod/).

MWS has prepared 1-inch to 200-foot scale topographic maps with a contour interval of 5 feet for Davidson County based on aerial photography from 1996. This data can be acquired from the Metro Planning Commission. These maps should be used for drainage boundary delineation and measurement, time of concentration flow path analysis, and flood plain mapping. More current mapping information may also be available at the time of study.

11.4.4 Streamflow

The USGS maintains a computerized data retrieval system that includes all the stream gage data available throughout Davidson County. Numerous gage sites of various types and with specific individual purposes have been identified. Data include coincident rainfall-runoff data, time series of annual peaks, and occasional crest-stage measurements. All available pertinent data should be reviewed before completing hydrologic studies. For a summary of existing data on each watershed the USGS should be consulted.

11.4.5 Groundwater

Data on groundwater levels and movements could be obtained from information on existing detention ponds and other ponds in the area; existing nonpumping wells or wells that could be temporarily shut off to determine the static groundwater level; observations made by inspectors and others during construction of sanitary sewers, storm sewers, and major buildings; and regional or areawide reports prepared by the USGS or state agencies. If existing data sources are not sufficient to define the position of the groundwater table, it may be necessary to construct special observation wells, particularly at potential sites of detention facilities. These wells could be installed in the boreholes used to take soil samples during a site-specific subsurface exploration.

11.5 Manmade Features

11.5.1 Land Use

If historical information such as flood records or high water marks is being considered in an analysis, older land use maps or aerial photos should be sought that can identify early conditions that may have undergone change. MWS should be contacted about the availability of such data.

Existing land use can first be ascertained from recent land use maps, zoning maps, and aerial photos. Data should be confirmed by checking the Property Map Book at the MWS office, and any recent development indicated therein verified by field reconnaissance. The effective date of the existing land use should be noted for future reference.

Future land use projections can be developed from information on planned development for the Nashville area in the "General Plan for Nashville" or the 14 subarea plans discussed in Section 11.2.2. The latest amendments to the plans should be obtained.

11.5.2 Impoundments

Site-specific watershed master plans, available from MWS as appended material to the manual, provide information on impoundments that impact basin-wide hydrologic analysis, and should be consulted before field reconnaissance is performed to assess the effect of impoundments on a particular site.

11.5.3 Stream Channels

Natural stream channel features, conveyance improvements, and channel conditions are generally described in the appended watershed master plans from MWS. More detailed information may be available from engineering reports for specific projects. For older facilities and areas not covered by watershed master plans, field surveys may be required to obtain appropriate information.

11.6 Field Investigations

A field investigation should be made at each site to verify site-specific conditions and obtain survey data when published data are inadequate.

11.6.1 Site Visits

Site visits allow the engineer to verify information obtained from published sources, identify conditions relevant to the analysis, and obtain first-hand knowledge concerning site-specific constraints. Each aspect of the analysis should be field-verified.

Soil and soil cover are important components requiring field verification. Rock outcrops, vegetative cover, and land use are important to select appropriate runoff coefficients or SCS curve numbers. Total imperviousness and the degree of hydraulic connection to the channel network are both important in developed areas. Unusual circumstances concerning interception of depression storage should be recorded for use in assigning a value of initial abstraction of rainfall.

Drainage boundaries identified on topographic maps should be checked, as streams may often have been re-aligned or mapped incorrectly, storm sewers may cross apparent drainage divides, or drainage divides shown on the maps may be incorrect. Areas draining to detention ponds or storm sewers should be identified when such features may impact peak flow calculations. Where flow routing or time of concentration is involved, flow paths should be observed for channel geometry and hydraulic roughness, or for length of flow for overland and gully flow. Overland and channel slopes can be measured from topographic maps unless substantial regrading has occurred.

Channel geometry and constrictions should be considered in locating cross sections and assigning roughness coefficients for hydraulic modeling to establish flood elevations and flood plain and floodway limits. High water marks and other historic flood evidence should be recorded in detail for future reference.

11.6.2 Stream Channels

Stream channel surveys may be required to obtain hydraulic data for calculating design flood elevations, flood plains, and floodways. They are also necessary to define buffers for no-disturbance as defined by Volume Sections 2.2.14 and 5.9. Project-specific needs and the method of hydraulic analysis will dictate the type of data to obtain and the accuracy required for a particular application. In general, the following guidelines will apply:

1. In determining permissible disturbance areas the floodway or the top of the stream bank must be determined and buffer width measurements taken in accordance with Volume 1 Section 5.9.

2. For backwater calculations, cross sections are needed at representative locations throughout a stream reach, at locations where changes occur in discharge, slope, shape, or roughness, and at bridges or control structures such as weirs. Where abrupt changes occur, several cross sections should be used to describe the change. Cross section spacing is also a function of stream size, slope, and the uniformity of cross section shape.

3. At any cross section, sufficient data points should be field surveyed to describe the section geometry representative of the nearby reach. Local depressions and tributary channels generally do not contribute to effective flow area and should be excluded from the cross section. Special attention should be given to the channel and below water areas where most flow occurs. Additional points should be taken where the flood plain ground slope changes. Cross sections should extend beyond the expected flood plain limits.

11.6.3 Survey Requirements

Survey information is generally required for the following items:

1. Channel cross sections
2. Elevation reference marks (ERMs)
3. Hydraulic structures

Third-order leveling closures within ±0.05 times the square root of the distance, in miles, shall be used to tie temporary bench marks and ERMs to the National Geodetic Vertical Datum (NGVD) of 1929; to determine the elevation of high-water marks; and, where needed, to establish vertical control for aerial photogrammetry. Available vertical control and detailed topographic maps should be used whenever possible.

Field surveys should normally be accomplished by trigonometric or differential leveling using transit-stadia or transit-electronic distance measurements, with vertical error tolerances of ±0.5 foot across the 100-year flood plain. Cross section elevations should be determined at those points that represent significant breaks in slope. Each cross section shall cross the entire 500-year flood plain and should be carefully selected to be representative of reaches that are as long as possible, without permitting excessive conveyance change between sections.

ERMs shall be established and recorded in and near the flood plains of streams studied in detail. These shall include existing elevation references and those ERMs that can be established in the course of setting temporary bench marks for cross sections or vertical control for photogrammetry. Third-order leveling methods and standards of accuracy, as defined above, shall be used for any ERMs determined for the study.

Necessary dimensions and elevations of all hydraulic structures and underwater sections along the streams shall be obtained from available sources or by field survey where necessary. Elevations of hydraulic structures may not be established by aerial photogrammetric methods.

CHAPTER 12
COMPUTER PROGRAMS

Chapter 12
COMPUTER PROGRAMS

SYNOPSIS

The procedures covered in this manual can, in general, be performed using computer programs. To promote consistency in the analysis and review of projects, general guidance is presented in this chapter for selecting appropriate computer programs dealing with flood hydrology, open channel hydraulics, culverts, storm sewers, detention flood routing, and bridge hydraulics.

12.1 General Guidance

Many hydrologic and hydraulic computer programs have been developed for the analysis and design of stormwater facilities. Each program is generally intended to meet a specific type of application. It is not the intent of MWS to restrict stormwater analysis to a select group of programs, but to encourage engineers to select the computer model best suited to the task at hand. However, if site characteristics and project requirements allow, MWS would prefer the use of the models identified in this chapter, which have proven useful in the Metro area. MWS personnel are familiar with their application and will be better able to review and evaluate plans based on these models.

Computer modeling must not replace engineering judgment and experience in stormwater analysis and design. The theory behind the model must be fully understood by the engineer to produce meaningful results. Modeling parameters must be calibrated or verified by field measurements or other comparisons to give confidence to the results. Accuracy of the results should not be based on the sophistication of the computer model, but on the experience of the modeler and on the correlation of results with observed data.

12.2 Flood Hydrology

MWS has developed flood hydrograph models for selected watersheds in the Metro area using the HEC-1 computer program developed by the U. S. Army Corps of Engineers (1990). Data for these models have been carefully developed and verified using calibration storms and long-term stream gage records. Available basin-specific HEC-HMS (1998) or HEC-1 (predecessor to HEC-HMS) data files can be obtained from MWS on microcomputer diskettes so that an applicant can perform consistent hydrologic analyses in these areas. The existing models should be limited to the analysis of drainage areas of 0.5 square mile or larger.

For drainage areas smaller and more detailed than the original HEC-HMS or HEC-1 setup, a new HEC-HMS or other model may be necessary; however, the results should be consistent with the HEC-HMS or HEC-1 models on file at MWS. Alternatively, the Stormwater Management Model (SWMM) developed by the Environmental Protection Agency (Huber et al, 1992;

Roesner et al, 1994) may be used. The ability to link or incorporate water quality data may warrant the use of this model, particularly in considering water quality impacts of on-site and regional facilities.

If hydrologic models other than HEC-HMS or SWMM are used, MWS prefers programs that utilize NRCS (formerly SCS) rainfall-runoff procedures and unit hydrograph theory to develop flood hydrographs. Methods of analysis and results should be consistent with previous studies wherever possible.

12.3 Open Channel Hydraulics

MWS has developed water surface profile models of many streams throughout the Metro area using the HEC-RAS and HEC-2 computer program developed by the U.S. Army Corps of Engineers (1998 and 1982). Data for these models have been carefully developed and verified where historic information was available. Available site-specific HEC-RAS and HEC-2 data files can be obtained from MWS on microcomputer diskettes to facilitate consistency when changes are proposed for these areas. Alternatively, the Stormwater Management Model (SWMM) developed by the Environmental Protection Agency (Huber et al, 1992; Roesner et al, 1994) may be used. New HEC-RAS or SWMM data files should be developed using field-surveyed data consistent with the requirements of Chapter 11, Section 11.5.3.

12.4 Culverts

The FHWA microcomputer program HY8 is useful in hydraulic analysis of culverts. This computer program is based on HDS-5 (USDOT, FHWA, 1985) and can perform reservoir routing in addition to evaluating culvert hydraulics. The program is primarily analysis oriented, but may be revised to provide design capabilities.

12.5 Storm Sewers

Many flood hydrology models do not have the capability to handle pressure flow in storm sewers. The EXTRAN block of the U. S. Environmental Protection Agency (EPA) Stormwater Management Model (SWMM) has this capability and is recommended for evaluating pressure flow in closed conduits. Flood hydrographs can be generated using the RUNOFF block of SWMM or can be computed externally and input as data files for use by EXTRAN.

12.6 Detention Flood Routing

Most hydrologic models, including HEC-HMS, are capable of performing flood routing through detention ponds. For watersheds covered in the master plans appended to this manual, the HEC-HMS data files should be obtained from MWS to evaluate flood detention requirements for a project (note that subbasin areas for these data files are greater than 0.5 square mile). When using models other than HEC-HMS to generate flood hydrographs for areas studied previously, consistent methods of analysis and results should be achieved.

12.7 Bridge Hydraulics

The preferred computer program for bridge hydraulic analysis are the HEC-RAS or SWMM models.

12.8 Water Quality

There are a wide variety of software packages that are designed to model stormwater quantity and/or quality. Some models are intended for very detailed, data intensive analysis of water quality. While most applications of water quality management practices will not require modeling and can be designed with criteria established in Volume 4, there may be large systems requiring modeling. The preferred water quality model is the Watershed Management Model 4.15 or later developed for the Rouge River National Wet Weather Demonstration Project (2000) applying principals discussed in the users manual (1998) to comparatively determine the optimal location of various management practices. This model is available at: http://www.waynecounty.com/rougeriver/proddata/wmm.html. With permission from MWS, other models discussed in "Compendium of Tools for Watershed Assessment and TMDL Development" (1997) may be used to perform water quality analyses.

CHAPTER 13
STORMWATER POLLUTION PREVENTION PLAN

Chapter 13
STORMWATER POLLUTION PREVENTION PLAN

Synopsis

Pollution prevention can take many forms at a construction site of an industrial/commercial facility. Some practices may be self-evident, while others may be subtle changes in employee routines and habits. However, there are some practices that require decisive planning, implementation and follow-up, inspection, and maintenance. This chapter is intended to provide guidance for preparing a Stormwater Pollution Prevention Plan (SPPP) in the form of the existing grading permit application process. This gives the contractor or employees awareness of the possible source of pollution on the work-site and suggests measures that prevent that pollution from entering the environment. It will guide designers, reviewers, inspectors, construction site operators, and contractors to Volume 4 Stormwater Best Management Practices (BMP) for a selection of acceptable measures that can be incorporated into an effective SPPP. It is intended to be a discussion parallel to Volume 4-Section 1 with broader implications to contractor management practices and industrial/commercial management practices.

This chapter predominately provides guidance for construction site SPPP preparation, but also provides a brief synopsis of industrial /commercial site SPPPs. Industrial and commercial sites should also consult the Code of Federal Register (CFR) for additional requirements and guidance. In particular 40 CFR Part 112.7 – *Guidelines for the preparation and implementation of a Spill Prevention Control and Countermeasure (SPCC) Plan* and 40 CFR part 112 – *Oil Pollution Prevention* should be reviewed.

13.1 Introduction to SPPP

In essence, a SPPP is a flexible plan by which site-specific pollution sources are identified and structural or non-structural measures are designed, implemented, maintained, and possibly redesigned to minimize negative impacts on the surrounding environment. This is a step beyond pre-2000 releases of the Stormwater Management Manual. While a SPPP predominately focuses on stormwater quality, it also gives consideration for air and groundwater quality.

For Metropolitan Nashville and Davidson County, the existing grading permit process administered by Metro Water Services will be the mechanism for preparation, review and approval of a SPPP. Many portions of what is traditionally known as a SPPP are already required by the grading permit process. However, the SPPP development described in this chapter does require that much of the data be considered in a different way and to a further extent in order to adequately address stormwater pollution prevention as required by Volume 1-Section 2.2.

Since the SPPP will be developed as part of the grading permit process, it does not have to be prepared as a separate document. However, the differences in how a SPPP should be distributed for a construction project and an industrial/commercial facility should be noted. To ensure that the SPPP provisions are understood and properly implemented on a construction site, it is best if they are intermingled with the construction plans and specifications to be used by the contractor. This will ensure that the SPPP is a project long process and will also ensure that the design engineer is considering the SPPP throughout the site design process and not as an afterthought. If the SPPP is developed for operation and/or maintenance (O&M) of an industrial /commercial facility, then it should be intermingled with new-employee and periodic training on related O&M procedure documentation.

13.2 SPPP Phases

The basic phases of SPPP preparation are the same for both a construction site and an industrial/commercial facility. They are illustrated in the Figure 13-1.

The SPPP for construction sites should focus predominately on temporary pollution prevention practices and address long-term or permanent pollution prevention measures that are implemented during the construction phase. The SPPP for industrial / commercial sites should focus on permanent or long-term practices developed for pollution prevention.

The SPPP phases are basically the same for construction and industrial / commercial development, except for some differences in how the two are implemented. If the difference is not noted in the following sections, then it can be assumed that the same basic procedure applies.

It is important to note that the process parallels the "traditional" site development design process described in Volume 1-Chapter 4. The SPPP preparation should be integrated into the construction design process and not left as an afterthought to site design. SPPPs that are prepared as an afterthought may cause the site to be redesigned, be more expensive to implement, increase long-term maintenance responsibilities, and reduce aesthetic characteristics.

Figure 13-1
SPPP Phases

Site Evaluation and Design Development
- Collect Site Information
- Develop Site Plan and Map

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Assessment
- Hydrologic and Receiving Waters Evaluation

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Practice Selection / Plan Design
- Select Permanent Treatment Practices
- Select Erosion Prevention and Sediment Control (EP&SC) Practices
- Select Other Temporary Practices
- Prepare Inspection And Maintenance Plan/Requirements
- Prepare Phased Schedule

↓

Certification, Notification and Plan Approval
- General Storm Water Permit NOI Certification
- Other Permitting Requirements Notification
- Grading Permit Application and Review Process
- Pre-construction Inspection

↓

Construction / Implementation
- Implement / Construct Practices
- Inspect and Maintain Practices
- Update / Change SPPP
- Reporting Requirements for Spilled Materials

↓

Final Stabilization / Termination and Evaluation
- Final Stabilization
- Notice of Termination
- Evaluate SPPP Performance

13.3 Construction Site SPPP

13.3.1 Site Evaluation and Design

13.3.1.1 Collect Site Information

Site information principally consists of the existing conditions survey (topography, existing site improvements, etc.), soils, receiving waters identification, and rainfall data. The data collection task should identify pollutants that are already on-site whether in the form of soils, groundwater, surface water or other means. The list / survey of on-site pollutants and other existing conditions will serve as the basis for all other SPPP preparation activities. Other site data collection activities are described in detail in Volume 2-Chapter 11.

For indoor construction activities such as renovations at a commercial or industrial facility, this task is generally not as intensive. However, there are several tasks that should be completed. The Superfund Amendments and Reauthorization (SARA) Title III Section 312 and 313 reports may be a quick tool in completing several of the tasks. The following tasks should be considered.

1. Complete a materials inventory that includes all hazardous or non-hazardous materials stored or handled that have a bearing on the potential for stormwater pollution. The Material Safety data sheets of all on-site stored or handled materials should be reviewed.

2. Complete a spills inventory that includes spills over the previous five years. This will be a useful tool in determining areas or personnel that may have a bearing on the SPPP.

3. Begin an activities list. If the list changes in any way, it may be time to reevaluate the SPPP.

4. Conduct a facility tour to confirm or locate activities and stormwater management practices. At a minimum, the following questions should be answered:
   a. Are catch basins full of sediment and not being cleared regularly?
   b. Are there signs of spills, pools, stains, or other traces of fuels and chemicals that have been spilled?
   c. Do you find leaking equipment, pipes, containers or lines?
   d. Do storage containers show signs of leakage?
   e. Are containers and exposed piping labeled?
   f. Are cleanup tools and materials readily available (drip pans, absorbents, rags, etc.)?
   g. Are there spill containment or other structural secondary containment measures in place where oils, greases, toxic or hazardous materials are stored or used?
   h. Where does stormwater from the site enter the drainage system (Metro infrastructure, creek, stream or river)? Are these outfalls protected with any type of stormwater quantity or quality BMP?

i. Does the facility show signs of poor housekeeping such as accumulated debris in open areas that is not being swept, uncovered materials, etc.?

5. Identify non-stormwater discharges (or outfalls) to the stormwater (drainage) system. This may include process water, cleaning or rinse waters or even sanitary sewer lines. If any of these waters are entering the stormwater system then contact the MWS Stormwater NPDES Office at (615) 880-2420.

13.3.1.2 Develop Site Plan and Map

The site plan and map are a narrative and visual description of the SPPP. They are interdependent document designed to provide details that can only be provided in one form or the other. Key pollution prevention principals that should be considered for construction sites include the following.

1. Install perimeter sediment control practices before any other land disturbance activities.
2. Disturb the smallest vegetated area possible.
3. Phase or schedule land disturbance activities.
4. Stabilize denuded areas quickly or before rain events with erosion prevention practices.
5. Keep cut and fill volume to a minimum by designing structures to the site rather than the site to fit the structures.
6. Limit impacts to steep slopes, waterways, sinkholes, and wetlands.
7. Consolidate or centralize contractor management activities such as material storage, concrete truck washing, vehicle maintenance and fueling, etc.

In addition to reducing the potential for pollution to leave the construction site, incorporating the above pollution prevention principals can also: reduce construction costs for grading and landscaping, reduce the amount of sediment and stormwater management controls, and improve the aesthetics of the completed project.

For SPPP consideration, the maps need to show pre and post construction slopes, drainage patterns, disturbed areas and disturbance restrictions such as a regulated buffer (Volume 1-Section 5.9), steep slopes, sinkholes or wetlands. Pre and post construction slopes are generally shown on a single plan sheet with different line types and symbols. Similar plan sheets should be prepared indicating the flow directions and drainage catchments for pre, post, and intermediate construction phases. The intermediate flow paths and drainage catchments will impact the design requirements for erosion prevention or sediment control measures. The same plan sheets can be used to delineate phased disturbance areas and stabilization techniques. All of the plans should indicate sensitive areas and buffers that shall not to be disturbed by any type of construction activity. Similarly, the plans should indicate areas where vehicles should be directed, such as entrances, interim stabilized roads, etc.

The narrative portion of the SPPP should be presented in notes on the construction plans. They should be specific enough to match details presented on the construction plans. In particular, it is preferred that separate sheets be used to describe the phased implementation of various management practices and other construction activities (such as clearing and grubbing, excavation and stockpiling, rough grading, final or finished grading, demolition, etc). The SPPP shall be stamped by a registered professional engineer in Tennessee.

For industrial or commercial operations (non-construction) a more simple map or site rendering may be appropriate. It should still indicate drainage areas or general flow paths and sensitive areas. It should also indicate areas where hazardous or other materials are stored or used.

13.3.2 Assessment

This task focuses primarily on the hydrologic evaluations that are described in Volume 2-Chapter 2. These activities are also required to develop stormwater quantity controls and involve calculating site drainage areas, runoff characteristics, and determining if any special provisions may be necessary to prevent negative impacts to receiving waters. Drainage areas should be calculated manually or electronically from accurate survey data. The runoff characteristics can be calculated in terms of the rational method runoff coefficient or the NRCS curve number as discussed in Volume 2-Section 2.3. The sensitivity of the receiving waters may guide the designer to implement more intensive internal and/or perimeter controls for sediment or other water quality issues. A good guide to determining the sensitivity of a receiving water is to determine if and why it appears on the Tennessee Department of Environment and Conservation (TDEC) 303(d) list of threatened or impaired waterways. The list may be acquired from TDEC at http://www.state.tn.us/environment/wpc/index.html.

For an industrial/commercial site, the results of the inventories and site tours described in Section 13.3.1.1 should be used to guide an assessment for how intensive stormwater pollution prevention practices need to be implemented. Special emphasis should be given to BMPs at outfalls and the receiving water sensitivity.

13.3.3 Practice Selection / Plan Design

Volume 4-Section 1 discusses the process for selecting BMPs in terms of defining objectives and categories in conjunction with the “BMP Treatment Train”. The March 2000 release of Volume 4 has an extensive set of BMP fact sheets sectioned as follows.

Section	Description
2	Contractor Management Practices – CP
3	Temporary Construction Site Management Practices – TCP
4	Industrial / Commercial Management Practices – ICP
5	Permanent Erosion Prevention and Sediment Control practices – PESC
6	Permanent Treatment Practices – PTP

Section 1 should be reviewed to help guide in the preliminary BMP selection process and Sections 2 through 6 should be reviewed to guide the final selection, design, construction / implementation, inspection and maintenance requirements, and other related discussion topics.

13.3.3.1 Select Permanent Treatment Practices

Volume 4-Sections 1.5 and 1.6.5 provides additional detail about selecting permanent treatment practices that address stormwater quantity (flood control) and quality. They refer to Volume 4-Section 6 (Permanent Treatment Practices – PTP) and should be reviewed to guide the selection of permanent practices.

It is preferred to integrate permanent stormwater management practices into the temporary measures. This often takes the form of an over-excavated detention pond and modified outlet structure so that it functions as a sediment trap or pond that holds the large sediment loads expected during construction. Other measures include, but are not limited to, protecting / reinforcing selected (not all) catch basins to trap sediment or using areas that will not be disturbed such as buffers and filter strips. Integrating permanent stormwater management features and site conditions into the temporary measures is generally most cost-effective, easier to construct, inspect, and maintain.

In Metropolitan Nashville and Davidson County, all construction sites of a size requiring a grading permit must have perimeter (outfall) EP&SC practices installed and inspected by Metro personnel before additional construction is permitted. This means that if a detention pond is going to serve as the control before discharging water from the site, then it must be constructed and operational before any other clearing and grubbing, grading or excavation can proceed. This is true whether the detention pond is designed to meet stormwater quantity and quality requirements or only stormwater quality requirements. In cases where a permanent pond will not be installed, some type of sediment trap/basin should be installed.

While integrating temporary and permanent treatment practices is generally not a concern for non-construction activities at industrial and commercial sites, it is important that employees are aware of their function and sensitivities (how their function can be impaired or overwhelmed).

More detailed guidance on selection and design of permanent treatment practices is provided in Volume 4-Sections 1 and 6 (Permanent Treatment Practices – PTP).

13.3.3.2 Select Erosion Prevention and Sediment Control (EP&SC) Practices

Volume 4-Section 1.6.4 provides additional detail about selecting EP&SC practices that minimize disturbed area, stabilize disturbed areas, protect the upstream and downstream site perimeter, create internal swales and ditches, minimize internal erosion and protect stormwater inlets and outlets. They refer to Sections 3 (Temporary Construction Site Management Practices

– TCP) and 5 (Permanent Erosion Prevention and Sediment Control practices – PESC). These Sections in Volume 4 should be reviewed to guide the selection of EP&SC practices.

This activity is generally not applicable to SPPPs for non-construction activities at industrial and commercial sites. However, inspection and maintenance of permanent EP&SC measures should be considered.

13.3.3.3 Select Other Temporary Practices

As discussed in Volume 4-Section 1.4, there are many other potential pollution problems on a construction site besides erosion and sedimentation. These include nutrients, oxygen demanding substances, metals, pesticides, oil, grease, fuels, toxic chemicals, and miscellaneous wastes. These pollutants can originate from a variety of activities including paving operations, painting, sandblasting, demolition, materials storage, equipment fueling and maintenance, and other daily activities necessary for project construction or site (industrial or commercial) management. Volume 4-Sections 2 (Contractor Management Practices – CP) and 4 (Industrial / Commercial Management Practices – ICP) contains a series of management practices that are intended to minimize the impact of non-sediment pollutants. These sections of Volume 4 include a variety of activities, often referred to as “good housekeeping practices,” that seem to be common-sense and self-evident activities, but are often overlooked or taken for granted on sites, to the point where the pollutant sources are not recognized or identified. For industrial/commercial sites, employee training about the SPPP provisions is critical for ensuring that the plan is effectively implemented and maintained.

13.3.3.4 Prepare Inspection and Maintenance Plan / Requirements

Inspection and maintenance procedures for a SPPP are often not well thought out and not effectively carried out. Often the procedures are overlooked because responsible personnel have not been identified. These procedures need to be developed in realistic terms and with expectations so they can be thoroughly explained to appropriate personnel. Ideally a pollution prevention team lead by a team coordinator and composed of a sufficient number of personnel to cover all aspects of the site operation should be established. The team should be used as a mechanism to discuss and train all personnel in detail on the SPPP. The team members should perform routine inspections and inform the team coordinator of any changes in operations that may affect the SPPP.

For a construction site the pollution prevention team this should include the site foreman, vehicle maintenance and fueling personnel, heavy equipment operators (grading and buffer issues), etc; and for an industrial/commercial site the plant supervisor, hazardous material handlers, management, etc. SPPP review and discussion should be a part of new employee training/certification programs.

For a construction site, the inspection and maintenance responsibilities should be presented along with other construction plans, specifications, and notes. It should explain the frequency that inspections should be performed, how they should be documented, and typical maintenance activities. Generally, inspections should be performed and documented weekly or after a rain event greater than 0.25 inches. There are more specific inspection recommendations for BMPs presented in Volume 4.

Permanent facilities that are to be turned over to Metro or to another owner or organization should have detailed inspection and maintenance requirements and expectations documented. These requirements and expectations should be submitted as part of the grading plan approval process for review by MWS Stormwater Development Review. At a minimum, a Stormwater Detention Agreement should be prepared and accepted as presented in Volume 1-Appendix C. Similar agreements or explanation of responsibilities should be prepared for property owners.

For an industrial / commercial site, the inspection and maintenance responsibilities should be presented in operation and maintenance documentation. They should be discussed to an appropriate level in new employee training and in thorough detail to personnel that are responsible for inspecting and maintaining the facilities. This information should be periodically reviewed to ensure the requirements are “fresh” in the minds of the employees.

13.3.3.5 Prepare Phased Schedule

As discussed in Section 13.3.1.2 above, phased construction can provide a significant benefit in implementing an SPPP. This schedule does not need to be date specific, but should indicate what BMPs should be constructed or modified while other aspects of the construction are being executed. This may be as simple as noting that temporary seeding, mulch and geotextiles be applied/installed within one week of rough grading. Similarly, other task schedules can be noted on the construction plans. There should be an overall schedule presented that indicates the overall phases of construction that major BMPs will be implemented/constructed. At a minimum, this should show that the perimeter outfall controls will be in-place before any grading is performed.

A documented schedule is also necessary for industrial/commercial sites, but is applied differently than for construction. There should be a schedule of inspection and maintenance activities as described in the previous section. This can be prominently posted, but should at least be identified in some manner, as in employee training.

13.3.4 Checklist , Certification / Notification

13.3.4.1 General Storm Water Permit NOI Certification

As required by Volume 1-Section 4.2.2 each application for a grading permit shall be accompanied by a certification that a Notice of Intent (NOI) has been submitted to the Tennessee Department of Environment and Conservation (TDEC) for a "General Storm Water Permit" or a certification that it is not required. The review of the plans will proceed if the certification indicates that the applicant will submit the permit number at a later date. If the site requires a Tennessee General Storm Water Permit, the permit number must be submitted to MWS before a grading permit is issued. This certification is presented in Volume 1-Appendix A.

13.3.4.2 Other Permitting Requirements Notification

An application checklist is provided in Volume 1-Appendix A to assist the applicant in preparing a complete application package and thereby help with a timely review. The applicant shall attach a signed copy of the checklist with the application to certify that a complete package is being submitted. Some requirements of the checklist will not be applicable to all projects, depending on the permit being requested. These should be checked as not applicable. Omission of any required items shall render the plans incomplete, and they shall be returned to the applicant, or their engineer, for additional information.

The checklist in Volume 1-Appendix A includes a notice that the applicant be aware of certain land disturbance activities that will impact "Waters of the State", "Wetlands", and/or "Sinkholes", which may be required to meet certain State and Federal regulations. It is the responsibility of the applicant to seek out and obtain any applicable State and Federal permits prior to the initiation of any land disturbance activities.

13.3.4.3 Grading Permit Application and Review Process

Volume 1-Section 3.3 through 3.6 discusses the grading and building permit requirements, exemptions, and variances. Volume 1-Chapter 4 discusses the grading permit procedures and should be reviewed before preparing plans for submittal.

Completion of the "Plan Submittal Information Form" is especially important for timely plan review by MWS. Each plan being submitted to MWS is to be accompanied by a completed "Plan Submittal Information Form". This form is available in Volume 1-Appendix A or on the Stormwater Program's website at http://www.nashville.gov/stormwater. The application for a grading permit will not be accepted unless this form has been completed. Each application for a grading permit or a building permit referred to the MWS shall contain site preparation plans certified by a registered engineer, landscape architect, or land surveyor, as appropriate. Plans are to include grading, drainage, and EP&SC plans with appropriate plan and profile sheets for proposed streets or roads, and details of the stormwater quantity and quality management systems (or SPPP provisions).

13.3.4.4 Pre-construction Inspection

According to Volume 1-Section 2.2.7 Metro is required to inspect temporary or permanent EP&SC measures for a site before additional construction may proceed. In essence, Metro inspectors will check downstream perimeter sedimentation controls and/or upstream erosion prevention measures to ensure that they are properly located and operating to a level that can keep sediment from leaving the site while under construction. This does not mean that no other EP&SC measures will be needed, but that the BMPs presented on the grading permit application plans will work sufficiently with the perimeter controls. This also gives Metro the authority to prevent work from progressing if a permanent facility does not adequately address stormwater quantity (flood) management.

Metro reserves the right to require the grading plans be modified given any observations or evaluations that suggest stormwater quantity or quality management is not sufficiently addressed. This may be caused by differences between the actual site conditions and the plans or other data evaluated by Metro.

13.3.5 Construction / Implementation

13.3.5.1 Implementation / Construction Practices

After the plans have been accepted by MWS Stormwater Development Review, other appropriate agencies, and the pre-construction inspection has been completed to the satisfaction of the MWS, construction may proceed. To ensure that BMPs are adequately implemented / constructed, it is important that the work crews which install the measures are experienced, adequately trained personnel or are overseen by experienced, adequately trained personnel. Improperly implementing / constructing some BMPs can result in little or no positive effect and may actually intensify the pollution impact they are intended to minimize.

For an industrial / commercial site, practices are implemented by integrating them into daily, weekly and monthly routines. It is important that all key staff are aware of the plan and that it is accessible to all employees. A copy of the plan should be kept in a conspicuous location where all employees may review it, whether for normal activities or for emergency and spill response.

13.3.5.2 Inspection and Maintenance Practices

According to Volume 1-Section 2.2.15 Metro may require that an “erosion prevention and sediment control professional”, or other similar person designated by TDEC or Metro be on-site. This is to ensure that these types of practices are adequately maintained and that it is clear who shall be held accountable for the BMP performance.

As indicated in Section 13.3.3.4, inspections should be performed and documented weekly, after a rain event greater than 0.25 inches, or at frequencies indicated for specific BMPs identified in Volume 4. Permanent BMPs generally require less frequent inspection and maintenance. However, they should be inspected at rates presented in Volume 4. For a construction site, weekly inspections and maintenance should be done even if the plans or specifications do not indicate the procedures as recommended in Section 13.3.3.4. Maintenance issues identified by inspections should be resolved within a week or less, depending on the severity of the potential pollutant impact. BMPs that are found to be insufficient should be augmented or replaced with other BMPs that can more effectively manage the pollutant of concern.

As an example, a construction site detention pond is being overwhelmed with sediment to the point that it can not contain the sediment and may cause flooding problems. It likely needs to be cleaned out (excavated) to an effective depth and mulch or geotextiles applied to the slope tributary to the pond and/or check dams installed in the small channels leading into the pond.

For an industrial/commercial site inspection and maintenance procedures, BMPs should be documented in a readily accessible document, as explained in the previous section. There needs to be some sort of follow-up checking on personnel activities and structural BMPs should be routinely inspected. Some mechanically intensive BMPs such as stormwater treatment systems may need thorough periodic inspection by a technician trained in the operation and maintenance of that specific system.

13.3.5.3 Update / Change SPPP

Inevitably, there are site or operating conditions that are not anticipated. This requires flexibility in implementing the SPPP and overall grading permit. The ultimate goals are to limit the release of sediment from the construction site and to prevent other pollutants from being released into the surrounding environment. Metro reserves the right to enforce the implementation of additional controls even if all the measures presented in a grading permit (and SPPP) are implemented effectively. This means that even if a contractor has constructed and implemented all the BMPs shown or described in the construction plans, additional BMPs must be implemented if sediment or a pollutant is leaving the site in levels that could negatively impact the surround environment, in the opinion of Metro staff.

A similar rationale should be applied to industrial/commercial sites. If a BMP is not effectively preventing stormwater pollution, then it should be modified or replaced by more intensive practices.

As the SPPP is changed, appropriate personnel and the MWS should be notified. MWS reserves the right to require additional changes if the plan is deemed to be inadequate for the conditions, as determined by MWS.

13.3.5.4 Reporting Requirements for Spilled Materials

Spill prevention and control are discussed in more detail in Volume 4-Section CP-06. The following are general guidelines for reporting a significant spill.

1. Notify the Engineer immediately and follow up with a written report.

2. Notify the local emergency response agency at (615) 862-8530. In addition to calling 911, the contractor will notify the proper county officials. It is the contractor's responsibility to have all emergency phone numbers at the construction site.

3. For spills of state reportable quantities or into a waterbody or adjoining shoreline, the contractor shall notify the TDEC general hotline – environmental assistance at 1-888-891-8332 (TDEC).

4. For spills of federal reportable quantities or into a waterbody or adjoining shoreline, the contractor shall notify the National Response Center at (800) 424-8802.

5. Notification should first be made by telephone and followed up with a written report.

6. The services of a spills contractor or a haz-mat team shall be obtained immediately. Construction personnel should not attempt to clean up the spill until the appropriate and qualified staff have arrived at the job site. The level where this is necessary should be defined in the SPPP and understood by all personnel.

7. Other agencies which may need to be consulted include, but are not limited to, the Fire Department, the Department of Public Works, Metro Water Services, the Metro Police Department, OSHA, etc.

A similar response procedure as presented above should be included in the SPPP and include pertinent phone numbers for staff members and emergency personnel and agencies.

13.3.6 Final Stabilization / Termination

13.3.6.1 Final Stabilization

For a construction site, the permanent BMPs should be cleaned and stabilized to the conditions indicated in the construction plans. This may require detention pond re-excavation, final grading and geosynthetic stabilization, system flushing (Volume 4-Section CP-20), catch basin cleaning, water quality inlet/device cleaning, filter system media replacement, etc. All of these and other related activities will be required by Metro before a Use and Occupancy Permit is issued or a bond is released.

13.3.6.2 Notice of Termination

For construction sites, Metro manages construction site termination through the Use and Occupancy Permit or the bond release. They are contingent upon proper construction, stabilization and operation of all stormwater quantity and quality management practices and other construction. If permits or notification were required by other agencies, then it is the responsibility of the developer, or their representative, to terminate the project with regards to that other agency.

13.3.6.3 Evaluate SPPP Performance

When a construction project is completed, the developer, engineer, and contractor should evaluate the practices that were effective for the specific site conditions. Noting how management practices may be more effectively implemented will reduce the costs of implementing adequate stormwater pollution prevention measures on the next construction project.

For an industrial/commercial site, if a pollutant source has been eliminated, then it is acceptable to stop certain non-structural practices or not to utilize a specific structural management practice. If that is the case, the SPPP should be reevaluated and modified to account for the change.

13.4 References

The following documents were either quoted or used as reference guides in preparing this section. They may be useful as additional guidance for preparing and implementing effective SPPPs.

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