City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 1: Introduction

Volume 3: Best Management Practices Manual

Revision: 0
June 2006

EnSafe Inc.
5724 Summer Trees Drive
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ACRONYM LIST

A	Watershed drainage area, in acres, tributary to the design point -2
A	Cross sectional area of flow, (ft^2) -3
A	Soil loss, in tons/acre, for the time period selected for R -9
A	Sediment yield from a project site with erosion control, in tons/acre/year -9
A_i	Land area contained in sub-area i with uniform land use conditions, in acres or square miles
A_m	Drainage area, in square miles
A_T	Total area of watershed, in acres or square miles
AASHTO	American Association of State Highway and Transportation Officials
AHW	Allowable headwater depth
ARI	Average recurrence interval
AMC II	Antecedent Moisture Condition II
B	Hydrograph shape factor
b	Bottom width, (ft)
BMP	Best Management Practice
BMPM	Best Management Practice Manual
C	Broad-crested weir coefficient -8
C	Cropping management factor, dimensionless -9
C	Constant -3
C	Weighted composite runoff coefficient -2
C_d	Discharge coefficient
C_i	Runoff coefficient for sub-area I
C_o	Control due to any erosion control practice not noted above
C_r	Control due to runoff reduction practices -9
C_r	Unsubmerged discharge coefficient
C_s	Control due to surface stabilization
C_T	Runoff coefficient for return period T, dimensionless
C_{10}	Runoff coefficient for a design storm return period of 10 years or less
CDM	Camp Dresser and McKee, Inc.
CMP	Corrugated Metal Pipe
CN	Curve Number
CN_i	Curve number for sub-area i with a given combination of uniform hydrologic group and land use conditions
CP	Control Practice Factor
d	Depth of flow, (ft) -3
d	Selected stone diameter, in feet -3
d_c	Critical depth, in feet
d_e	Equivalent hydraulic depth at outlet, in feet
dt	Routing computational interval, in seconds
d_o	Depth at outlet, in feet
d_w	Culvert Width
d_1	Depth above jump, in feet

d_2	Depth below jump, in feet
d_{30}	Diameter of stone for which 30 percent, by weight, of the gradation is finer, in feet
d_{50}	Diameter of stone for which 50 percent, by weight, of the gradation is finer, in feet
D	Hydraulic depth, (ft), (D= A/T) -3
D	Height of culvert opening, in feet -5
D	Pipe diameter, in feet -6
D_c	Sediment delivery ratio with controls
DA	Drainage Area
DD	Number of Increments
DDF	Depth-Duration-Frequency
EL_{hi}	Headwater elevation for inlet control, in feet
EL_{ho}	Headwater elevation for outlet control, in feet
EL_i	Elevation of the culvert entrance, in feet
EL_{sf}	Elevation of the streambed at the face
EL_o	Elevation of the culvert outlet, in feet
EI	Erosion Index
Eo	Gutter Flow Ratio
EP&SC	Erosion Prevention and Sediment Control
F_p	Pond and swamp adjustment factor
Fr	Froude Number
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FHW	Federal Highway
g	Acceleration due to gravity, (32.2 ft/sec^2)
h_e	Eddy head loss, in feet
h_o	Design tailwater, in feet
h_s	Scourhole Depth
H	Total head loss for outlet control, in feet -5
H	Head above the weir crest excluding velocity head, in feet -8
H	Head on vortex of notch, in feet -8
H_1	Friction head loss in the tapered inlet, in feet -5
H_1	Upstream head above crest, in feet -8
H_2	Downstream head above crest, in feet
H_c	Height of weir crest above channel bottom, in feet
H_f	Energy loss due to friction, in feet
Hm	Minimum height
H_L	Head loss due to pipe form conditions, in feet
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center – River Analysis System
HEC-9	Hydraulic Engineering Circular No. 9
HEC-14	Hydraulic Engineering Circular No. 14
HEC-18	Hydraulic Engineering Circular No. 18
HDS-1	
HSG	Hydrologic Soil Groups
HW	Headwater

HW_i	Headwater depth, for inlet control, in feet
HW_r	Upstream depth
HW_o	Headwater depth for outlet control, in feet
I	Rainfall intensity
I_a	Initial Abstraction
I_{tc}	Average rainfall intensity, in inches/hour, during a period of time equal to t_c or the return period T
I_2	Inflow rate at the end of routing time period Δt, in cfs
I_1	Inflow rate at the beginning of routing time period Δt, in cfs
IDF	Intensity-duration-frequency
K	Muskingum channel routing time constant for a particular channel segment -2
K	Loss coefficient for pipe form conditions -5
K	Soil erodibility factor, in tons/acre/R unit -9
K	Channel conveyance -3
k_e	Eddy head loss coefficient, in feet -3
k_e	Entrance loss coefficients -5
K_tC_r	Discharge coefficient
K_t	Submergence factor
K_T	Trapezoidal open channel conveyance factor
K_p	Circular pipe open channel conveyance factor
k_v	Pipe velocity factor
L	Approximate length of culvert, in feet -5
L	Broad-crested weir length, in feet -8
L	Barrel length, in feet -5
L	Length of roadway crest, in feet -5
L	Conduit length, in feet -6
L	Horizontal weir length, in feet -8
L	Sheet flow length, in feet -2
L_a	Apron Length
L_B	Basin length
L_i	Length of the flow path for segment i, in feet
L_s	Scourhole Length
L_1	Length of the tapered inlet, in feet
LoD	Loring Silt Loam
LS	Length-slope factor, dimensionless
MCC	Municipal Code Corporation
MDPW	
MeB	Memphis Silt Loam
m_5	Coefficient associated with the degree of channel meandering
n	Total number of areas with uniform runoff coefficients -2
n	Manning's roughness coefficient for sheet flow -2
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

N	Number of diversions placed across a uniform slope
N_s	Number of blocks
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
O_1	Outflow rate at the beginning of routing time period Δt, in cfs
O_2	Outflow rate at the end of routing time period Δt, in cfs
O_1	Outflow rate at time 1, in cfs -8
O_2	Outflow rate at time 2, in cfs -8
O_3	Outflow rate at time 3, in cfs -8
P	Erosion control factor, dimensionless -9
P	Wetted perimeter (the cross section length touched by water), (ft) -3
P	Perimeter of grate opening, in feet -4
P	Precipitation -2
P_T	Precipitation depth for return period T, in inches
P_2	2-year, 24-hour rainfall, in inches
PESC	Permanent Erosion Prevention and Sediment Control
PI	Plasticity index from Atterburg limits
PTP	Permanent Treatment Practices
q_p	Peak flow rate, in cfs
q_u	Unit peak discharge, in cubic feet per second per square mile per inch (csm/inch)
Q	Flow rate, in cfs -2
Q	Discharge rate, (ft^3/s) -3
Q	Gutter flow upgradient (upstream) of inlet ($Q_{upstream}$), in cfs -4
Q_i	Gutter flow intercepted ($Q - Q_{downstream}$), in cfs -4
Q_i	Rate of discharge into grate opening, in cfs -4
Q_i	Peak inflow rate -8
Q_f	Free flow, in cfs
Q_o	Peak outflow rate -8
Q_o	Overtopping flow rate, in cfs -5
Q_s	Submergence flow, in cfs
Q_T	Peak runoff rate for return period T, in cubic feet per second (cfs)
Q(Y)	A representative flow rate for channel routing at representative depth Y, in cfs
R	Rainfall factor -9
R	Hydraulic radius, (ft),
R_c	Mean radius of the bend, in feet
R_T	Rainfall excess for return period T, in inches
RUSLE	Revised Universal Soil Loss Equation
s	Average slope of sheet flow path, in feet/foot
S	Maximum soil storage, in inches -2
S	Energy grade line slope, (ft/ft) -3
S	Longitudinal street grade, in feet/foot -4
S_f	Friction slope, in feet/foot
S_f	Fall slope -5
S_o	Slope of channel bottom, in feet/foot -2
S_o	Barrel slope, in feet/foot -5

S_v	Saturated shear strength, in pounds/square inch
S_W	Gutter cross slope, in feet/foot
S_x	Pavement cross slope, in feet/foot
SCS	Formerly Soil Conservation Service
SWMM	Storm Water Management Model
T	Top width of water surface, (ft) -3
T	Width of flow or spread, in feet -4
T	Soil loss tolerance, in tons/acre/year -9
T_i	Duration of basin inflow
t_b	Time base, in minutes
t_c	Time of concentration
t_i	Travel time for the i^{th} segment, in minutes
t_l	Lag time, in minutes
t_p	Time to peak, in minutes
t_T	Travel time
t_1	Sheet flow travel time, in minutes
t_2	Shallow concentrated flow (typically rill or gutter flow) travel time, in minutes
t_3	Channelized flow time, in minutes
TCP	Temporary Construction Site Runoff Management Practices
TDEC	Tennessee Department of Environment and Conservation
TDOT	Tennessee Department of Transportation
TP-40	
TR-20	
TR-55	
TW	Tailwater depth, in feet
USBR	U.S. Bureau of Reclamations
USDOT	U.S. Department of Transportation
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
v	Velocity of a small kinematic wave, in feet/second
v_i	Average flow velocity for segment i, in feet/second
v_o	Outlet mean velocity, in feet/second
V	Volume under the hydrograph, in inches -2
V	Mean cross sectional velocity, (ft/s) -3
V_m	Maximum velocity
V_s	Storage volume estimate -8
V_s	Scourhole Volume -9
V_t	Total storm water volume discharged during designated period, in cubic feet
v_1	Velocity above jump, in feet/second
W	Minimum basin width -9
W	Stone weight, in pounds -3
W	Width of gutter, in inches -4
W_B	Basin width
W_s	Scourhole width
WinTR-20	Windows version of TR-20, 2005
WinTR-55	Windows version of TR-55, 2005

WSPRO	Water Surface Profile
X	Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume
X_T	Design storm frequency factor for the return period T
y_1	Incoming depth
y_2	Jump height
y_c	Critical depth, in feet
y_e	Equivalent brink depth
y_n	Normal depth, in feet
z_1	Basin elevation
Z	Critical flow section factor
Z_C	Reciprocal of composite pavement cross slope ($1/S_c$), in feet/foot
Z_W	Reciprocal of gutter cross slope ($1/S_W$), in feet/foot
Z_x	Reciprocal of pavement cross slope ($1/S_x$), in feet/foot
α	Energy coefficient
$\sum q_i$	Sum of the runoff hydrograph ordinates, in cfs, for each time increment I
Δd	Super elevation of the water surface profile due to the bend, in feet
ΔD	Incremental duration of runoff producing rainfall, in minutes
ΔE	Change in specific energy, in feet
Δt	Time increment of the runoff hydrograph ordinates, in seconds
ΔL	Channel routing segment length, in feet
ΔL_{min}	Minimum channel length for routing calculations, in feet
Δx	Length of channel between consecutive depths of flow, in feet
ΔY	A small increase in the representative depth of flow in the channel
θ	Angle of v-notch, in degrees
τ_c	Critical tractive shear stress, in pounds/square inch
γ_s	Specific weight of stone, in pounds/cubic foot

1.0 INTRODUCTION

1.1 Purpose

In April 1987, the City of Memphis Division of Engineering adopted the Storm Water Drainage Design Manual. The manual was revised in 1990 to improve technical correctness, calibrate the hydrology section using local runoff data, and make the manual more practical and cost effective. This revised manual was adopted by other local municipalities as a technical guidance for storm water compliance. After 25 years, the City of Memphis Division of Engineering and Shelby County's Engineering Department decided to update the manual once again to incorporate new technologies, updated resource data, and to improve the manual's functionality.

This volume of the Memphis and Shelby County Storm Water Management Manual has been prepared by the City of Memphis and Shelby County Engineering Departments to establish technical guidelines to enforce the departments' accepted drainage design policies. It provides a compilation of readily available literature relevant to storm water management activities in Memphis and Shelby 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 Volume 1 — Policies and includes references to and from Volume 3 — Best Management Practices.

1.2 Format and Contents

The Memphis and Shelby County Storm Water Management Manual is presented in three volumes:

- Volume 1 — Policy Manual
- Volume 2 — Drainage Manual
- Volume 3 — 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

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. Terms and symbols are defined immediately at the beginning of each chapter and after each equation. Example problems are provided at the end of selected sections to demonstrate the application of procedures. 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. Computer applications require appropriate user documentation and experience and should be considered for the following types of storm water 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. Detention routing and volume control calculations
5. 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 Memphis and Shelby County Storm Water Management Manual will be updated and revised, as necessary, to reflect up-to-date engineering practices and information applicable to the design of drainage-related structures in Memphis and Shelby County. Manual users are advised to review the procedures shown herein periodically to ensure that they are using the most current version in their design processes.

Questions and comments on projects in the City of Memphis and its reserve areas should be submitted to:

City of Memphis, Public Works Division
Attn: Stormwater Management
125 N. Main Street, Room 644
Memphis, Tennessee 38103

Phone: (901) 576-6700
Fax: (901) 576-6960
Email: pubworks@memphistn.gov

Questions and comments on projects in Shelby County should be submitted to:

Shelby County Engineer's Office
Attn: Storm Water Management
160 N. Main Street, Suite 350
Memphis, Tennessee 38103

Phone: (901) 545-4320
Fax: (901) 545-3963
Email: michael.oakes@shelbycountytn.gov

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Acronym List
(Chapter 2)

ΔD	Incremental duration of runoff producing rainfall, (min)
Δt	Time increment of the runoff hydrograph ordinates, (sec)
ΔL	Channel routing segment length, (ft)
ΔL_{min}	Minimum channel length for routing calculations, (ft)
ΔY	A small increase in the representative depth of flow in the channel
$\sum q_i$	Sum of the runoff hydrograph ordinates, (cfs), for each time increment i
A	Watershed drainage area, in acres, tributary to the design point
A_i	Land area contained in sub-area i with uniform land use conditions, (ac or mi^2)
A_m	Drainage area, (mi^2)
A_T	Total area of watershed, (ac or mi^2)
AMC II	Antecedent Moisture Condition II
ARI	Average recurrence interval
B	Hydrograph shape factor
B	Top width of water surface, (ft)
C	Weighted composite runoff coefficient
C_i	Runoff coefficient for sub-area i
C_T	Runoff coefficient for return period T, dimensionless
C_{10}	Runoff coefficient for a design storm return period of 10 years or less
cfs	Cubic feet per second
CN	NRCS Watershed Curve Number
CN_i	Curve number for sub-area i with a given combination of uniform hydrologic group and land use conditions
DA	Drainage Area
DDF	Depth-Duration-Frequency
FHWA	Federal Highway Administration
F_p	Pond and swamp adjustment factor
HEC-HMS	Hydrologic Engineering Center — Hydrologic Modeling System
HSG	Hydrologic Soil Groups
I	Rainfall intensity, (in/hr)
I_a	Initial Abstraction
I_{tc}	Average rainfall intensity, (in/hr), during a period of time equal to t_c or the return period T
I_1	Inflow rate at the beginning of routing time period Δt
I_2	Inflow rate at the end of routing time period Δt
IDF	Intensity-Duration-Frequency
K	Muskingum channel routing time constant for a particular channel segment

L	Sheet flow length, (ft)
L_i	Length of the flow path for segment i, (ft)
LoD	Loring Silt Loam
MeB	Memphis Silt Loam
n	Manning's roughness coefficient for sheet flow
n	Total number of areas with uniform runoff coefficients
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
O_1	Outflow rate at the beginning of routing time period Δt
O_2	Outflow rate at the end of routing time period Δt
P	Precipitation
P_T	Precipitation depth for return period T, (in)
P_2	2-year, 24-hour rainfall, (in)
q_u	Unit peak discharge, (cfs/mi^2/in)
q_p	Peak flow rate, (cfs)
Q	Flow rate, (cfs)
Q_T	Peak runoff rate for return period T, (cfs)
Q(Y)	A representative flow rate for channel routing at representative depth Y, (cfs)
$Q(Y+\Delta Y)$	Flow rate at the new depth $Y + \Delta Y$, (cfs)
R_T	Rainfall excess for return period T, (in)
s	Average slope of sheet flow path, (ft/ft)
S	Maximum soil storage, (in)
S_o	Slope of channel bottom, (ft/ft)
SCS	Soil Conservation Service
SWMM	Storm Water Management Model
t_b	Time base, (min)
t_c	Time of Concentration, (min)
t_i	Travel time for the ith segment, (min)
t_l	Lag time, (min)
t_p	Time to peak, (min)
t_1	Sheet flow travel time, (min)
t_2	Shallow concentrated flow (typically rill or gutter flow) travel time, (min)
t_3	Channelized flow time, (min)
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
v	Velocity of a small kinematic wave, (ft/sec)
v_i	Average flow velocity for segment i, (ft/sec)
V	Volume under the hydrograph, (in)

WinTR-20	Windows version of TR-20, 2005
WinTR-55	Windows version of TR-55, 2005
X	Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume
X_T	Design storm frequency factor for the return period T

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2.0 HYDROLOGY

2.1 Synopsis

Hydrology is generally defined as the science dealing with the interrelationship between water on and under the earth and in the atmosphere. For this manual, the term hydrology will refer to the estimation of storm water runoff discharge rates and/or volumes from precipitation for the design of drainage systems associated with urban and rural development in the City of Memphis and Shelby County. Hydrologic studies are required to develop appropriate input data for hydraulic calculations to evaluate the impact of land development on existing systems. Current conditions must be compared to predictions for post-construction conditions to assess the impact of proposed construction. This chapter describes techniques recommended for use in the City of Memphis and Shelby County for estimating peak discharges for designated rain events and flood hydrographs resulting from said events. This chapter is not intended to be an all-encompassing explanation of hydrologic theory, but a guide for implementing accepted methods for hydrologic analyses selected for their relative accuracy, ease of use, and flexibility. There are other acceptable methods available to designers not outlined in this chapter. Alternative methods of hydrologic analysis may only be used with the approval of the city and/or county engineer. 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, soil characteristics, and ground cover), select an appropriate hydrologic procedure from Section 2.2.

2. Identify rainfall data requirements for appropriate design-storm conditions from Section 2.3. If required for hydrograph generation, develop a rainfall hyetograph for the design-storm event using the method described in Section 2.3.

3. Estimate rainfall excess using Rational Method runoff coefficients or Natural Resource Conservation Service (NRCS — formerly Soil Conservation Service) curve numbers as outlined in Section 2.4.

4. Compute the watershed time of concentration using the procedures in Section 2.5.

5. Compute the peak runoff rate using methods described in Section 2.6, as appropriate for the procedure selected in Step 1. If required, generate a complete flood (runoff) hydrograph using one of the methods from Section 2.7.

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 2.8.

2.2 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/runoff hydrograph is usually required.

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

The Rational Method (see Section 2.6.1) 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 $\leq$ 10 acres

The Rational Method is meant for use as a tool for preliminary estimations of hydrologic conditions for small sites. Beyond the above limits, results shall be compared using other methods, and use of Rational Method estimations requires approval by the city and/or county engineer.

The **NRCS TR-55 (1986) graphical method** (see Section 2.6.2) is subject to the following limitations:

1. Estimates of peak design flows only.
2. Design storm = NRCS 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, NRCS, 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 $\leq I_a/P \leq$ 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 most current version of TR-55.
10. No consideration of hydrologic channel routing.

The **NRCS TR-55 (1986) tabular method** (see Section 2.7.3) can be used to estimate flood hydrographs and to approximate the effects of hydrologic channel routing, subject to the following limitations:

1. Design storm = NRCS 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 latest version of TR-55 procedures (1986 revision).

Unit hydrograph theory (see Section 2.7.1) provides a generally applicable procedure for developing flood/runoff 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.

Computer modeling (see Section 2.7.4) is appropriate when limitations of simpler methods are exceeded, complex situations are being studied, or more detailed information is required. In many cases, the use of computer modeling can be faster and more accurate than hand computation methods. The following computer programs are acceptable tools for use in development of flood/runoff hydrographs:

HEC-HMS developed by the U.S. Army Corps of Engineers (2005) (http://www.hec.usace.army.mil/software/hec-hms/)

WinTR-55 and WinTR-20 developed by the Natural Resources Conservation Service (2002) (http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models.html)

SWMM-RUNOFF developed by the U.S. Environmental Protection Agency (Huber et al., 1992; Roesner et al., 1994) (http://www.epa.gov/ednnrmrl/models/swmm/)

There are many other computer software packages available that may be used to develop flood/runoff hydrographs. Use of modeling software must be approved by the city and/or county engineer.

2.3 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 the City of Memphis and Shelby County include intensity-duration-frequency (IDF) curves and tabular data as well as depth-duration-frequency (DDF) curves and tabular data, which are required for predicting peak discharge rates and for developing runoff hydrographs.

2.3.1 Intensity-Duration-Frequency and Depth-Duration-Frequency Relationships

The IDF curves, DDF curves, and associated tables presented for use in the City of Memphis and Shelby County are based on data published in the "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 2, Version 2 (G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M. Yekta, and D. Riley) and made available by the Hydrometeorological Design Studies Center, part of the National Weather Service's Office of Hydrologic Development, Hydrology Laboratory (http://www.weather.gov/oh/hdsc/). The IDF and DDF curves and associated tabular data used in this chapter were taken directly from NOAA Atlas 14 data gathered at the Memphis NOAA observation station (NOAA station: Memphis WSCMO AP, Tennessee [40-5954] 35.0564 N 89.9864 W 249 feet). These IDF and DDF curves and associated tables replace previous data published in NOAA Technical Paper 40 and referenced in previous versions of the City of Memphis Drainage Design Manual (1990). IDF curves for durations up to 60 minutes and tabular data for up to 24 are presented in Figure 2-1 for return periods of 2, 5, 10, 25, 50, 100, 200, 500, and 1,000 years. DDF curves and corresponding tabular data for durations up to 24 hours are included on Figure 2-2. The rainfall intensities and depths shown in Figures 2-1 and 2-2 are representative for any singlepoint in Shelby County; however, as the drainage area increases, the intensity of precipitation should be reduced as recommended by the National Weather Service. Areal reduction curves from TP-40 (Hershfield, 1961), which are appropriate for use with all recurrence intervals, are shown in Figure 2-3.

2.3.2 Rainfall Hyetographs

The rainfall data presented in Section 2.3.1 identify average depth or intensity over specific durations. To develop a flood hydrograph, however, a time variable distribution (hyetograph) is required.

Hyetographs for the City of Memphis and Shelby County for a 24-hour storm duration are developed using the NRCS Type II 24 Hour Storm Distribution. A dimensionless hyetograph for a 24-hour storm is shown in Figure 2-4. Tabular data for the dimensionless hyetograph along with the 2-, 5-, 10-, 25-, 50-, 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 2-2) for the return frequency in question (see Example 2-1).

If smaller time intervals than are presented in the tabular hyetographs in Table 2-2 are required, additional data points may be obtained from the dimensionless hyetograph curve in Figure 2-4 or interpolated directly from the tabular data.

2.3.3 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 2-2, the 5-year, 24-hour rainfall depth is 4.89 inches

2. From Table 2-2, for a time of 1 hour, the P_x/P_{24} ratio is 0.0110

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

   $P_{1\ hour}$ = 4.89 inches x 0.0110

   $P_{1\ hour}$ = 0.054 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 (hr)	P_x/P_{24} Ratio	Hyetograph Ordinates (in)
0.00	0.0000	0.000
1.00	0.0110	0.054
2.00	0.0230	0.112
3.00	0.0350	0.171
4.00	0.0480	0.235
5.00	0.0640	0.313
6.00	0.0800	0.391
7.00	0.1000	0.489
8.00	0.1200	0.587

Time (hr)	P_x / P_{24} Ratio	Hyetograph Ordinates (in)
9.00	0.1470	0.719
10.00	0.1810	0.885
11.00	0.2360	1.154
12.00	0.6629	3.242
13.00	0.7760	3.795
14.00	0.8250	4.034
15.00	0.8560	4.186
16.00	0.8810	4.308
17.00	0.9030	4.416
18.00	0.9220	4.509
19.00	0.9380	4.587
20.00	0.9530	4.660
21.00	0.9650	4.719
22.00	0.9770	4.778
23.00	0.9890	4.836
24.00	1.0000	4.890

2.4 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 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. In general, the sum of initial abstractions can be expressed mathematically as shown below in Equation 2-1. Table 2-3 contains values of initial abstraction for various NRCS curve numbers. Rainfall excess is expressed mathematically by use of Equation 2-2, which is a function of precipitation for a defined return period and maximum soil storage for watershed soil conditions described by an NRCS curve number (CN). Maximum soil storage is expressed mathematically using Equation 2-3.

Initial Abstraction

$$I_a = 0.2S \qquad (2\text{-}1)$$

Rainfall Excess

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

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

Where:

R_T = Rainfall excess for return period T, in inches

P_T = Precipitation depth for return period T, in inches

S = Maximum soil storage, in inches

CN = NRCS watershed curve number

I_a = Initial abstraction

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

NRCS 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.

2.4.1 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 2-4. Runoff coefficients for various land uses, soil types, and watershed slopes in Table 2-4 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 composite C value shall be calculated as follows:

$$\overline{C} = \frac{\sum_{i=1}^{n} C_i A_i}{A_T} \qquad (2\text{-}4)$$

Where:

C = Weighted composite runoff coefficient

N = Total number of areas with uniform runoff coefficients

C_i = Runoff coefficient for subarea i from Table 2-4

A_i = and 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 runoff coefficients from Table 2-4 shall be multiplied by the frequency factors from Table 2-5. The following relationship is used to combine the data presented in Tables 2-4 and 2-5:

$$C_T = C_{10} X_T \qquad (2\text{-}5)$$

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 2-4)
X_T = Design storm frequency factor for the return period T (Table 2-5)

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

2.4.2 NRCS Curve Numbers

The procedure for determining the NRCS curve number uses soil survey information published by the NRCS. Selection of an appropriate NRCS 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 NRCS soil survey report (1970) for Shelby County. Soil classifications present in Shelby County are presented in Table 2-6. Soil survey information can be found online at: http://websoilsurvey.nrcs.usda.gov/app/

2. Assign a hydrologic group to each soil type. Hydrologic soil groups (HSG) are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms. The soils in the United States are placed into four groups A, B, C, and D, and three dual classes, A/D, B/D, and C/D. Hydrologic soil groups of the soils present in Shelby County are presented in Table 2-6. Definitions of the four hydrologic soil groups are:

   Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well-drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

   Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well-drained or well-drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a clay pan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only soils that are rated D in their natural condition are assigned to dual classes.

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 2-7, 2-8, and 2-9. Table 2-7 provides curve numbers for selected urban and suburban land uses; Table 2-8 gives information on agricultural land uses; Table 2-9 provides curve numbers for 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 amount of barren pervious area (with little sod established). Any one of these factors could move a soil normally placed in hydrologic group A or B to group B or C, respectively.

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

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

Where:

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 2-7, 2-8, and 2-9)

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

2.4.3 Example Problems

Example 2-2. Rational Method Weighted Composite Runoff Coefficient

A 10-acre watershed with an average overland and shallow channel slope of 2.76 percent and fair/good ground cover on both MeB (Memphis Silt Loam) and LoD (Loring Silt Loam) soils is to be developed.

Pre-development conditions are as follows:

1. 4.0 acres woodland in good condition on MeB soil (HSG B)
2. 2.0 acres pasture in fair condition on MeB soil (HSG B)
3. 4.0 acres pasture in fair condition on LoD soil (HSG C)

Compute a weighted composite runoff coefficient for pre-development conditions when considering a 25-year, 24-hour design storm.

1) The composite weighted runoff coefficient is computed from Equation 2-4 as follows:

   a) For sub-area 1: From Table 2-4, for 2-7% sloped wooded areas on MeB soil (moderate soil — midway between sandy soils and heavy soils), use C_1 = 0.16

b) For sub-area 2: From Table 2-4, for 2-7% sloped unimproved areas on MeB soil (moderate soil), use $C_2 = 0.21$

c) For sub-area 3: From Table 2-4, for 2-7% sloped unimproved areas on LoD soil (heavy soil), use $C_3 = 0.25$

Sub Area (i)	Area in Acres (A_i)	Runoff Coefficient (C_i)	Runoff Coefficient x Area (C_iA_i)
1	4.0	0.16	0.64
2	2.0	0.21	0.42
3	4.0	0.25	1.00
Total	10.0		2.06

2) Using Equation 2-4, the weighted composite runoff coefficient is:

$$\overline{C} = \frac{2.06}{10} = 0.21$$

3) Using Equation 2-5 (repeated below) and Table 2-5, the weighted composite runoff coefficient for a 25 year return period is:

$$C_{25} = 0.21(1.1) = 0.23$$

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

A 35-acre watershed with an average overland and shallow channel slope of 2.78 percent and fair/good ground cover on both MeB (Memphis Silt Loam) and LoD (Loring Silt Loam) soils is to be developed. Pre-development conditions are as follows:

1. 10.4 acres woodland in good condition on MeB soil (HSG B)
2. 14.6 acres pasture in fair condition on MeB soil (HSG B)
3. 10.0 acres pasture in fair condition on LoD soil (HSG C)

Calculate the rainfall excess for pre-development conditions from a 10-year, 24-hour storm using the NRCS curve number.

1. From Figure 2-2, the 10-year, 24-hour rainfall depth is 5.58 inches.

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

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

a. For sub-area 1: From Table 2-9, woods in good condition, HSG B; use CN = 55
b. For sub-area 2: From Table 2-9, pasture in fair condition, HSG B; use CN = 69
c. For sub-area 3: From Table 2-9, pasture in fair condition, HSG C; use CN = 79

Sub Area (i)	Area in Acres (A_i)	NRCS Curve Number (CN_i)	NRCS Curve Number x Area ($CN_i x A_i$)
1	10.4	55	572.00
2	14.6	69	1,007.40
3	10.0	79	790.00
Total	**35.0**		**2,369.40**

Using Equation 2-6: $\overline{CN} = \frac{2369.40}{35} \approx 68$

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

$$S = \frac{1{,}000}{68} - 10 = 4.71\, inches$$

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

$$R_{10(scs)} = \frac{[5.58 - 0.2(4.71)]^2}{5.58 + 0.8(4.71)}$$

$$R_{10(scs)} = \frac{(4.64)^2}{9.35}$$

$$R_{10(scs)} = 2.30\, inches$$

2.5 Time of Concentration

Time of concentration is a fundamental watershed parameter. Time of concentration is defined as the longest time required for a drop of water to travel from the watershed divide to the watershed outlet. Caution should be used when determining the path along which the time of concentration is determined. This path should be representative of the drainage area as a whole. On some irregularly shaped drainage areas, it is possible to find the time of concentration along a particular path, which is representative of only a small portion of the drainage area. The time of concentration is used to compute the peak discharge for a watershed using the rational method. The peak discharge is a function of the rainfall intensity, which is based on the time of concentration. Time of concentration is also used in determining time to peak for the NRCS unit hydrograph method, and can be used to determine lag time as well (see Section 2.7.1).

To calculate the time of concentration of a watershed, at least three runoff components should be considered: sheet flow, shallow concentrated flow (typically rill or gutter), and channelized flow. Time of concentration may also include a time component for pipe flow in post-development conditions. Whatever the combination of flow types in the drainage basin, the time for each portion is identified as an individual travel time (t_i). Summing up the travel times for each flow path segment will yield the time of concentration. As a general guideline, if the time of concentration is calculated to be less than 5 minutes, a minimum value of 5 minutes shall be used.

The time of concentration is expressed as:

$$t_c = t_1 + t_2 + t_3 + \ldots + t_i \qquad (2\text{-}7)$$

Where:

- t_c = Time of concentration, in minutes
- t_1 = Sheet flow travel time, in minutes
- t_2 = Shallow concentrated flow (typically rill or gutter flow) travel time, in minutes
- t_3 = Channelized flow time, in minutes
- t_i = Travel time for the i^{th} segment, in minutes

2.5.1 Sheet Flow

Sheet flow is flow over plane surfaces. Friction in sheet flow, usually described by Manning's roughness coefficient, n, is usually comprised of drag over the plane surface, which may include obstacles such as litter, vegetation, crop ridges, sediment, and rocks. Sheet flow is normally considered when water flows at a depth of 0.1 feet (1.2 inches) or less. It is not always apparent when flow changes from sheet flow to shallow concentrated flow. If shallow ridges, swales, or small channels are not evident in the field, it is reasonable to assume the water stays in sheet flow for a maximum of 300 feet, though it is more likely that a length closer to 100 feet should be used in overland flow computations for unpaved areas. After 300 feet, it is presumed that the water will find shallow rills, ridges, or swales on the ground's surface in which to collect. Paved areas may have longer lengths of sheet flow until flow becomes channelized in gutters or low areas of parking lots. Once to this point, the water is termed shallow concentrated flow. The time of travel for sheet flow will be, for the purposes of this manual, approximated by utilizing the NCRS Runoff Method and the Kinematic Wave Method. In most cases, there are numerous other acceptable methods for determining sheet flow time of travel, the use of which requires approval by the City of Memphis and/or Shelby County engineer.

The NCRS Runoff Method uses a non-iterative approximation to the sheet flow travel time for flow paths of less than 300 feet. Equation 2-8 employs Manning's kinematic solution (Overton and Meadows, 1976) based on a single constant rainfall excess from a 2-year, 24-hour rainfall event, shallow steady uniform flow, and minor effect from infiltration on travel time. For many cases, this approximate method will yield acceptable results; however, sheet flow travel time shall be checked using the iterative Kinematic Wave Method (Equation 2-9) with results of the NCRS Runoff Method as a starting point. The NRCS Runoff method is expressed mathematically as:

$$t_1 = 0.42\left(\frac{(nL)^{0.8}}{P_2^{\ 0.5} s^{0.4}}\right) \qquad (2\text{-}8)$$

Where:

- t_1 = Sheet flow travel time, in minutes
- L = Sheet flow length, in feet
- n = Manning's roughness coefficient for sheet flow (see Table 2-10)
- P_2 = 2-year, 24-hour rainfall, in inches (see Figure 2-2)
- s = Average slope of sheet flow path, in feet/foot

Manning's n values reported in Table 2-10 were determined specifically for sheet flow conditions and are not appropriate for conventional open-channel flow calculations.

The kinematic wave formula (Equation 2-9) (Morgali and Linsley, 1965; Aron and Erborge, 1973) is an overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces. Use of the Kinematic Wave Method requires an iterative approach since both I (rainfall intensity) and t_1 (sheet flow travel time) are unknown. Using time of concentration from the NCRS Runoff Method (Equation 2-8), an intensity, I, for use in the kinematic wave formula can be obtained using Figure 2-1 for the appropriate design storm average recurrence interval (ARI). For the City of Memphis and Shelby County, time of concentration shall be calculated based upon parameters dictated by use of a rain event with an ARI of 10 years. Design storm ARI is dictated by design standards set forth in Volume 1: Regulations. Figure 2-5 presents a nomograph that can be used to solve the kinematic wave formula graphically, which is expressed mathematically as:

$$t_1 = 0.94\left(\frac{L^{0.6}n^{0.6}}{I^{0.4}s^{0.3}}\right) \qquad (2\text{-}9)$$

Where:

t_1	=	Sheet flow travel time, in minutes
L	=	Sheet flow length, in feet
n	=	Manning's roughness coefficient for sheet flow (see Table 2-10)
I	=	Rainfall intensity, in inches/hour (Use ARI of 10 years in Figure 2-1)
s	=	Average slope of sheet flow path, in feet/foot

Manning's n values reported in Table 2-10 were determined specifically for sheet flow conditions and are not appropriate for conventional open-channel flow calculations.

Computation of sheet-flow travel time entails a trial-and-error process using the following steps:

1. Use the NRCS runoff (Equation 2-8) to solve for sheet-flow travel time, t_1, based on given basin conditions.

2. Apply t_1 as calculated in Step 1 as well as other runoff components t_2, t_3, t_i (see Sections 2.4.2 and 2.4.3) to Equation 2-7, solving for an initial t_c estimate.

3. Choose an estimated intensity, I, from Figure 2-1 using the appropriate design storm ARI (10 years for Memphis and Shelby County) and duration t_c from Step 2.

4. Using intensity, I, from Step 3, find the sheet-flow travel time, t_1, by solving Equation 2-9 or using Figure 2-5.

5. Apply t_1 as calculated in Step 4 as well as other runoff components t_2, t_3, t_i (see Sections 2.4.2 and 2.4.3) to Equation 2-7, solving for an initial t_c estimate.

6. Choose an estimated intensity, I, from Figure 2-1 using the appropriate design storm ARI and duration t_c from Step 5.

7. Repeat Steps 4 — 6 using computed intensity, I, from Step 6 in Step 4 to calculate the next estimate of t_1. This process is repeated until consecutive estimates of time of concentration, t_c, have converged sufficiently.

2.5.2 Shallow Concentrated Flow

After sheet flow, the water usually becomes shallow concentrated flow. Shallow concentrated flow normally has a depth greater than 0.1 feet (1.2 inches). Average velocities for shallow channel flow in rills (applicable to swales) and gutters can be obtained directly from Figure 2-6, if the slope of the flow segment is known. Knowing the flow path length and average flow velocity, the travel time is estimated using Equation 2-10. 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. Travel time, t_i, can be approximated using the following expression:

$$t_i = \frac{L_i}{(60)v_i} \qquad (2\text{-}10)$$

Where:

t = 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

2.5.3 Channelized Flow

After some distance, shallow concentrated flow further concentrates to a pipe, ditch, or channel, which is termed channelized flow. Travel time for pipe, ditch, or channel flow can be estimated from the hydraulic properties of the conduit or channel. After first determining the average velocity in the pipe or channel, the travel time is obtained by dividing the pipe or channel length by the determined velocity, as expressed in Equation 2-10. In watersheds with storm drains, pipes, or channels, the travel time for these items must be added to sheet flow travel time and shallow concentrated flow travel time to find the total time of concentration. Manning's Equation can be used to determine average water velocity for these types of flow (see Chapter 3). When calculating the velocity for channels, the designer should presume bank-full conditions. For an engineered channel that has been well maintained or is in the process of design, the designer may assume one foot of freeboard. In special situations, additional design information may be available and a different depth of flow may be used. More than one channelized flow segment shall be used where needed to account for varying channel slope, roughness, or cross section within individual channels.

2.5.4 Time of Concentration Example Problem

Example 2-4. Time of Concentration Computation

The hydrologic flow path of the watershed described in Example 2-3 is about 1,825 feet in length with a total elevation change of about 50 feet, which results in an average watershed slope of 2.74%. Though an average watershed slope is useful for determining runoff coefficients or NRCS curve numbers, it is required that the selected flow path be divided into individual segments with lengths and elevation changes describing each segment. This flow path may be divided into the following three segments:

Segment No.	Type of Flow	Segment Length (ft)	Elevation Change (ft)	Slope (%)
1	Sheet Flow (woodland, n = 0.45)	125	4.4	3.50
2	Shallow Flow (bare soil rills)	600	21.8	3.63
3	Main Channel (bare soil, n = 0.020)	1,100	23.8	2.16

Roughly rectangular main channel conditions:

n = 0.020 (Chapter 3, Table 3-3), width = 12 feet, depth = 1.33 feet

Compute watershed time of concentration for the standard design storm for the City of Memphis and Shelby County: the 10-year, 24-hour storm.

1. Compute the sheet flow travel time, t_1, using the NCRS Runoff Method from Equation 2-8 (repeated below).

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

From Table 2-10, for woodlands, n = 0.45

Sheet flow length, L, is 125 feet

From Figure 2-2, the 2-year, 24-hour rainfall, P_2, is 4.01 inches

The average slope, s, of sheet flow path is 0.0547 feet per foot

$$t_l = 0.42\left(\frac{(0.45 \times 125)^{0.8}}{(4.01)^{0.5}(0.035)^{0.4}}\right)$$

t_1 = 20.1 minutes

2. Compute the shallow channel flow travel time using the rill flow curve in Figure 2-6.

From Figure 2-6, for a slope of 3.63 percent, the flow velocity is about 3.1 feet/second

Using Equation 2-10,

$$t_2 = \frac{600}{(60)(3.1)}$$

t_2 = 3.2 minutes

3. Compute the main channel flow travel time.

The flow velocity is given by Manning's Equation (see Chapter 3),

$$v = \frac{1.49}{n} R^{2/3} s^{1/2}$$

For a rectangular channel with a 12-foot bottom width and an estimated depth of 2 feet, calculate hydraulic radius, R (see Chapter 3) and use Table 3-3 (see Chapter 3), to choose an appropriate Manning’s friction factor, n, to solve Manning’s Equation:

$$R = \frac{A}{P} = \frac{(12 \times 1.33)}{[2(1.33) + 12]}$$

R = 24/16 = 1.09 feet

$$v = \frac{1.49}{0.020} 1.09^{2/3} 0.0216^{1/2}$$

v = 11.6 feet/second

By Equation 2-10,

$$t_3 = \frac{1100}{60(11.60)}$$

t_3 = 1.6 minutes

4. Compute the watershed time of concentration,

From Equation 2-7,

t_c = 20.1 + 3.2 + 1.6

t_c = 24.9 minutes

From Figure 2-1, for duration = t_c = 24.9 minutes, the 10-year ARI rainfall intensity is 4.05 inches/hour.

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

$$t_1 = 0.94\left(\frac{L^{0.6}n^{0.6}}{I^{0.4}S^{0.3}}\right)$$

From Step 1, n = 0.45, from Step 4, the rainfall intensity is 4.05 inches/hour.

$$t_1 = 0.94\left(\frac{125^{0.6}0.45^{0.6}}{4.05^{0.4}0.035^{0.3}}\right)$$

t_1 = 16.5 minutes

From Step 2, t_2 = 3.2 minutes. From Step 3, t_3 = 1.6 minutes. From Equation 2-7:

t_c = 16.5 + 3.2 + 1.6
t_c = 21.3 minutes

From Figure 2-1, for duration = t_c = 21.3 minutes, the 25-year return frequency rainfall intensity is 4.38 inches/hour

Computed rainfall intensity (I=4.38 in/hr) and rainfall intensity from Step 4 (I=4.05 in/hr) do not agree

6. Repeat computation assuming rainfall intensity is 4.38 inches/hour from Step 5.

$$t_1 = 0.94\left(\frac{125^{0.6}0.45^{0.6}}{4.38^{0.4}0.035^{0.3}}\right)$$

t_1 = 16.0 minutes

From Step 2, t_2 = 3.2 minutes. From Step 3, t_3 = 1.6 minutes. From Equation 2-7:

t_c = 16.0 + 3.2 + 1.6
t_c = 20.8 minutes

From Figure 2-1, for duration = t_c = 20.8 minutes, rainfall intensity is 4.42 inches/hour

Trial rainfall intensity from Step 5 (I=4.38 in/hr) and computed rainfall intensity (I=4.42 in/hr) are close, but to be safe, complete one more trial

7. Repeat computation assuming rainfall intensity is 4.42 inches/hour from Step 6.

$$t_1 = 0.94\left(\frac{125^{0.6}0.45^{0.6}}{4.42^{0.4}0.035^{0.3}}\right)$$

t_1 = 15.9 minutes

From Step 2, t_2 = 3.2 minutes. From Step 3, t_3 = 1.6 minutes. From Equation 2-7:

t_c = 15.9 + 3.2 + 1.6
t_c = 20.7 minutes

From Figure 2-1, for duration = t_c = 20.7 minutes, rainfall intensity is 4.43 inches/hour

Trial rainfall intensity from Step 6 and computed rainfall intensity are close enough

8. Use t_c = 20.7 minutes and I = 4.43.

Note: Some cases may require further iterations to achieve sufficient convergence.

2.6 Peak Runoff Rates

2.6.1 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 (see Section 2.4.1)
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 2.5), 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 2.5

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 2.4.1

5. Compute the peak runoff rate for the return period T, using Equation 2-11

2.6.2 SCS (NRCS) TR-55 Graphic Method

The NRCS 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, NRCS, 1983) and is subject to certain limitations. A description of the NRCS procedure and details on limitations is contained in NRCS TR-55 (1986).

The graphical peak discharge method described in Chapter 4 of NRCS TR-55 (1986) is based on the following equation:

$$Q_t = q_u \, A_m \, R_T \, F_p \qquad (2\text{-}12)$$

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 = Rainfall Runoff/Excess, 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, P, is determined from Figure 2-2 for the selected return frequency.

2. The runoff curve number, CN, and total rainfall runoff/excess, R_T, are estimated using the procedures in Section 2.4.2.

3. The CN value is used to determine the initial abstraction, I_a, from Equations 2-1 and 2-3 or Table 2-3 and the ratio I_a/P is then computed.

4. The watershed time of concentration is computed using the procedures in Section 2.5 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 shall be used.

5. The pond and swamp adjustment factor, F_p, is estimated from below (TR-55, USDA, NRCS, 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 2-12.

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 shall 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 procedures should be used if the TR-55 Graphic Method is chosen for use in determining peak runoff rate. 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. TR-55 (1986) can also be found on the NRCS web site at: http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-other-models.html.

2.6.3 Other Techniques

Other methods may be used for computation of design flow rates, subject to the approval of the city and/or county engineer. The following computer models are recommended for complex hydrologic conditions:

HEC-HMS developed by the U.S. Army Corps of Engineers (2005)
(http://www.hec.usace.army.mil/software/hec-hms/)

WinTR-55 and WinTR-20 developed by the Natural Resources Conservation Service (2002)
(http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models.html)

SWMM-RUNOFF developed by the U.S. Environmental Protection Agency (Huber et al., 1992; Roesner et al., 1994) (http://www.epa.gov/ednnrmrl/models/swmm/)

2.6.4 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. Below is a summary of watershed conditions:

Segment No.	Type of Flow	Segment Length (ft)	Elevation Change (ft)	Slope (%)
1	Sheet Flow (woodland, n = 0.45)	150	5.0	3.33
2	Shallow Flow (bare soil rills)	200	8.0	4.00
3	Main Channel (grass lined, n = 0.05)	500	10.5	2.10

1. Total area of watershed = 10 acres

2. From Example 2-2, the 25-year runoff coefficient, C_{25}, is 0.23

3. Following procedures in Section 2.5, the watershed time of concentration, t_c, is 20 minutes

4. Using t_c = 20 minutes, the rainfall intensity for a 25-year storm is 4.96 inches per hour

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

Q_{25} = (0.23) (4.96) (10)
Q_{25} = 11.41 cfs

Example 2-6. NRCS TR-55 Graphical Method Peak Runoff Rate

Use the NRCS graphical peak discharge method to compute the peak runoff rate from a 10-year, 24-hour storm event from the watershed in Example 2-3. Use the watershed time of concentration computed in Example 2-4.

1. From Figure 2-2, the 10-year, 24-hour rainfall depth is 5.58 inches

2. From Example 2-3, the watershed area is 35 acres

3. From Example 2-3, the curve number, CN, is 68

4. From Example 2-3, Step 3, the soil storage, S, is 4.71 inches

5. From Example 2-3, Step 4, the rainfall excess, $R_{10\ (NRCS)}$, is 2.30 inches

6. From Table 2-3 or Equation 2-1, the initial abstraction, I_a, is 0.942 inches

7. The I_a/P ratio is:

   I_a/P = 0.942/5.58
   I_a/P = 0.17

8. From Figure 2-7, for the time of concentration, t_c, from Example 2-4 of 0.35 hour (20.7 minutes) and with an I_a/P ratio of 0.17, the unit peak discharge, q_u, is approximately 600 csm/inch of runoff.

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

10. The peak runoff rate is computed using Equation 2-12, as follows:

    Q_{25} = (600) (35/640) (2.30) (1.0)
    Q_{25} = 75.5 cfs

2.7 Flood Hydrographs

Flood hydrograph procedures presented include unit hydrograph theory, the rational hydrograph method, and the NRCS TR-55 (1986) tabular method.

2.7.1 Unit Hydrographs

Unit hydrographs should be developed using observed rainfall and stream-flow 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 NRCS dimensionless unit hydrograph approach is presented below.

Two types of dimensionless unit hydrographs were developed by the NRCS 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 NRCS curvilinear dimensionless unit hydrograph is as follows:

1. Estimate the time of concentration, t_c, using an appropriate method (see Section 2.5)

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

$$\Delta D = 0.133\ t_c \qquad (2\text{-}13)$$

Where:

ΔD = Incremental duration of runoff producing rainfall, in minutes
t_c = Time of concentration, in minutes

(Note that ΔD is often expressed in terms of NRCS lag time, t_l. Time of concentration and lag time are NOT one and the same; the relationship between the two can be expressed as: $t_l = 0.6\ t_c$. Applying this relationship to Equation 2-13 results in a common expression of incremental duration: $\Delta D = 0.222\ t_l$. It is important not to confuse time of concentration and lag time and their associated expressions of ΔD.

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

$$t_p = \frac{\Delta D}{2} + t_l = \frac{\Delta D}{2} + 0.6 t_c \qquad (2\text{-}14)$$

Where:

t_p	=	Time to peak, in minutes
ΔD	=	Incremental duration of runoff producing rainfall, in minutes
t_l	=	Lag time, in minutes
t_c	=	Time of concentration, in minutes

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

$$q_p = 60\ (BA)/t_p \qquad (2\text{-}15)$$

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 NRCS standard B value of 484 shall be used in the City of Memphis and Shelby County unless another value is approved by the city and/or county engineer.
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 NRCS 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 calculated as shown in Section 2.5, the incremental duration of runoff producing rainfall, ΔD, is computed using Equation 2-13, the time to peak, t_p, is computed using Equation 2-14, and the time base, t_b, is computed as follows:

$$t_b = 2.67\ t_p \qquad (2\text{-}16)$$

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 NRCS procedure.

2. Develop a design storm hyetograph using the time interval for which the unit hydrograph was developed (as presented in Section 2.3).

3. Develop a rainfall excess hyetograph using an appropriate procedure as presented in Section 2.4.

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:

$$V = \frac{12\Delta t \Sigma q_i}{A(43{,}560)} \qquad \text{(2-17)}$$

Where:

V	=	Volume under the hydrograph, in inches
Δt	=	Time increment of the runoff hydrograph ordinates, in seconds
Σq_i	=	Sum of the runoff hydrograph ordinates, in cfs, for each time increment i
A	=	Watershed drainage area, in acres

2.7.2 Rational Hydrograph Method

A rational hydrograph method may be developed for small homogeneous watersheds when attenuation is insignificant. A small, paved parking lot is one example where using this method may be appropriate. The Rational Method is meant for use as a tool for preliminary estimations of hydrologic conditions for small sites. Beyond the limits outlined in Section 2.2, results shall be compared using other methods, and use of Rational Method estimations requires approval by the city and/or county engineer.

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 2-9):

1. Determine appropriate design storm for facilities being evaluated

2. Estimate the runoff coefficient (Section 2.4.1)

3. Compute the watershed time of concentration (see Section 2.5)

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

2.7.3 NRCS TR-55 Tabular Method

The NRCS 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, NRCS, 1983) and is subject to certain limitations. A description of the NRCS procedure and details on limitations are contained in NRCS TR-55 (1986).

The 1986 version of TR-55 procedures should be used if the TR-55 Tabular Method is chosen for use in determining peak runoff rate. The 1986 version can be obtained from the National Technical Information Service in Springfield, Virginia. 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. TR-55 (1986) can also be found on the NRCS Web site at: http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-other-models.html.

2.7.4 Other Methods

Other methods of developing flood hydrographs may be used subject to approval by the city and/or county engineer. In addition, other methods may be used for computation of design flow rates, subject to the approval of the city and/or county engineer. Use of computer modeling software is perhaps the most popular and most efficient method for estimating peak flows, developing storm runoff hydrographs, and flood routing procedures. There are many acceptable computer models available, however, the following computer models are recommended for use in Memphis and Shelby County:

HEC-HMS developed by the U.S. Army Corps of Engineers (2005) (http://www.hec.usace.army.mil/software/hec-hms/)

WinTR-55 and WinTR-20 developed by the Natural Resources Conservation Service (2002) (http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models.html)

SWMM-RUNOFF developed by the U.S. Environmental Protection Agency (Huber et al., 1992; Roesner et al., 1994) (http://www.epa.gov/ednnrmrl/models/swmm/)

2.7.5 Hydrograph Example Problems

Example 2-7. NRCS Dimensionless Unit Hydrograph

Develop a synthetic unit hydrograph for the watershed in Example 2-3 using the NRCS curvilinear approach.

1. From Example 2-4, the watershed time of concentration, t_c, is 20.7 minutes.

2. From Equation 2-13, the incremental duration of runoff producing rainfall, ΔD, is

   ΔD = 0.133 (20.7)
   ΔD = 2.75 minutes or ≈ 3 minutes (0.05 hours)

3. From Equation 2-14, the time to peak, t_p, is:

$$t_p = \frac{3}{2} + 0.6(20.7)$$

   t_p = 13.9 minutes = 0.23 hours ≈ 0.25 hours (rounded to the nearest 0.05 (ΔD) hours)

4. From Equation 2-15, the unit hydrograph peak flow rate, q_p, is:

$$q_p = 60\left(\frac{484(35/640)}{15}\right) = 105.9\ cfs$$

5. From Table 2-11 or Figure 2-8, 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/0.25 hours)	Discharge Ratios q/q_p	Discharge, q (q/105.9 cfs)
0.00	0.0	0.000	0.0
0.05	0.2	0.100	10.6
0.10	0.4	0.310	32.8
0.15	0.6	0.660	69.9
0.20	0.8	0.930	98.5
0.25	1.0	1.000	105.9
0.30	1.2	0.930	98.5
0.35	1.4	0.780	82.6
0.40	1.6	0.560	59.3
0.45	1.8	0.390	41.3
0.50	2.0	0.280	29.7
0.55	2.2	0.207	21.9
0.60	2.4	0.147	15.6
0.65	2.6	0.107	11.3
0.70	2.8	0.077	8.2
0.75	3.0	0.055	5.8
0.80	3.2	0.040	4.2
0.85	3.4	0.029	3.1
0.90	3.6	0.021	2.2
0.95	3.8	0.015	1.6
1.00	4.0	0.011	1.2
1.05	4.2	0.008	0.9
1.10	4.4	0.006	0.6
1.15	4.6	0.004	0.4
1.20	4.8	0.002	0.2
1.25	5.0	0.000	0.0
		Sq_i (cfs) =	**704.1**

6. Check that the unit hydrograph volume equals 1 inch using Equation 2-17:

$$V = \frac{12(0.05)(3{,}600)(704.1)}{(35)(43{,}560)} = 0.998 \approx 1.0 \quad (\textit{close enough})$$

Example 2-8. Flood Hydrograph Using Unit Hydrograph Theory

Develop a synthetic runoff hydrograph for a 25-year, 1-hour design storm for the watershed described in Example 2-3 using the unit hydrograph developed in Example 2-7.

1. Develop a balanced storm hyetograph and cumulative mass curve using the IDF curve for a 25-year storm and the tabular data on 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	0.05	0.05
0.05	8.06	0.67	0.67	0.05	0.11
0.10	8.34	0.77	0.10	0.06	0.16
0.15	6.87	1.00	0.23	0.07	0.23
0.20	5.97	1.20	0.19	0.09	0.32
0.25	5.43	1.36	0.16	0.10	0.42
0.30	4.99	1.51	0.15	0.12	0.54
0.35	4.67	1.65	0.14	0.14	0.68
0.40	4.43	1.78	0.13	0.16	0.84
0.45	4.23	1.90	0.12	0.23	1.08
0.50	4.02	2.01	0.11	0.67	1.75
0.55	3.80	2.11	0.10	0.19	1.94
0.60	3.62	2.20	0.09	0.15	2.10
0.65	3.46	2.28	0.08	0.13	2.22
0.70	3.32	2.35	0.07	0.11	2.33
0.75	3.19	2.41	0.06	0.10	2.43
0.80	3.08	2.47	0.06	0.08	2.51
0.85	2.98	2.52	0.05	0.06	2.57
0.90	2.88	2.57	0.05	0.05	2.63
0.95	2.78	2.63	0.05	0.05	2.68
1.00	2.68	2.68	0.05	0.00	2.68

2. Develop a rainfall excess hyetograph using the NRCS curve number approach (Equation 2-2). From Example 2-3, CN is 68 and S is 4.71 inches. From Figure 2-2, the 25-year, 1-hour rainfall depth is 2.68 inches. Apply Equation 2-2 to each incremental cumulative depth to obtain a corresponding cumulative excess. The change in cumulative excess at each time increment makes up the ordinates for the rainfall excess hyetograph shown below:

Time, t (hours)	Cumulative Depth (inches)	Cumulative Excess (inches)	Excess Hyetograph (Incremental Excess) (inches)
0.00	0.05	0.00	0.000
0.05	0.11	0.00	0.000
0.10	0.16	0.00	0.000
0.15	0.23	0.00	0.000
0.20	0.32	0.00	0.000
0.25	0.42	0.00	0.000
0.30	0.54	0.00	0.000
0.35	0.68	0.00	0.000
0.40	0.84	0.00	0.000
0.45	1.08	0.00	0.004
0.50	1.75	0.12	0.114
0.55	1.94	0.17	0.057
0.60	2.10	0.23	0.052
0.65	2.22	0.27	0.047
0.70	2.33	0.32	0.044
0.75	2.43	0.36	0.041
0.80	2.51	0.39	0.034
0.85	2.57	0.42	0.028
0.90	2.63	0.44	0.024
0.95	2.68	0.47	0.024
1.00	2.68	**0.47**	0.000
		S Excess (inches) =	**0.47**

3. Route the rainfall excess hyetograph through the watershed using the unit hydrograph developed in Example 2-7. 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 first rainfall excess occurred. The rainfall excess hydrograph is obtained by summing the ordinates of each routed incremental hydrograph, as shown in Table 2-12.

4. Check that hydrograph volume is equal to the rainfall excess (0.47 in this case). Use $\sum q_i$ from the rainfall excess hydrograph on Table 2-12. Using Equation 2-17,

$$V = \frac{12(0.05)(3{,}600)(328.2)}{(35)(43{,}560)} = 0.46\ inches \approx 0.47\ inches \quad (close\ enough)$$

Note that routing of longer design storms requires considerably more calculation, as the number of increments, DD, and corresponding incremental excesses increase proportionately with increase in design-storm length. For example, development of a 25-year, 24-hour design-storm synthetic runoff hydrograph would have required approximately 24 times the number of lagged incremental excess flows as the 25-year, 1-hour flood hydrograph developed on Table 2-12. For this reason, it is usually much more efficient to develop flood hydrographs using computer models rather than hand calculations for all but the simplest design storms.

Example 2-9. Rational Hydrograph Method

Develop a runoff hydrograph for the watershed described in Examples 2-2 and 2-5 using the rational hydrograph method for a 25-year, 6-hour design storm.

1. Watershed characteristics:

   a. Area = 10 acres
   b. Time of concentration, $t_c \cong 20$ minutes
   c. Runoff coefficient, C = 0.23

2. Develop a balanced storm for time increments equal to the time of concentration using the procedure from Example 2-8, Step 1.

Time, t (hours)	**Intensity, i (inches/hour)**	**Rainfall Depth (inches)**	**Incremental Depth (inches)**	**Balanced Storm (inches)**
0.00	0.00	0.00	0.00	0.08
0.33	4.77	1.61	1.61	0.08
0.67	3.41	2.30	0.70	0.09
1.00	2.68	2.68	0.38	0.09
1.33	2.16	2.98	0.30	0.09
1.67	1.88	3.17	0.19	0.10
2.00	1.66	3.32	0.15	0.15
2.33	1.46	3.44	0.12	0.30
2.67	1.32	3.52	0.08	0.70
3.00	1.20	3.60	0.08	1.61
3.33	1.09	3.69	0.09	0.38
3.67	1.00	3.79	0.09	0.19
4.00	0.94	3.88	0.10	0.12
4.33	0.89	3.98	0.09	0.09
4.67	0.85	4.07	0.09	0.09
5.00	0.83	4.16	0.09	0.09
5.33	0.80	4.25	0.09	0.09
5.67	0.77	4.34	0.09	0.08
6.00	0.74	4.42	0.08	0.00

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

Time, t (hours)	Balanced Storm (inches)	(C x A) / t_c	Runoff Hydrograph Ordinate (cfs)
0.00	0.08	6.90	0.55
0.33	0.08	6.90	0.56
0.67	0.09	6.90	0.62
1.00	0.09	6.90	0.65
1.33	0.09	6.90	0.65
1.67	0.10	6.90	0.66
2.00	0.15	6.90	1.05
2.33	0.30	6.90	2.04
2.67	0.70	6.90	4.80
3.00	1.61	6.90	11.10
3.33	0.38	6.90	2.59
3.67	0.19	6.90	1.33
4.00	0.12	6.90	0.83
4.33	0.09	6.90	0.65
4.67	0.09	6.90	0.65
5.00	0.09	6.90	0.63
5.33	0.09	6.90	0.59
5.67	0.08	6.90	0.55
6.00	0.00	6.90	0.00

Example 2-10. Computer Modeling

Develop a synthetic runoff/flood hydrograph for the watershed in Example 2-3 using a computer modeling program. For this example HEC-HMS will be used. HEC-HMS is capable of using several different methods for modeling rainfall events, modeling basin conditions, and predicting runoff hydrographs associated with those models. For this example, the NRCS-related methods were chosen for the basin and meteorologic models. The following is a summary of required input parameters for pre-development watershed conditions used in this example:

1. The watershed area is 35 acres (0.0547 mi^2)
2. From Example 2-3, pre-development CN = 68
3. From Example 2-3, maximum soil storage, S, is 4.71 inches
4. Using Equation 2-1 ($I_a = 0.2S$), the sum of initial abstractions, I_a is 0.942 inches
5. From Example 2-4, the time of concentration, t_c, is 20.7 minutes
6. From section 2.7.1, watershed lag time, t_l, is expressed in terms of time of concentration, t_c, using the following equation: $t_l = 0.6\ t_c$. So, t_l = 12.4 minutes
7. From Figure 2-2, the 10-year 24-hour storm depth is P_{10} = 5.58 inches

8. The storm is modeled as an NRCS type II storm
9. The simulation is set to run for 24 hours, using 2 minute time intervals
10. Assume no base flow

Using the above parameters, HEC-HMS calculates the following runoff result summary from the pre-development basin produced by the 10-year, 24-hour storm event:

1. The peak flow is 87.41 cfs, occurring at time 12:06
2. Total Precipitation is 5.58 inches (16.28 acre-ft)
3. Total loss is 3.28 inches (9.56 acre-ft)
4. Total excess is 2.30 inches (6.72 acre-ft)
5. Total direct runoff is 2.30 inches (6.72 acre-ft)

HEC-HMS also allows graphic output of rainfall hyetographs, outflow hydrographs, and direct runoff hydrographs, as well as graphs of incremental precipitation vs. time, excess precipitation vs. time, precipitation loss vs. time, and base flow vs. time (flat line at zero in this case). In addition to results summary output, HEC-HMS provides a time-series table that includes the ordinates for each of the previously mentioned graphs at increments equal to the selected time interval (2 minutes, in this case). Example times-series output (from time 11:30 to 12:40) is provided in Table 2-13.

2.8 Hydrologic Channel Routing

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

2.8.1 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:

$$v = \frac{1}{B}\left(\frac{Q(Y+\Delta Y)-Q(Y)}{\Delta Y}\right) \quad (2\text{-}18)$$

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}:

$$\Delta L_{\min} = \frac{Q}{BS_o v} \quad (2\text{-}19)$$

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):

$$K = \frac{\Delta L}{v} \quad (2\text{-}20)$$

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:

$$X = 0.5(1 - \frac{Q}{BS_o v \Delta L}) \tag{2-21}$$

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, Δt, using the criteria expressed by the following inequality:

$$\frac{K}{3} \leq \Delta t \leq K \tag{2-22}$$

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

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \tag{2-23}$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \tag{2-24}$$

$$C_2 \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \tag{2-25}$$

Where:

K = Muskingum channel routing time constant for a particular segment
X = Dimensionless factor that determines the relative weights of inflow and outflow on the channel storage volume
Δt = Routing time period, in hours

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

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \qquad (2\text{-}26)$$

Where:

O_2 = Outflow rate at the end of routing time period Δt, in cfs
I_2 = Inflow rate at the end of routing time period Δt, in cfs
I_1 = Inflow rate at the beginning of routing time period Δt, in cfs
O_1 = Outflow rate at the beginning of routing time period Δt, in cfs

The routing is performed by repetitively solving Equation 2-26, 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.

2.8.2 NRCS TR-55 Tabular Method

The NRCS 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 1986 version can be obtained from the National Technical Information Service in Springfield, Virginia. 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. TR-55 (1986) can also be found on the NRCS Web site at: http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-other-models.html

2.9 Chapter Equations

$$I_a = 0.2S \quad (2\text{-}1)$$

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

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

Where:

R_T = Rainfall excess for return period T, in inches

P_T = Precipitation depth for return period T, in inches

S = Maximum soil storage, in inches

CN = NRCS watershed curve number

I_a = Initial abstraction

$$\overline{C} = \frac{\sum_{i=1}^{n} C_i A_i}{A_T} \quad (2\text{-}4)$$

Where:

C = Weighted composite runoff coefficient

n = Total number of areas with uniform runoff coefficients

C_i = Runoff coefficient for subarea i from Table 2-4

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

$$C_T = C_{10}\, X_T \quad (2\text{-}5)$$

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 2-4)

X_T = Design storm frequency factor for the return period T (Table 2-5)

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

Where:

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 2-7, 2-8, and 2-9)

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

$$t_c = t_1 + t_2 + t_3 + \ldots + t_i \qquad (2\text{-}7)$$

Where:

t_c = Time of concentration, in minutes

t_1 = Sheet flow travel time, in minutes

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

t_3 = Channelized flow time, in minutes

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

$$t_1 = 0.42\left(\frac{(nL)^{0.8}}{P_2^{\;0.5} s^{0.4}}\right) \qquad (2\text{-}8)$$

Where:

t_1 = Sheet flow travel time, in minutes

L = Sheet flow length, in feet

n = Manning's roughness coefficient for sheet flow (see Table 2-10)

P_2 = 2-year, 24-hour rainfall, in inches (see Figure 2-2)

s = Average slope of sheet flow path, in feet/foot

$$t_1 = 0.94\left(\frac{L^{0.6} n^{0.6}}{I^{0.4} s^{0.3}}\right) \qquad (2\text{-}9)$$

Where:

t_1 = Sheet flow travel time, in minutes
L = Sheet flow length, in feet
n = Manning's roughness coefficient for sheet flow (see Table 2-10)
I = Rainfall intensity, in inches/hour (Use ARI of 10 years in Figure 2-1)
s = Average slope of sheet flow path, in feet/foot

$$t_i = \frac{L_i}{(60)v_i} \qquad (2\text{-}10)$$

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

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

Where:

Q_T = Peak runoff rate for return period T, in cubic feet per second (cfs)
C_T = Runoff coefficient for return period T (see Section 2.4.1)
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 2.5), in minutes
A = Watershed drainage area, in acres, tributary to the design point

$$Q_t = q_u\, A_m\, R_T\, F_p \qquad (2\text{-}12)$$

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 = Rainfall Runoff/Excess, in inches
F_p = Pond and swamp adjustment factor

$\Delta D = 0.133\ t_c$ (2-13)

Where:

ΔD = Incremental duration of runoff producing rainfall, in minutes.

t_c = Time of concentration, in minutes

$$t_p = \frac{\Delta D}{2} + t_l = \frac{\Delta D}{2} + 0.6 t_c \quad (2\text{-}14)$$

Where:

t_p = Time to peak, in minutes

ΔD = Incremental duration of runoff producing rainfall, in minutes

t_l = Lag time, in minutes

t_c = Time of concentration, in minutes

$q_p = 60\ (BA)/t_p$ (2-15)

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 NRCS standard B value of 484 shall be used in the City of Memphis and Shelby County unless another value is approved by the city and/or county engineer.

A = Drainage area, in square miles

t_p = Time to peak, in minutes

$t_b = 2.67\ t_p$ (2-16)

Where:

t_p = Time to peak, in minutes

t_b = Time base, in minutes

$$V = \frac{12\Delta t \Sigma q_i}{A(43{,}560)} \qquad (2\text{-}17)$$

Where:

- V = Volume under the hydrograph, in inches
- Δt = Time increment of the runoff hydrograph ordinates, in seconds
- Σq_i = Sum of the runoff hydrograph ordinates, in cfs, for each time increment i
- A = Watershed drainage area, in acres

$$v = \frac{1}{B}\left(\frac{Q(Y+\Delta Y) - Q(Y)}{\Delta Y}\right) \qquad (2\text{-}18)$$

Where:

- v = Velocity of a small kinematic wave, in feet/second
- $Q(Y)$ = A representative flow rate for channel routing at representative depth Y, in cfs
- ΔY = A small increase in the representative depth of flow in the channel
- $Q(Y+\Delta Y)$ = Flow rate at the new depth $Y + \Delta Y$, in cfs
- B = Top width of water surface, in feet

$$\Delta L_{\min} = \frac{Q}{BS_o v} \qquad (2\text{-}19)$$

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

$$K = \frac{\Delta L}{v} \qquad (2\text{-}20)$$

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

$$X = 0.5(1 - \frac{Q}{BS_o v \Delta L}) \qquad (2\text{-}21)$$

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

$$\frac{K}{3} \leq \Delta t \leq K \qquad (2\text{-}22)$$

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (2\text{-}23)$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (2\text{-}24)$$

$$C_2 \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (2\text{-}25)$$

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

$$O_2 = C_0I_2 + C_1I_1 + C_2O_1 \tag{2-26}$$

Where:

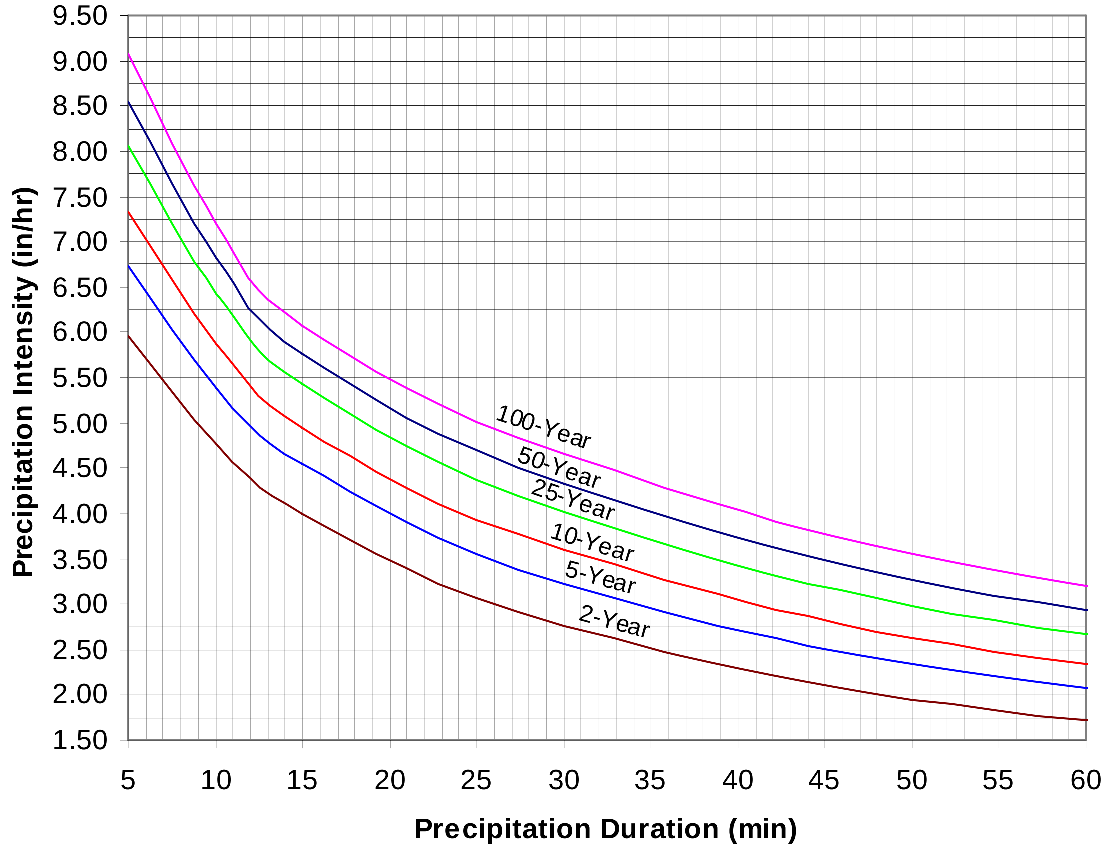
O_2 = Outflow rate at the end of routing time period Δt, in cfs

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

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

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

Figure 2-1: Intensity Duration Frequency Curve

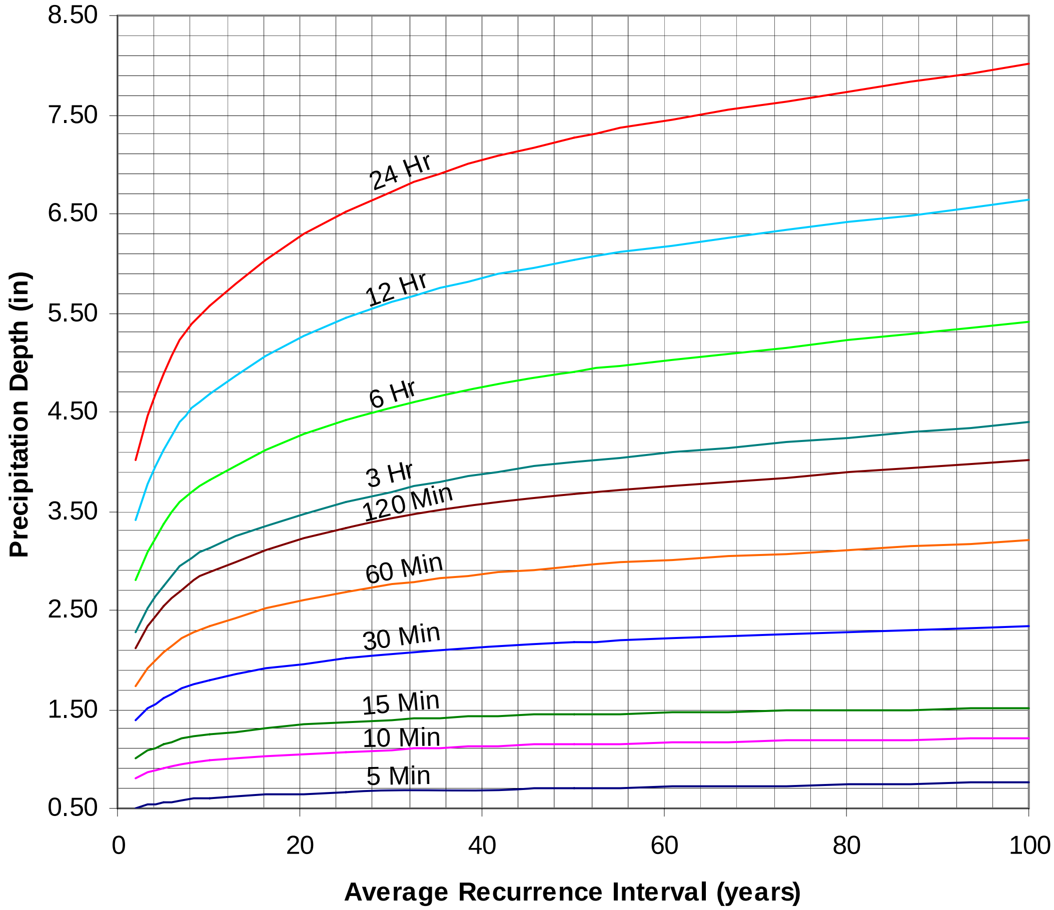


Precipitation Intensity Estimates (in/hr)										
ARI* (Years)	**5 Min**	**10 Min**	**15 Min**	**30 Min**	**60 Min**	**120 Min**	**3 Hr**	**6 Hr**	**12 Hr**	**24 Hr**
2	5.96	4.77	4.00	2.76	1.73	1.06	0.76	0.47	0.28	0.17
5	6.74	5.39	4.55	3.23	2.07	1.27	0.91	0.56	0.34	0.20
10	7.34	5.87	4.95	3.59	2.33	1.44	1.04	0.64	0.39	0.23
25	8.06	6.43	5.43	4.02	2.68	1.66	1.20	0.74	0.45	0.27
50	8.56	6.82	5.76	4.33	2.94	1.83	1.33	0.82	0.50	0.30
100	9.07	7.21	6.08	4.65	3.20	2.01	1.46	0.90	0.55	0.33
200	9.50	7.53	6.34	4.93	3.46	2.18	1.59	0.98	0.60	0.37
500	10.01	7.92	6.64	5.29	3.79	2.41	1.77	1.10	0.67	0.41
1000	10.39	8.18	6.85	5.55	4.05	2.58	1.91	1.18	0.73	0.44

*ARI is the Average Recurrence Interval.

Back to Section 2.3.1, 2.5.1, 2.6.1, 2.7.2

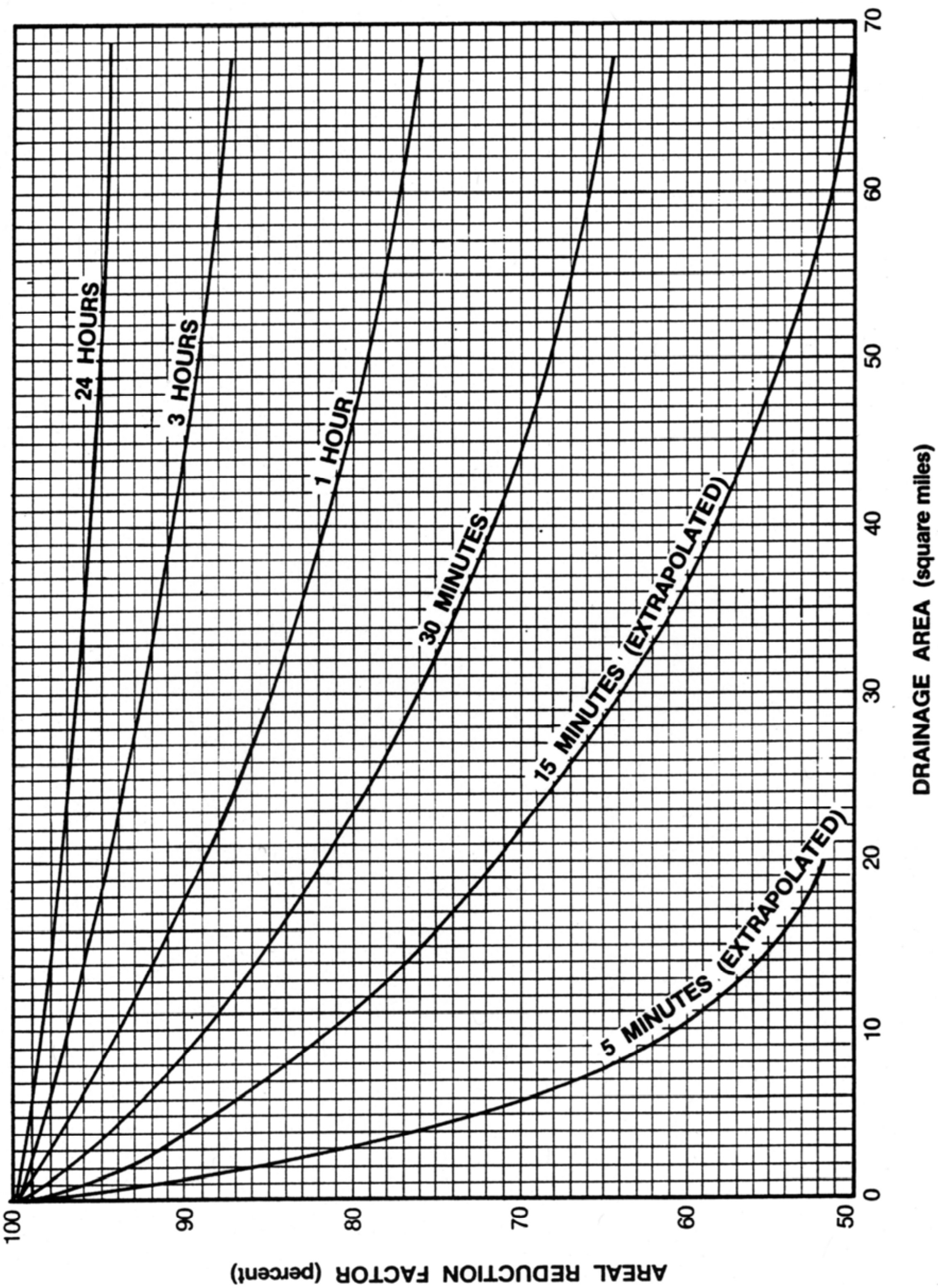
Figure 2-2: Depth Duration Frequency Curve



Precipitation Frequency Estimates (inches)										
ARI* (Years)	**5 Min**	**10 Min**	**15 Min**	**30 Min**	**60 Min**	**120 Min**	**3 Hr**	**6 Hr**	**12 Hr**	**24 Hr**
2	0.50	0.80	1.00	1.38	1.73	2.12	2.28	2.80	3.40	4.01
5	0.56	0.90	1.14	1.62	2.07	2.55	2.74	3.36	4.11	4.89
10	0.61	0.98	1.24	1.79	2.33	2.88	3.12	3.82	4.68	5.58
25	0.67	1.07	1.36	2.01	2.68	3.32	3.60	4.42	5.44	6.52
50	0.71	1.14	1.44	2.17	2.94	3.67	3.99	4.90	6.04	7.27
100	0.76	1.20	1.52	2.33	3.20	4.01	4.39	5.40	6.65	8.02
200	0.79	1.25	1.58	2.47	3.46	4.36	4.79	5.90	7.28	8.80
500	0.83	1.32	1.66	2.64	3.79	4.82	5.32	6.57	8.12	9.84
1000	0.87	1.36	1.71	2.77	4.05	5.16	5.73	7.09	8.76	10.64

*ARI is the Average Recurrence Interval.

Back to Section 2.3.1, 2.3.2, 2.5.1, 2.6.2



Reference: Hershfield (1961).

Figure 2-3
Areal Reduction Factors for Precipitation Durations from 5 Minutes to 24 Hours

Back to [Section 2.3.1]

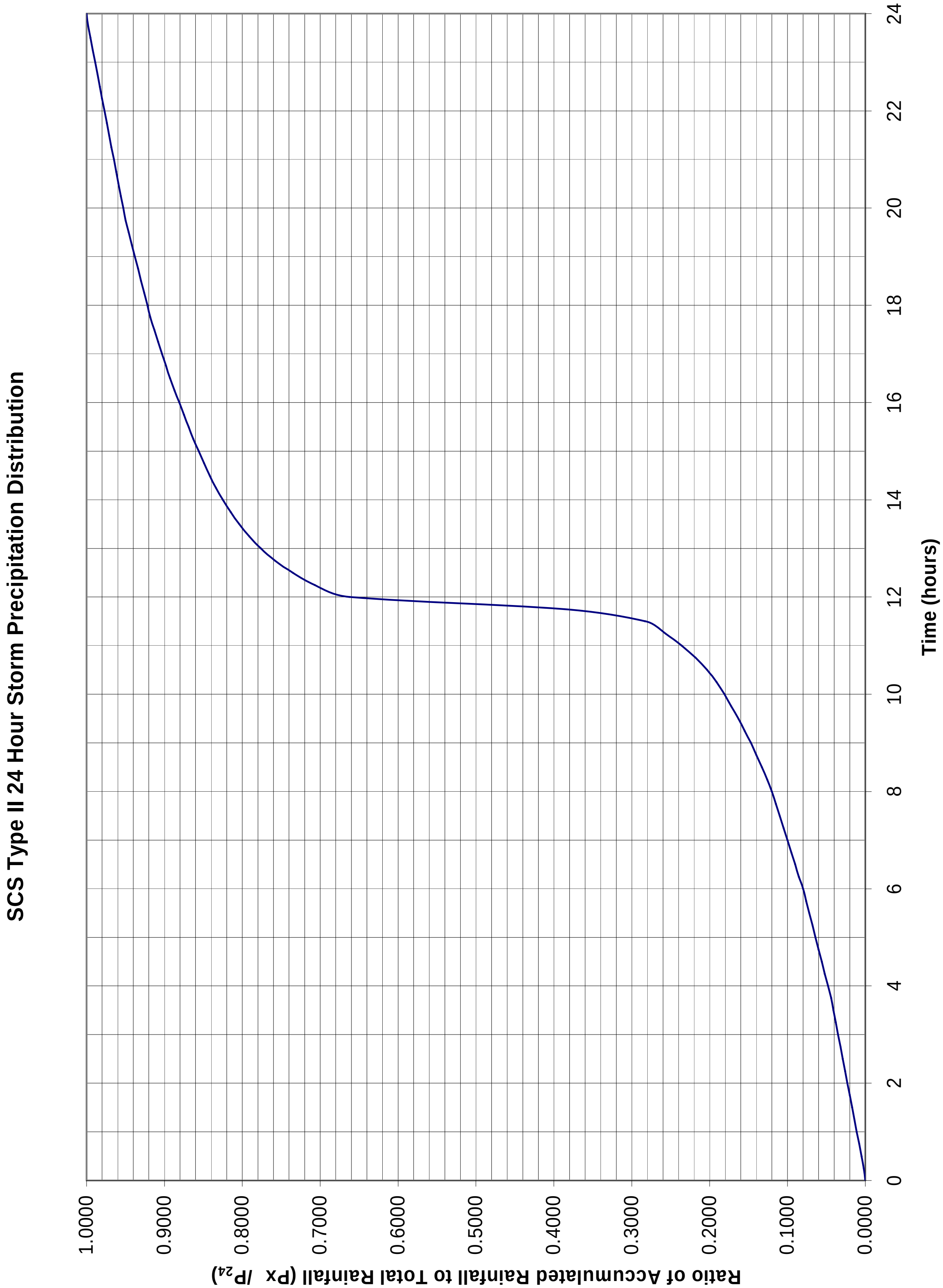
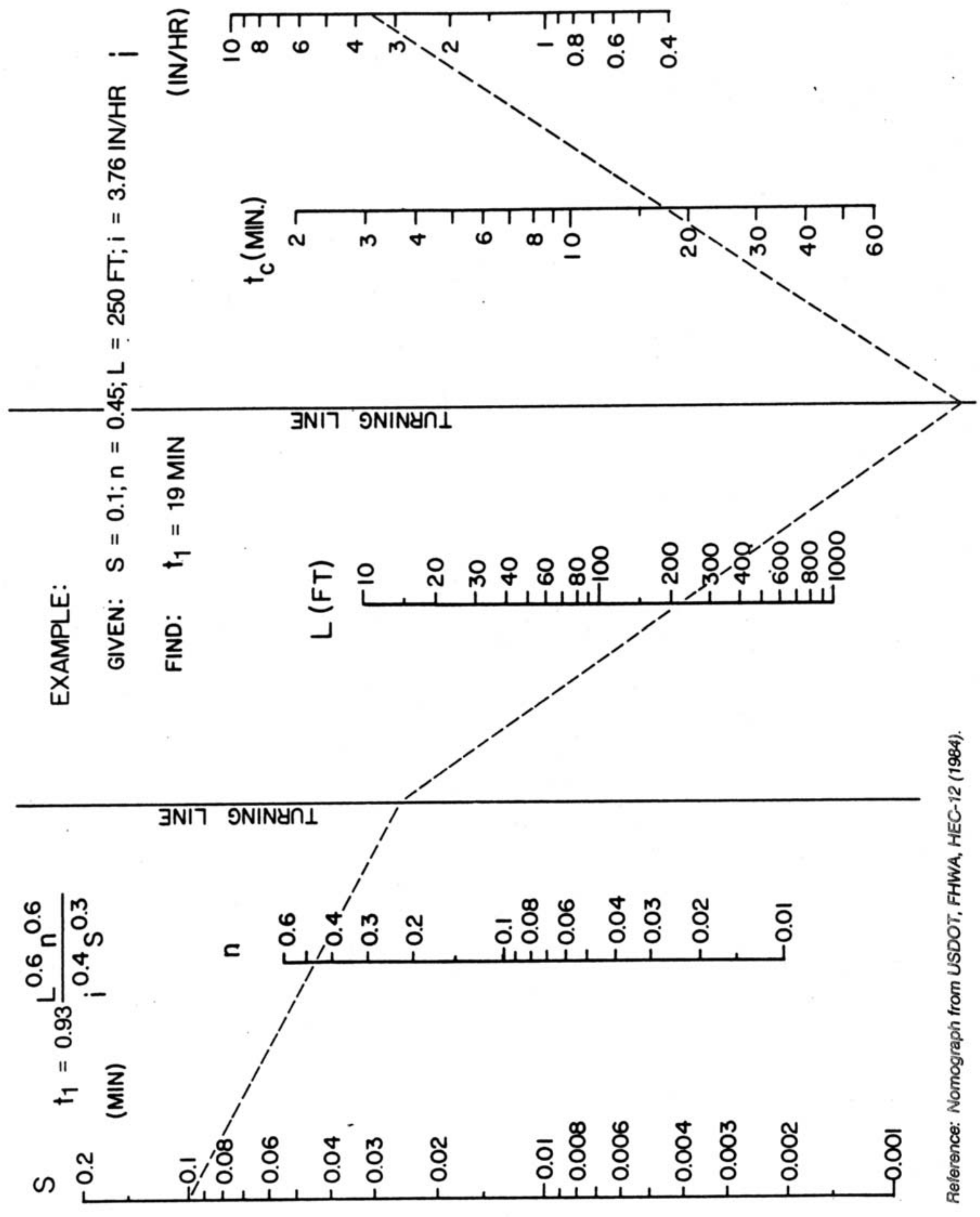


Figure 2-4
24-Hour Rainfall Hyetograph for the City of Memphis and Shelby County

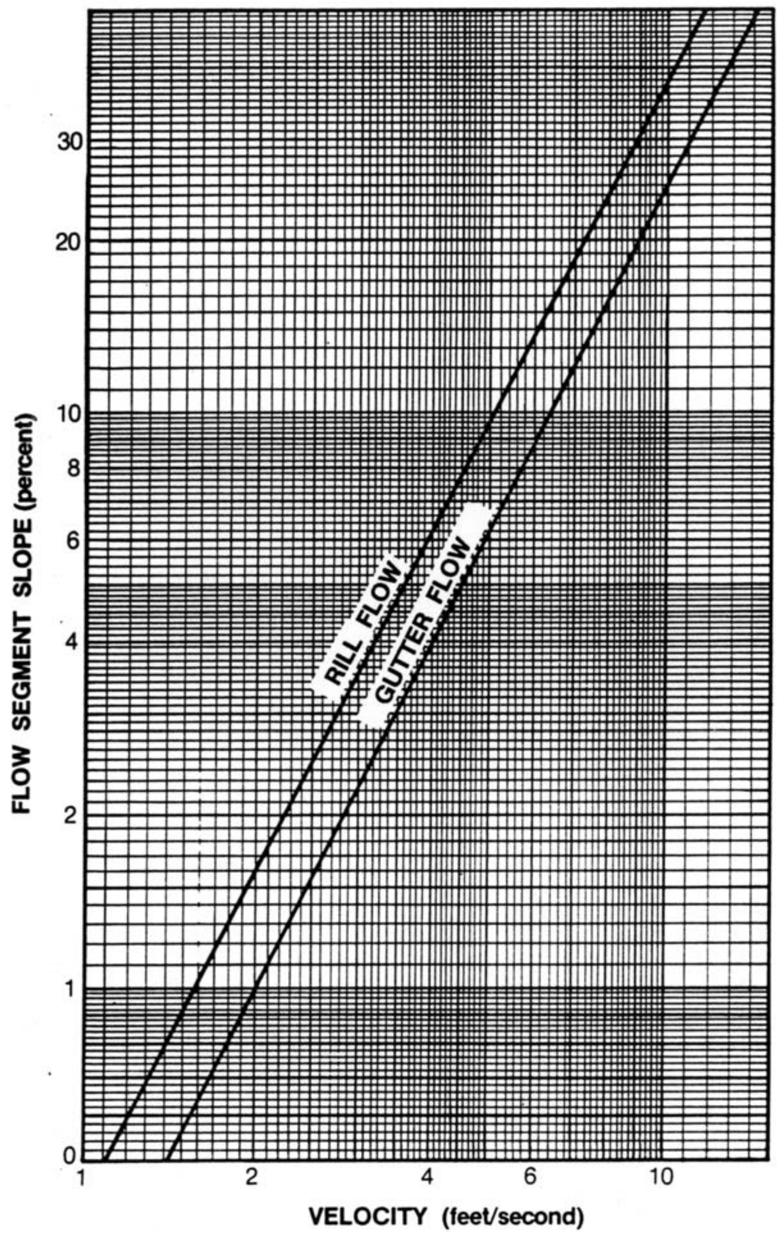
Back to Section 2.3.2



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

Figure 2-5
Kinematic Wave Nomograph for Estimating Sheet Flow Travel Time

Back to Section 2.5.1



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

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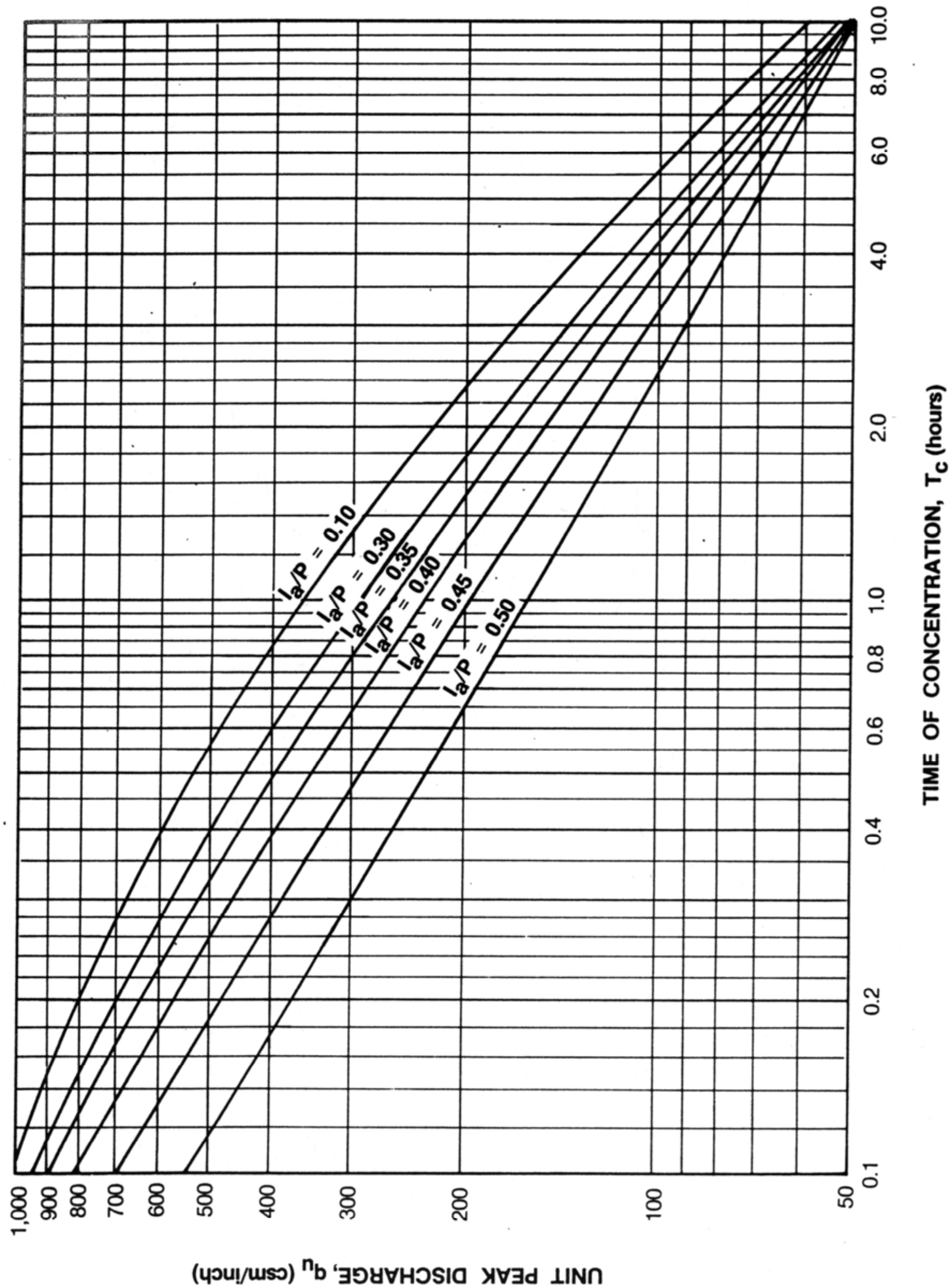
Figure 2-6
Average Velocities for Estimating Travel Time for Shallow Channel Flow



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

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

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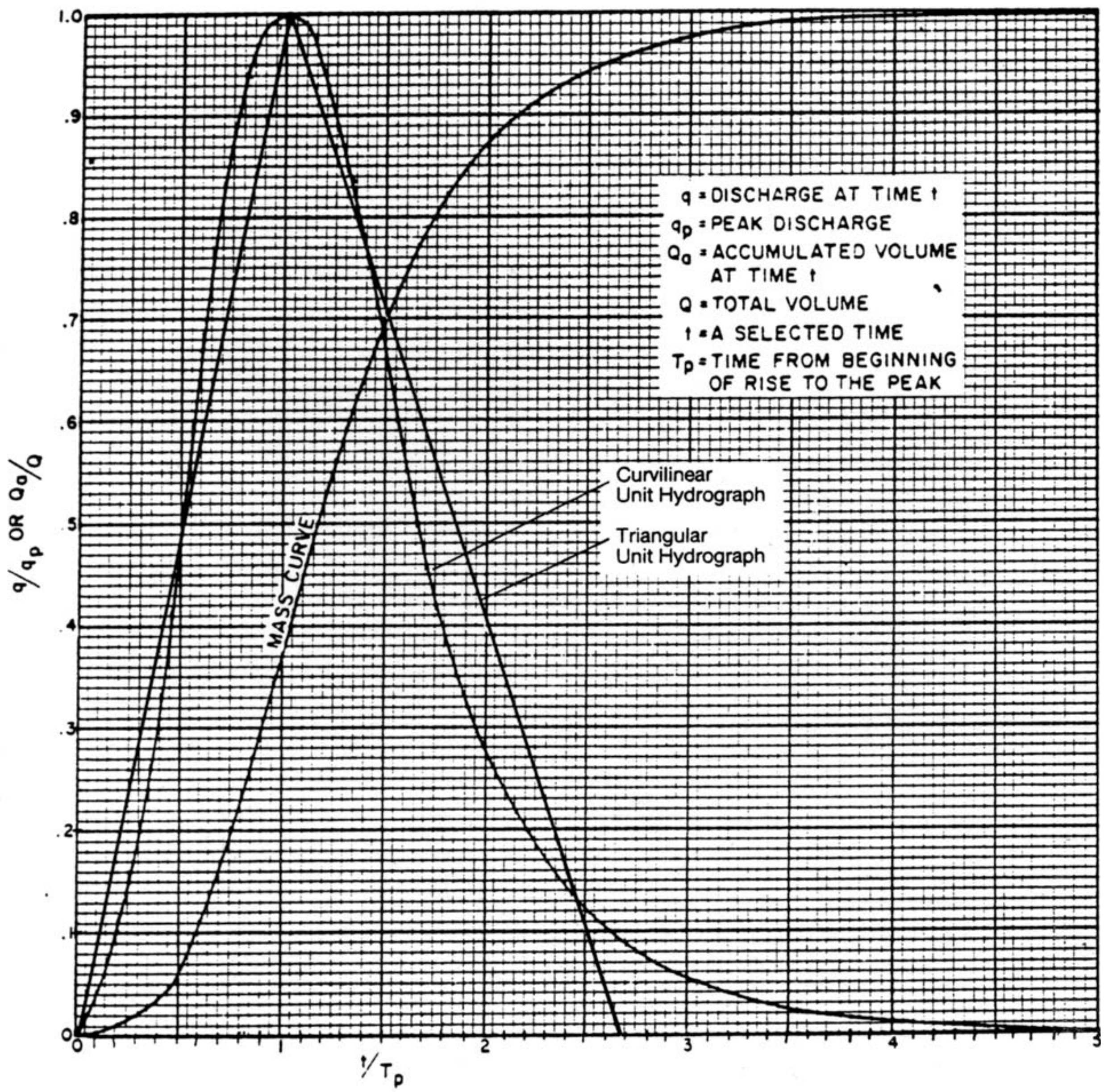

Reference: USDA, SCS, NEH-4 (1972).

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Figure 2-8

NRCS Dimensionless Unit Hydrograph and Mass Curve

Table 2-1
Guidelines For Selecting Hydrologic Procedures

	Hydrologic Method	Section of Manual	Design Peak Flow	Design Hydrograph
1.	Rational Method[a]	2.6.1	Yes	No
2.	NRCS TR-55 Graphical	2.6.2	Yes	No
3.	NRCS TR-55 Tabular	2.7.3	Yes	Yes
4.	Unit Hydrograph Theory	2.7.1	Yes	Yes
5.	Computer Modeling	2.7.4	Yes	Yes

Limits of Application

		Design Storm	Time of Concentration (tc)	Drainage Area (DA)	Impervious (IMP)	Ia/P
1.	Rational Method[a]	t_c	5 min. $\leq t_c \leq$ 30 min	$\leq$ 10 acres	0-100%	N/A
2.	NRCS TR-55 Graphical	24 hr Type II	0.1 hr $\leq t_c \leq$ 10 hr	b	40 $\leq$ CN[d] $\leq$ 98	0.1-0.5
3.	NRCS TR-55 Tabular	24 hr Type II	0.1 hr $\leq t_c \leq$ 2 hr	c	40 $\leq$ CN[d] $\leq$ 98	0.1-0.5
4.	Unit Hydrograph	Any	> 0	> 0	0-100%	N/A
5.	Computer Modeling	Varies	> 0	> 0	0-100% 40 $\leq$ CN[d] $\leq$ 98	N/A

Notes:

[a] = The Rational Method is meant for use as a tool for preliminary estimations of hydrologic conditions for small sites. Use of the Rational Method beyond the limits shown requires approval by city and/or county engineer, 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, NRCS, 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, NRCS, 1983) computer analysis with a DA of 1 square mile.

[d] = Limitations on NRCS curve numbers apply to composite weighted curve numbers only.

N/A = Not Applicable

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Table 2-2
NRCS Type II 24-Hour Storm Precipitation Distribution

Time (hr)	P_x/P_{24} Ratio	Cumulative Rainfall (inches) 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
0.00	0.0000	0.00	0.00	0.00	0.00	0.00	0.00
0.25	0.0020	0.01	0.01	0.01	0.01	0.01	0.02
0.50	0.0050	0.02	0.02	0.03	0.03	0.04	0.04
0.75	0.0080	0.03	0.04	0.04	0.05	0.06	0.06
1.00	0.0110	0.04	0.05	0.06	0.07	0.08	0.09
1.25	0.0140	0.06	0.07	0.08	0.09	0.10	0.11
1.50	0.0170	0.07	0.08	0.09	0.11	0.12	0.14
1.75	0.0200	0.08	0.10	0.11	0.13	0.15	0.16
2.00	0.0230	0.09	0.11	0.13	0.15	0.17	0.18
2.25	0.0260	0.10	0.13	0.15	0.17	0.19	0.21
2.50	0.0290	0.12	0.14	0.16	0.19	0.21	0.23
2.75	0.0320	0.13	0.16	0.18	0.21	0.23	0.26
3.00	0.0350	0.14	0.17	0.20	0.23	0.25	0.28
3.25	0.0380	0.15	0.19	0.21	0.25	0.28	0.30
3.50	0.0410	0.16	0.20	0.23	0.27	0.30	0.33
3.75	0.0440	0.18	0.22	0.25	0.29	0.32	0.35
4.00	0.0480	0.19	0.23	0.27	0.31	0.35	0.38
4.25	0.0520	0.21	0.25	0.29	0.34	0.38	0.42
4.50	0.0560	0.22	0.27	0.31	0.37	0.41	0.45
4.75	0.0600	0.24	0.29	0.33	0.39	0.44	0.48
5.00	0.0640	0.26	0.31	0.36	0.42	0.47	0.51
5.25	0.0680	0.27	0.33	0.38	0.44	0.49	0.55
5.50	0.0720	0.29	0.35	0.40	0.47	0.52	0.58
5.75	0.0760	0.30	0.37	0.42	0.50	0.55	0.61
6.00	0.0800	0.32	0.39	0.45	0.52	0.58	0.64
6.25	0.0854	0.34	0.42	0.48	0.56	0.62	0.68
6.50	0.0900	0.36	0.44	0.50	0.59	0.65	0.72
6.75	0.0950	0.38	0.46	0.53	0.62	0.69	0.76
7.00	0.1000	0.40	0.49	0.56	0.65	0.73	0.80
7.25	0.1050	0.42	0.51	0.59	0.68	0.76	0.84
7.50	0.1100	0.44	0.54	0.61	0.72	0.80	0.88
7.75	0.1150	0.46	0.56	0.64	0.75	0.84	0.92
8.00	0.1200	0.48	0.59	0.67	0.78	0.87	0.96
8.25	0.1260	0.51	0.62	0.70	0.82	0.92	1.01
8.50	0.1330	0.53	0.65	0.74	0.87	0.97	1.07
8.75	0.1400	0.56	0.68	0.78	0.91	1.02	1.12
9.00	0.1470	0.59	0.72	0.82	0.96	1.07	1.18
9.25	0.1550	0.62	0.76	0.86	1.01	1.13	1.24
9.50	0.1630	0.65	0.80	0.91	1.06	1.18	1.31
9.75	0.1720	0.69	0.84	0.96	1.12	1.25	1.38
10.00	0.1810	0.73	0.89	1.01	1.18	1.32	1.45
10.25	0.1910	0.77	0.93	1.07	1.25	1.39	1.53
10.50	0.2030	0.81	0.99	1.13	1.32	1.48	1.63

Table 2-2
NRCS Type II 24-Hour Storm Precipitation Distribution

Time (hr)	P_x / P_{24} Ratio	Cumulative Rainfall (inches) 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
10.75	0.2180	0.87	1.07	1.22	1.42	1.58	1.75
11.00	0.2360	0.95	1.15	1.32	1.54	1.72	1.89
11.25	0.2570	1.03	1.26	1.43	1.68	1.87	2.06
11.50	0.2830	1.13	1.38	1.58	1.84	2.06	2.27
11.75	0.3869	1.55	1.89	2.16	2.52	2.81	3.10
12.00	0.6629	2.66	3.24	3.70	4.32	4.82	5.32
12.25	0.7070	2.84	3.46	3.94	4.61	5.14	5.67
12.50	0.7350	2.95	3.59	4.10	4.79	5.34	5.89
12.75	0.7580	3.04	3.71	4.23	4.94	5.51	6.08
13.00	0.7760	3.11	3.79	4.33	5.06	5.64	6.22
13.25	0.7910	3.17	3.87	4.41	5.16	5.75	6.34
13.50	0.8040	3.22	3.93	4.49	5.24	5.85	6.45
13.75	0.8150	3.27	3.99	4.55	5.31	5.93	6.54
14.00	0.8250	3.31	4.03	4.60	5.38	6.00	6.62
14.25	0.8340	3.34	4.08	4.65	5.44	6.06	6.69
14.50	0.8420	3.38	4.12	4.70	5.49	6.12	6.75
14.75	0.8490	3.40	4.15	4.74	5.54	6.17	6.81
15.00	0.8560	3.43	4.19	4.78	5.58	6.22	6.87
15.25	0.8630	3.46	4.22	4.82	5.63	6.27	6.92
15.50	0.8690	3.48	4.25	4.85	5.67	6.32	6.97
15.75	0.8750	3.51	4.28	4.88	5.70	6.36	7.02
16.00	0.8810	3.53	4.31	4.92	5.74	6.40	7.07
16.25	0.8870	3.56	4.34	4.95	5.78	6.45	7.11
16.50	0.8930	3.58	4.37	4.98	5.82	6.49	7.16
16.75	0.8980	3.60	4.39	5.01	5.85	6.53	7.20
17.00	0.9030	3.62	4.42	5.04	5.89	6.56	7.24
17.25	0.9080	3.64	4.44	5.07	5.92	6.60	7.28
17.50	0.9130	3.66	4.46	5.09	5.95	6.64	7.32
17.75	0.9180	3.68	4.49	5.12	5.99	6.67	7.36
18.00	0.9220	3.70	4.51	5.14	6.01	6.70	7.39
18.25	0.9260	3.71	4.53	5.17	6.04	6.73	7.43
18.50	0.9300	3.73	4.55	5.19	6.06	6.76	7.46
18.75	0.9340	3.75	4.57	5.21	6.09	6.79	7.49
19.00	0.9380	3.76	4.59	5.23	6.12	6.82	7.52
19.25	0.9420	3.78	4.61	5.26	6.14	6.85	7.55
19.50	0.9460	3.79	4.63	5.28	6.17	6.88	7.59
19.75	0.9500	3.81	4.65	5.30	6.19	6.91	7.62
20.00	0.9530	3.82	4.66	5.32	6.21	6.93	7.64
20.25	0.9560	3.83	4.67	5.33	6.23	6.95	7.67
20.50	0.9590	3.85	4.69	5.35	6.25	6.97	7.69
20.75	0.9620	3.86	4.70	5.37	6.27	6.99	7.72
21.00	0.9650	3.87	4.72	5.38	6.29	7.02	7.74
21.25	0.9680	3.88	4.73	5.40	6.31	7.04	7.76

Table 2-2
NRCS Type II 24-Hour Storm Precipitation Distribution

Time (hr)	P_x / P_{24} Ratio	Cumulative Rainfall (inches) 2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
21.50	0.9710	3.89	4.75	5.42	6.33	7.06	7.79
21.75	0.9740	3.91	4.76	5.43	6.35	7.08	7.81
22.00	0.9770	3.92	4.78	5.45	6.37	7.10	7.84
22.25	0.9800	3.93	4.79	5.47	6.39	7.12	7.86
22.50	0.9830	3.94	4.81	5.49	6.41	7.15	7.88
22.75	0.9860	3.95	4.82	5.50	6.43	7.17	7.91
23.00	0.9890	3.97	4.84	5.52	6.45	7.19	7.93
23.25	0.9920	3.98	4.85	5.54	6.47	7.21	7.96
23.50	0.9950	3.99	4.87	5.55	6.49	7.23	7.98
23.75	0.9980	4.00	4.88	5.57	6.51	7.26	8.00
24.00	1.0000	4.01	4.89	5.58	6.52	7.27	8.02

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Table 2-3
Ia 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, NRCS, TR-55 (1986)

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Table 2-4
Runoff Coefficients for Use with the Rational Method

Typical % Impervious Area	Average Slope 0-2%	2-7%	7%+	Zoning[a] and Uses
0	0.10	0.12	0.15	Woodlands — Sandy Soil
0	0.15	0.20	0.25	Woodlands — Heavy Soil
0	0.12	0.17	0.23	Lawns, Unimproved Areas — Sandy Soil
0	0.17	0.25	0.35	Lawns, Unimproved Areas — Heavy Soil
10	0.25	0.30	0.41	Parks and Cemeteries
20	0.28	0.33	0.44	Playgrounds
30	0.34	0.38	0.48	Railroad Yard
40	0.40	0.46	0.55	Suburban Residential[b]
50	0.49	0.53	0.60	(RS-8, RS-10, RS-15) [c]
60	0.56	0.58	0.65	(RS-6, RD, RTH) [c]
70	0.68	0.70	0.75	(R-ML, R-MM, R-MH)
80	0.82	0.83	0.85	(C-L, C-H, C-P, O-L, O-G, I-L, I-H)
90	0.88	0.90	0.92	Dirt/Gravel Roads
100	0.98	0.98	0.98	Asphalt & Concrete Surfaces

Notes:

[a] = See City of Memphis and Shelby County Zoning Ordinance for definition of zoning classification. A zoning atlas for the City of Memphis and Shelby County can be found at: www.dpdgov.com.

[b] = For lot sizes greater than 0.5 acre and less than 1.0 acre. Suburban residential lots smaller than 0.5 acre are subject to use of runoff coefficients for lots of similar size as defined in the City of Memphis and Shelby County Zoning Ordinance. Suburban residential lots greater than 1.0 acre shall be evaluated on an individual basis using runoff coefficients for multiple land uses.

[c] = Recommended coefficients of imperviousness as listed in Table 2-4 shall be the minimum used for new projects unless the designer can document that a lower value should be used. Value noted for single-family residential zonings represents worst case of large houses plus large amounts of additional impervious surfaces, lower value may be justified in most cases. Calculations computing percentage of impervious area for the purpose of justifying a lower C values shall include roofs, driveways, adjacent streets and sidewalks, patios, and storage buildings.

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Table 2-5
Design Storm Frequency Factors
For Pervious Area Runoff Coefficients

Return period (years)	Design Storm Frequency Factor, XT
2, 5, 10	1.0
25	1.1
50	1.2
100	1.25

Note:
The product of C and X_T shall not exceed 1

Reference: Wright-McLaughlin Engineers (1969)

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Section 2.4.3

Table 2-6
Shelby County Soil Classifications and Hydrologic Soil Groups

Soil Survey Area Map Unit Symbol	Map Unit Name	HSG	Total Acres in Shelby County	Percent Area of Shelby County
Ad	Adler Silt Loam	C	855.9	0.2
Bo	Bonn Silt Loam	D	273.4	0.1
Bw	Bowdre Silty Clay	C	5,180.3	1.0
Ca	Calloway Silt Loam	C	15,093.9	3.0
Co	Collins Silt Loam	C	22,670.9	4.5
Cr	Commerce Silt Loam	C	3,532.2	0.7
Cs	Convent Silt Loam	C	2,315.6	0.5
Cu	Crevasse Fine Sand	A	2,491.3	0.5
Cv	Crevasse Silt Loam (Bruno Overwash)	A	585.9	0.1
Fm	Falaya Silt Loam	D	84,402.3	16.8
Fs	Filled Land, Silty (Udorthent, Silty)	B	9,702.9	1.9
Fy	Filled Land, Sandy (Udorthent, Loamy)	B	2,527.8	0.5
GaA	Grenada Silt Loam, 0 To 2 Percent Slopes	C	524.7	0.1
GaB	Grenada Silt Loam, 2 To 5 Percent Slopes	C	30,551.7	6.1
GaB2	Grenada Silt Loam, 2 To 5 Percent Slopes, Eroded	C	8,890.5	1.8
GaC	Grenada Silt Loam, 5 To 8 Percent Slopes	C	904.8	0.2
GaC3	Grenada Silt Loam, 5 To 8 Percent Slopes, Severely Eroded	C	19,048.2	3.8
GaD	Grenada Silt Loam, 8 To 12 Percent Slopes	C	1,269.3	0.3
GaD2	Grenada Silt Loam, 8 To 12 Percent Slopes, Eroded	C	9,499.4	1.9
GgD3	Grenada Complex, 5 To 12 Percent Slopes, Severely Eroded	C	10,614.9	2.1
Gr	Graded Land, Silty Materials(Udorthent, Silty)	B	38,548.1	7.7
Gs	Gullied Land, Silty(Udorthent, Silty)	B	8,724.8	1.7
He	Henry Silt Loam	D	9,975.2	2.0
Ib	Iberia Silt Loam	D	754.0	0.1
Lb	Levees And Borrow Pits(Udorthents, Silty)	Null	1,138.9	0.2

Table 2-6
Shelby County Soil Classifications and Hydrologic Soil Groups

Soil Survey Area Map Unit Symbol	Map Unit Name	HSG	Total Acres in Shelby County	Percent Area of Shelby County
LoB	Loring Silt Loam, 2 To 5 Percent Slopes	C	11,379.1	2.3
LoB2	Loring Silt Loam, 2 To 5 Percent Slopes, Eroded	C	3,254.9	0.6
LoC2	Loring Silt Loam, 5 To 8 Percent Slopes, Eroded	C	3,882.4	0.8
LoD	Loring Silt Loam, 8 To 12 Percent Slopes	C	533.7	0.1
LoD2	Loring Silt Loam, 8 To 12 Percent Slopes, Eroded	C	3,175.4	0.6
LoD3	Loring Silt Loam, 5 To 12 Percent Slopes, Severely Eroded	C	1,995.6	0.4
MeB	Memphis Silt Loam, 2 To 5 Percent Slopes	B	46,714.9	9.3
MeB2	Memphis Silt Loam, 2 To 5 Percent Slopes, Eroded	B	6,159.2	1.2
MeC2	Memphis Silt Loam, 5 To 8 Percent Slopes, Eroded	B	5,467.8	1.1
MeD2	Memphis Silt Loam, 8 To 12 Percent Slopes, Eroded	B	8,167.1	1.6
MeD3	Memphis Silt Loam, 5 To 12 Percent Slopes, Severely Eroded	B	1,736.4	0.3
MeE	Memphis Silt Loam, 12 To 20 Percent Slopes	B	16,566.6	3.3
MeF3	Memphis Silt Loam, 12 To 30 Percent Slopes, Severely Eroded	B	9,602.5	1.9
MeG	Memphis Silt Loam, 30 To 65 Percent Slopes	B	5,429.4	1.1
MP	Mines And Gravel Pits	Null	1,877.3	0.4
Rb	Robinsonville Fine Sandy Loam	B	1,683.4	0.3
Rn	Robinsonville Silt Loam	B	5,187.5	1.0
Sh	Sharkey Clay	D	9,547.2	1.9
SNS	Soils Not Surveyed	Null	3,623.9	0.7
Sw	Swamp(Rosebloom, Ponded)	D	956.0	0.2
Tu	Tunica Silty Clay	D	11,838.4	2.4
W	Water	Null	29,218.0	5.8
Wv	Waverly Silt Loam	D	25,641.7	5.1

Source: NRCS Web Soil Survey, searched February 7, 2006

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Table 2-7
Runoff Curve Numbers for Urban Areas

Cover Type and Hydrologic Condition	Average Percent Impervious Area[b]	A	B	C	D
Fully developed urban areas (vegetation established)[c]:					
Open space (lawn, parks, golf courses, cemeteries, etc.)					
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
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas:					
Newly graded areas (pervious areas only, no vegetation)[d]		77	86	91	94
Idle lands (CNs determined using cover types similar to those in Table 2-8)					

Notes:

[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. CNs for other combinations of conditions may be computed using methods described in TR 55.

[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 shall be computed based on the degree of development (impervious area percentage) and the CNs for the newly graded pervious areas.

Sources:
American Association of State Highway and Transportation Officials. *Model Drainage Manual* [Metric Edition]. Washington, D.C. 1999.

Metropolitan Government of Nashville and Davidson County Department of Public Works Engineering Division. *Stormwater Management Manual*. Nashville, Tennessee. Sept. 1999.

United States Department of Agriculture. Soil Conservation Service. Engineering Division. *Urban Hydrology for Small Watersheds — Technical Release 55*. June 1986 (available for download at http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-tr55.html.

Reference: USDA Urban Hydrology for Small Watersheds — 1986

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Table 2-8
Runoff Curve Numbers for Cultivated Agricultural Lands[a]

Cover Description			Curve numbers for hydrologic soil group			
Cover Type	**Treatment[b]**	**Hydrologic Condition[c]**	**A**	**B**	**C**	**D**
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84

Table 2-8
Runoff Curve Numbers for Cultivated Agricultural Lands[a]

Cover Description: Cover Type	Treatment[b]	Hydrologic Condition[c]	Curve numbers for hydrologic soil group: A	B	C	D
Small grain (continued)	C	Poor	63	74	82	85
		Good	61	73	81	84
	C +CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

Reference: USDA Urban Hydrology for Small Watersheds — 1986

Notes:

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

[b] = Crop residue cover applies only if residue is on at least 5% of the surface throughout the year

[c] = Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close seeded legumes, (d) percent of residue cover on the land surface (good > 20%),and (e) degree of surface roughness

Poor: Factors impair infiltration and tend to increase runoff

Good: Factors encourage average and better than average infiltration and tend to decrease runoff

Sources:

American Association of State Highway and Transportation Officials. *Model Drainage Manual* [Metric Edition]. Washington, D.C. 1999.

Metropolitan Government of Nashville and Davidson County Department of Public Works Engineering Division. *Stormwater Management Manual*. Nashville, Tennessee. Sept. 1999.

United States Department of Agriculture. Soil Conservation Service. Engineering Division. *Urban Hydrology for Small Watersheds — Technical Release 55*. June 1986 (available for download at http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-tr55.html.

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Table 2-9
Runoff Curve Numbers for Rural Areas and Other Agricultural Lands[a]

Cover Description		Curve numbers for hydrologic soil group			
Cover Type	**Hydrologic Condition[c]**	**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

Notes:

[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.

Sources:

American Association of State Highway and Transportation Officials. *Model Drainage Manual* [Metric Edition]. Washington, D.C. 1999.

Metropolitan Government of Nashville and Davidson County Department of Public Works Engineering Division. *Stormwater Management Manual*. Nashville, Tennessee. Sept. 1999.

United States Department of Agriculture. Soil Conservation Service. Engineering Division.
Urban Hydrology for Small Watersheds — Technical Release 55. June 1986 (available for download at http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-tr55.html.

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Table 2-10
Sheet 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 prairie[a]	.15	.10 — .20
Dense grass[c]	.24	.17 — .30
Bermuda grass[c]	.41	.30 — .48
Woods	.45	— —

Notes:
[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 sheet 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

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Table 2-11
NRCS Dimensionless Unit Hydrograph Ratios

Time Ratios (t/tp)	Discharge Ratios (q/qp)	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, NRCS, NEH-4 (1972)

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Table 2-12
Flood / Excess Hydrograph Computation for Example 2-8

Excess Time[a], t (hrs)		0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80	0.85	0.90	0.95	
Excess[a] (inches)		0.004	0.114	0.057	0.052	0.047	0.044	0.041	0.034	0.028	0.024	0.024	
Time t (hours)	**UHG Discharge, q**	**Lagged Incremental Excess Flows (cfs)**											**Total Runoff (cfs)**
0.00	0.0	0.00											
0.05	10.6	0.04	0.00										*0.00*
0.10	32.8	0.12	1.20	0.00									*1.20*
0.15	69.9	0.26	3.73	0.61	0.00								*4.34*
0.20	98.5	0.37	7.95	1.88	0.55	0.00							*10.39*
0.25	105.9	0.40	11.20	4.01	1.71	0.50	0.00						*17.43*
0.30	98.5	0.37	12.05	5.65	3.64	1.54	0.46	0.00					*23.35*
0.35	82.6	0.31	11.20	6.08	5.13	3.27	1.44	0.43	0.00				*27.56*
0.40	59.3	0.22	9.40	5.65	5.52	4.61	3.06	1.35	0.36	0.00			*29.95*
0.45	41.3	0.16	6.75	4.74	5.13	4.96	4.31	2.86	1.11	0.29	0.00		*30.16*
0.50	29.7	0.11	4.70	3.41	4.31	4.61	4.63	4.04	2.36	0.91	0.26	0.00	*29.22*
0.55	21.9	0.08	3.37	2.37	3.09	3.87	4.31	4.34	3.33	1.93	0.80	0.25	*27.67*
0.60	15.6	0.06	2.49	1.70	2.15	2.78	3.61	4.04	3.58	2.72	1.70	0.79	*25.57*
0.65	11.3	0.04	1.77	1.26	1.55	1.93	2.59	3.39	3.33	2.93	2.40	1.67	*22.82*
0.70	8.2	0.03	1.29	0.89	1.14	1.39	1.81	2.43	2.79	2.72	2.58	2.36	*19.41*
0.75	5.8	0.02	0.93	0.65	0.81	1.03	1.30	1.69	2.01	2.28	2.40	2.54	*15.63*
0.80	4.2	0.02	0.66	0.47	0.59	0.73	0.96	1.22	1.40	1.64	2.01	2.36	*12.03*
0.85	3.1	0.01	0.48	0.33	0.43	0.53	0.68	0.90	1.00	1.14	1.44	1.98	*8.92*
0.90	2.2	0.01	0.35	0.24	0.30	0.38	0.50	0.64	0.74	0.82	1.01	1.42	*6.40*
0.95	1.6	0.01	0.25	0.18	0.22	0.27	0.36	0.46	0.53	0.61	0.72	0.99	*4.59*
1.00	1.2	0.00	0.18	0.13	0.16	0.20	0.25	0.33	0.38	0.43	0.53	0.71	*3.31*
1.05	0.9	0.00	0.13	0.09	0.12	0.14	0.19	0.24	0.28	0.31	0.38	0.53	*2.40*
1.10	0.6	0.00	0.10	0.07	0.08	0.10	0.13	0.17	0.20	0.23	0.28	0.37	*1.73*
1.15	0.4	0.00	0.07	0.05	0.06	0.07	0.10	0.13	0.14	0.16	0.20	0.27	*1.26*
1.20	0.2	0.00	0.05	0.04	0.04	0.05	0.07	0.09	0.10	0.12	0.14	0.20	*0.90*
1.25	0.0	0.00	0.02	0.02	0.03	0.04	0.05	0.07	0.08	0.08	0.10	0.14	*0.64*
1.30			0.00	0.01	0.02	0.03	0.04	0.05	0.05	0.06	0.07	0.10	*0.44*

Table 2-12
Flood / Excess Hydrograph Computation for Example 2-8

Excess Time[a], t (hrs)		**0.45**	**0.50**	**0.55**	**0.60**	**0.65**	**0.70**	**0.75**	**0.80**	**0.85**	**0.90**	**0.95**	
Excess[a] (inches)		**0.004**	**0.114**	**0.057**	**0.052**	**0.047**	**0.044**	**0.041**	**0.034**	**0.028**	**0.024**	**0.024**	
Time t (hours)	**UHG Discharge, q**	**Lagged Incremental Excess Flows (cfs)**											**Total Runoff (cfs)**
1.35				0.00	0.01	0.02	0.03	0.04	0.04	0.04	0.05	0.07	*0.30*
1.40					0.00	0.01	0.02	0.03	0.03	0.03	0.04	0.05	*0.21*
1.45						0.00	0.01	0.02	0.02	0.02	0.03	0.04	*0.14*
1.50							0.00	0.01	0.01	0.02	0.02	0.03	*0.09*
1.55								0.00	0.01	0.01	0.02	0.02	*0.05*
1.60									0.00	0.01	0.01	0.02	*0.03*
1.65										0.00	0.00	0.01	*0.01*
1.70											0.00	0.00	*0.00*
1.75												0.00	*0.00*
1.80													*0.00*
												Sq$_i$ (cfs) =	**328.2**

Note:
[a]Excess rainfall runoff occurs between t = 0.45 hours and t = 0.95 hours. See Rainfall Excess Hyetograph in Example 2-8.

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Table 2-13
Example HEC-HMS Time-Series Output

Date	Time	Precip (IN)	Loss (IN)	Excess (IN)	Direct Flow (CFS)	Base Flow (CFS)	Total Flow (CFS)
1-Jan-06	11:30	0.025	0.019	0.005	3.323	0.000	3.323
1-Jan-06	11:32	0.032	0.025	0.007	3.546	0.000	3.546
1-Jan-06	11:34	0.043	0.033	0.010	3.836	0.000	3.836
1-Jan-06	11:36	0.058	0.043	0.015	4.268	0.000	4.268
1-Jan-06	11:38	0.075	0.055	0.020	4.946	0.000	4.946
1-Jan-06	11:40	0.090	0.064	0.026	5.992	0.000	5.992
1-Jan-06	11:42	0.100	0.069	0.032	7.521	0.000	7.521
1-Jan-06	11:44	0.110	0.072	0.037	9.621	0.000	9.621
1-Jan-06	11:46	0.136	0.086	0.050	12.379	0.000	12.379
1-Jan-06	11:48	0.181	0.108	0.072	15.949	0.000	15.949
1-Jan-06	11:50	0.237	0.133	0.104	20.651	0.000	20.651
1-Jan-06	11:52	0.266	0.138	0.128	26.935	0.000	26.935
1-Jan-06	11:54	0.262	0.126	0.137	35.194	0.000	35.194
1-Jan-06	11:56	0.228	0.102	0.126	45.442	0.000	45.442
1-Jan-06	11:58	0.180	0.076	0.104	57.023	0.000	57.023
1-Jan-06	12:00	0.123	0.050	0.073	68.613	0.000	68.613
1-Jan-06	12:02	0.065	0.026	0.039	78.501	0.000	78.501
1-Jan-06	12:04	0.026	0.010	0.016	85.109	0.000	85.109
1-Jan-06	12:06	0.015	0.006	0.009	87.412	0.000	87.412
1-Jan-06	12:08	0.025	0.010	0.015	85.233	0.000	85.233
1-Jan-06	12:10	0.033	0.013	0.020	79.306	0.000	79.306
1-Jan-06	12:12	0.035	0.013	0.021	71.058	0.000	71.058
1-Jan-06	12:14	0.030	0.011	0.019	62.071	0.000	62.071
1-Jan-06	12:16	0.026	0.010	0.016	53.568	0.000	53.568
1-Jan-06	12:18	0.024	0.009	0.015	46.205	0.000	46.205
1-Jan-06	12:20	0.023	0.009	0.015	40.173	0.000	40.173
1-Jan-06	12:22	0.023	0.008	0.015	35.397	0.000	35.397
1-Jan-06	12:24	0.022	0.008	0.014	31.623	0.000	31.623
1-Jan-06	12:26	0.020	0.007	0.012	28.545	0.000	28.545
1-Jan-06	12:28	0.018	0.007	0.012	25.913	0.000	25.913
1-Jan-06	12:30	0.017	0.006	0.011	23.607	0.000	23.607
1-Jan-06	12:32	0.016	0.006	0.010	21.581	0.000	21.581
1-Jan-06	12:34	0.016	0.006	0.010	19.814	0.000	19.814
1-Jan-06	12:36	0.015	0.005	0.010	18.270	0.000	18.270
1-Jan-06	12:38	0.015	0.005	0.010	16.915	0.000	16.915
1-Jan-06	12:40	0.015	0.005	0.010	15.729	0.000	15.729

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City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations
Volume 2: Drainage Manual
Chapter 3: Open Channel Hydraulics
Volume 3: Best Management Practices Manual

Revision: 0
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Acronym List
(Chapter 3)

α	Energy coefficient
Δd	Super elevation of the water surface profile due to the bend, (ft)
ΔE	Change in specific energy, (ft)
Δx	Length of channel between consecutive depths of flow, (ft)
γ_s	Specific weight of stone, (lbs/ft^3)
A	Cross sectional area of flow
b	Bottom width, (ft)
C	Constant
C_b	Bend correction coefficient
C_g	Wight correction coefficient
d	Depth of flow, (ft)
d	Selected stone diameter
d_1	Depth above jump, (ft)
d_2	Depth below jump, (ft)
d_{30}	Diameter of stone for which 30 percent, by weight, of the gradation is finer, (ft)
d_{50}	Diameter of stone for which 50 percent, by weight, of the gradation is finer, (ft)
D	Hydraulic depth, (ft)
D	Depth of flow above stone
E	Specific Energy
FHWA	Federal Highway Administration
Fr	Froude Number
g	Acceleration due to gravity, (32.2 ft/sec^2)
h_e	Eddy head loss, (ft)
H	Total Head
HEC-RAS	Hydrologic Engineering Center — River Analysis System
k_e	Eddy head loss coefficient, (ft)
K	Channel conveyance
K_T	Trapezoidal open channel conveyance factor
L_p	Length of downstream protection
m_5	Coefficient associated with the degree of channel meandering
n	Manning's roughness coefficient
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

P	Wetted perimeter (the cross section length touched by water), (ft)
Q	Discharge rate, (ft^3/sec)
R	Hydraulic radius, (ft)
R_b	Centerline bend radius
R_c	Mean radius of the bend, (ft)
S	Energy grade line slope, (ft/ft)
S	Channel bottom slope, (ft/ft)
S_f	Friction slope, (ft/ft)
S_o	Channel bottom slope, (ft/ft)
SWMM	Storm Water Management Model
T	Top width of water surface, (ft)
TDEC	Tennessee Department of Environment and Conservation
USDOT	United States Department of Transportation
USGS	United States Geological Survey
v	Average velocity
v	Mean velocity above the stone, (ft/sec)
v_1	Velocity above jump, (ft/sec)
V	Mean cross sectional velocity, (ft/sec)
V_m	Maximum velocity
W	Stone weight, in pounds
y_c	Critical depth, (ft)
y_n	Normal depth, (ft)
z	Surface elevation
Z	Critical flow section factor

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3.0 OPEN CHANNEL HYDRAULICS

This chapter contains criteria and methods for open-channel hydraulics design, for a project with artificial and natural channel improvements. In general, this chapter provides criteria for the geometric and hydraulic design of open channels, as well as guidance on providing them with linings that will serve to minimize erosion and promote channel stability. To properly utilize the information in this chapter, the designer should be familiar with the behavior of water in open channels and understand the basic concepts related to hydraulic forces exerted by the flow of water. The designer should have a firm understanding of the basic hydraulics of open-channel flow.

Storm water runoff from projects should not discharge onto adjacent property except at appropriate outlet points. Thus, the purpose of open channels is to convey storm water runoff from, through, and/or around facilities without damage to the facilities or adjacent property.

A variety of reference materials is available to assist the designer in gaining a more complete understanding of open-channel hydraulics. Online sources include the U.S. Army Corps of Engineers' HEC-RAS Hydraulic Reference Guide and the Hydraulic Engineering Center, and the Federal Highway Administration HDS-5 and Design Charts for Open-Channel Flow (FHWA #EPD-86-102).

3.1 Variations in Open-Channel Flow

To provide an effective design for open-channel flow, the designer should have a clear understanding of the principles of open-channel hydraulics. Open-channel flow occurs when the water surface is exposed to the atmosphere and the force driving the flow is gravity. Flow in open channels may be classified as follows:

- *Steady vs. Unsteady Flow:* Steady flow occurs when the flow rate is constant, that is, where it does not vary with time. In contrast, unsteady flow occurs in situations where discharge varies with time.

- *Uniform vs. Non-Uniform Flow:* Steady flows are further classified according to the configuration of the water surface profile. Uniform steady flow occurs where the channel cross section, roughness, and slope are constant. In this situation, the water surface and energy grade line are parallel to the flow line of the channel. If the channel properties vary along the ditch alignment, the water surface will no longer be parallel to the flow line, and the flow is classified as non-uniform or varied flow.

- *Gradually vs. Rapidly Varied Flow:* Varied flow may be classified as gradually or rapidly varied flow, depending on the rate of change in the flow rate, velocity, area, or slope. Rapidly varied flows usually involve highly turbulent flow conditions, such as flows over spillways or hydraulic jumps.

The design methods in this chapter for open-channel flow assumes steady, uniform flow.

3.2 Channel Linings

The three main classifications of open-channel linings are vegetative, flexible, and rigid. Vegetative linings include grass with mulch, sod, and lapped sod. Flexible linings include rock riprap and soil bioengineering techniques that generally include vegetative lining. Rigid linings are generally concrete.

3.2.1 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 velocity for bare soils (see Section 3.3.5)
2. Standing or continuous flowing water
3. Lack of regular maintenance necessary to prevent domination by taller vegetation
4. 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. If soils do not support grass growth, 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. Sod should be staked on steeper slopes to prevent sliding (4:1 or steeper). Additional information on vegetation is presented in Volume 3 — Best Management Practices.

3.2.2 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 erosive velocity. Filter fabric is required beneath the riprap to allow the infiltration and exfiltration of water without allowing the migration of underlying soils. Additional information on riprap is presented in Volume 3 — Best Management Practices Manual: ES-23. These linings are usually less expensive than other rigid linings and have self-healing qualities that reduce maintenance.

Another form of flexible lining blends vegetation and geotextile reinforcements, or "bioengineering" techniques. This is presented in more detail in Volume 3 — Best Management Practices Manual: ES-22. These fact sheets should be consulted for additional information on the application of and the design criteria for these practices.

3.2.3 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 and flexible lining 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.

3.3 Design Criteria

3.3.1 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 10 to 1.

2. Low-flow sections shall be considered in the design of channels with large cross sections (flows greater than 100 cfs).

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

4. Superelevation of the water surface at horizontal curves shall be accounted for by increased freeboard (see Section 3.4.8).

3.3.2 Channel Transitions

The following criteria shall be considered at channel transitions:

1. Transition to channel sections shall be smooth and gradual

2. The transition length between a flow lines shall be, at a minimum, 5-feet in length to every 1-foot increase in width

3. Energy losses in transitions shall be accounted for as part of water surface profile calculations (see Section 3.5)

3.3.3 Manning’s n Value

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 floodplain 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.

3.3.4 Return Period

Minor drainage systems shall be sized to handle a 10-year design storm within drainage easement; major systems shall be sized to handle a 100-year design storm without flooding structures. 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.

3.3.5 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 (see tables which are located at the end of the chapter). 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 3.4.9 and Volume 3, and riprap procedures are presented in Section 3.4.10 and Volume 3.

3.3.6 Existing Stream Modifications

Modifications to existing streams may impact facilities upstream and downstream of the proposed modification area. When modifications to existing streams are required, channel stability must be assessed. The following considerations and actions should be considered/performed:

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 man-made. The composition and erodibility of bed and bank material should be determined.

2. Backwater calculations (see Section 3.5) 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 3-3 can be used to evaluate the likely channel response to proposed changes.

4. Channel modifications should be sized to match existing sizes and shapes. 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.

6. Streams that are classified as "Waters of the State" require the Tennessee Department of Environmental Conservation's (TDEC's) approval prior to any modifications.

3.4 Open-Channel Flow Methods and Equations

This chapter describes the most common equations used in designing and analyzing open-channel flow. The basic principles of fluid mechanics can be applied to open-channel flow analysis (i.e., continuity, momentum, and energy).

3.4.1 Continuity Equation

The conservation of mass in fluid mechanics is defined by the continuity equation. The continuity equation is used for a one-dimensional, steady-flow system of water. The continuity equation is expressed as follows:

$$Q = VA \tag{3-1}$$

Where:

Q	=	discharge, (ft^3/s)
A	=	cross sectional area of flow, (ft^2)
V	=	mean cross sectional velocity, (ft/s)

The continuity equation can be used to compute the unknown variable if the other two variables are known.

Figures 3-1 and 3-2 plot specific energy vs. depth of flow, and discharge vs. specific energy, respectively, and illustrate important properties of critical flow.

3.4.2 Flow Energy

The energy head relative to the channel bottom is defined as the specific energy, E. It is the sum of the pressure head (or depth, d) and the velocity head ($V^2/2g$). If the channel is not too steep (generally slopes less than 10 percent) and the streamlines are straight and parallel, the specific energy is expressed as:

$$E = d + V^2/2g \qquad (3\text{-}2)$$

Where:

d	=	depth of flow, (ft)
V	=	mean cross-sectional velocity, (ft/s)
g	=	acceleration due to gravity, (32.2 ft/sec^2)

3.4.3 Manning's n Values

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 3-3 (see figures which are located at the end of the chapter), 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 3-3 is used iteratively as described in Section 3.4.9.

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

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

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 3-3 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 floodplains.

3.4.4 Uniform Flow Equations

Manning's equation is used to compute the mean velocity in an open channel with steady uniform flow as follows:

$$V = (\frac{1.486}{n})R^{0.667}S^{0.5} \qquad (3\text{-}4)$$

Where:

V	=	mean cross sectional velocity, (ft/s)
n	=	Manning's coefficient of channel roughness, (dimensionless)
A	=	cross sectional area of flow, (ft^2)
P	=	wetted perimeter (the cross section length touched by water), (ft)
R	=	hydraulic radius, (ft), (R = A/P)
S	=	energy grade line slope, (ft/ft)

The flow rate of an open-channel flow can be determined by combining Manning's equation with the continuity equation. The open-channel flow rate equation is as follows:

$$Q = (\frac{1.486}{n})AR^{0.667}S^{0.5} \qquad (3\text{-}5)$$

Where:

Q	=	Flow Rate (ft^3/sec)
n	=	Manning's coefficient of channel roughness, (dimensionless)
A	=	cross sectional area of flow, (ft^2)
P	=	wetted perimeter (the cross section length touched by water), (ft)
R	=	hydraulic radius, (ft), (R = A/P)
S	=	energy grade line slope, (ft/ft)

The energy grade line slope of an open channel can be defined by rearranging the flow rate equations and computed as follows:

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

Where:

S	=	energy grade line slope, (ft/ft)
Q	=	Flow Rate (ft^3/sec)
n	=	Manning's coefficient of channel roughness, (dimensionless)
A	=	cross sectional area of flow, (ft^2)
P	=	wetted perimeter (the cross section length touched by water), (ft)
R	=	hydraulic radius, (ft), (R = A/P)

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.

3.4.5 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 3-4. 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 3-5.

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).

3.4.6 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from Equation 3-5. The slope can be calculated using Equation 3-6 when the discharge, roughness coefficient, area, and hydraulic radius are known. Nomographs for obtaining direct solutions to Manning's Equation are presented in Figure 3-6. Figure 3-6 provides a general solution for the velocity form of Manning's Equation, while Figure 3-7 provides a solution of Manning's Equation for trapezoidal channels.

The following steps are used for the general solution nomograph in Figure 3-6:

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

The trapezoidal channel nomograph solution to Manning's Equation in Figure 3-7 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. 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.

3.4.7 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:

$$AR^{2/3} = \frac{Qn}{1.49S^{1/2}} \qquad (3\text{-}7)$$

Where:

A	=	Cross-sectional area, (ft^2)
R	=	Hydraulic radius, (ft)
Q	=	Discharge rate for design conditions, (ft^3/sec)
n	=	Manning's roughness coefficient, (dimensionless)
S	=	Slope of the energy grade line, (ft/ft)

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 3-7 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 3-8 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:

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

Where:

K_T	=	Trapezoidal open channel conveyance factor
Q	=	Discharge rate for design conditions, (ft3/sec)
n	=	Manning's roughness coefficient, (dimensionless)
b	=	Bottom width, (ft)
S	=	Slope of the energy grade line, (ft/ft)

3. Enter the x-axis of Figure 3-8 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.

3.4.8 Critical Flow Calculations

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

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad (3\text{-}9)$$

Where:

Q	=	Discharge rate for design conditions, (ft3/sec)
g	=	Acceleration due to gravity, (32.2 ft/sec2)
A	=	Cross-sectional area, (ft2)
T	=	Top width of water surface, (ft)

A trial-and-error procedure is needed to solve Equation 3-9. Semi-empirical equations (as presented in Table 3-6) or section factors (as presented in Figure 3-4) 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:

$$Z = Q / \sqrt{g} \qquad (3\text{-}10)$$

Where:

Z	=	Critical flow section factor
Q	=	Discharge rate for design conditions, (ft3/sec)
g	=	Acceleration due to gravity, (32.2 ft/sec2)

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 is a dimensionless number representing the ratio of the inertial forces to gravitational forces. The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = V/(gd)^{0.5} \tag{3-11}$$

Where:

Fr	=	Froude Number
V	=	mean cross sectional velocity, (ft/s)
g	=	acceleration due to gravity, (32.2 ft/sec^2)
D	=	hydraulic depth*, (ft), (D= A/T)
A	=	cross sectional area of flow, (ft^2)
T	=	channel top width at the water surface, (ft)

* For rectangular channels, hydraulic depth equals depth of flow.

The Froude Number can be used to determine if the flow regime is subcritical or supercritical, and can be applied to channel flow at any cross section. When the Froude Number is less than one, the flow is considered subcritical. The flow is supercritical where the Froude Number is greater than one. The Froude Number is equal to one for critical flow conditions.

3.4.9 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 3-3. Use retardance Class D for permanent vegetation and E for temporary construction.

4. Calculate the hydraulic radius using the equation:

$$R = \frac{(vR)}{V_m} \qquad (3\text{-}12)$$

Where:

R = Hydraulic radius of flow, in feet
vR = Value obtained from Figure 3-3 in Step 3
V_m = Maximum velocity from Step 2

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

$$vR = \frac{1.49 R^{5/3} S^{1/2}}{n} \qquad (3\text{-}13)$$

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 3-3 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 3-7 or 3-8, as described in Section 3.4.7. The depth of flow for other channel shapes can be evaluated using the trial-and-error procedure in Section 3.4.7.

8. If bends are considered, calculate the length of downstream protection, L_p, for the bend using Figure 3-9. 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 3-4 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 3-3 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 3-4) or Figure 3-6 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 super elevation of the water surface profile at the bend using the equation:

$$\Delta d = \frac{v^2 T}{g R_c} \tag{3-14}$$

Where:

Δd	=	Super elevation 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.

3.4.10 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 the city of Memphis and/or Shelby County Engineer 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 3.4.9). Manning's n values for riprap can be calculated from the equation:

$$n = 0.0395\ (d_{50})^{1/6} \qquad (3\text{-}15)$$

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 3-10 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 3-11.

4. Determine the required minimum d_{30} value from Figure 3-12, which is based on the equation:

$$d_{30}/D = 0.193\ Fr^{2.5} \qquad (3\text{-}16)$$

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 3-11), 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:

$$W = 0.5236\ \gamma_s\ d^3 \qquad (3\text{-}17)$$

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 3-13 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 upstream and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

3.4.11 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 floodplain 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 floodplain.

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 floodplain 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:

$$K = \frac{1.49}{n} A R^{2/3} \qquad (3\text{-}18)$$

Where:

K	=	Channel conveyance
A	=	Cross-sectional area, in square feet
R	=	Hydraulic radius A/P, in feet
P	=	Wetted perimeter, in feet

$$Q = CS^{1/2} \tag{3-19}$$

Where:

Q	=	Discharge, in cfs
C	=	Constant
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.

3.4.12 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 3-6:

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 120 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 Bermuda grass mixture with a bottom slope less than 5 percent is 5 feet/second.

2. Assume an n value of 0.035 and find the value of vR from Figure 3-3 (using the Design Stability Curve).

   vR = 5.4

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

$$R = \frac{5.4}{5} = 1.08 \, feet$$

4. Use Equation 3-13 to calculate the value of vR:

$$vR = \frac{1.49(1.08)^{5/3}(0.015)^{1/2}}{(0.035)} = 5.93$$

5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed **n Value**	vR **(Figure 3-3)**	R **(Equation 3-11)**	vR **(Equation 3-12)**
0.035	5.4	1.08	5.93
0.037	4.1	0.82	3.54
0.038	3.8	0.72	3.04

Select n = 0.037 for stability criteria.

6. Use Figure 3-7 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

Qn = (120) (0.037) = 4.44
S = 0.015

For: b = 10 feet, d = (10) (0.15) = 1.5 feet
b = 8 feet, d = (8) (0.22) = 1.76 feet

Select: b = 10 feet, such that R is approximately 0.82 feet
z = 3 (from side slope 3:1)
d = 1.5 feet

$$V = \frac{1.49}{0.036}(.096)^{0.667}(0.015)^{0.5}$$

V = 9.3 feet/second

$$Fr = \frac{4.92}{(32.2)(1.5)^{0.5}}$$

Fr = 0.62

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 1.5 foot and calculate the following (see Figure 3-3):

A = (b + zd) d
A = [10+ (3) (1.5)] (1.5)
A = 21.8 square feet

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

$$R = \frac{[10 + (3)(1.5)](1.5)}{10 + (2)(1.5)\sqrt{1 + 3^2}}$$

R = 1.116 feet

2. Find the velocity:

v = 120/21.8
v = 5.52 feet/second

3. Find the value of vR:

vR = (5.52) (1.116) = 6.16

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

n = 0.037

5. Use Figure 3-6 or Equation 3-4 to find the velocity for S = 0.015, R = 1.116, and n = 0.037:

   v = 5.29 feet/second

6. Since 5.29 feet/second is less than 5.52 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)***
1.5	21.8	1.116	5.52	6016	0.037	5.29
1.6	23.7	1.177	5.07	5.96	0.0375	5.41
1.7	25.7	1.237	4.67	5.78	0.038	5.52

7. Select a depth of 1.6 with an n value of 0.035 for design capacity requirements. Add at least another 20% of design depth to the total depth for freeboard to give a total depth of 1.9 feet. Design data for the trapezoidal channel are summarized as follows:

   Vegetation lining = grass mixture, v_m = 5 feet/second
   Q = 120 cfs
   b = 10 feet, d = 1.6 feet, z = 3, S = 0.015
   Top width = (10) + (2) (3) (1.6) = 21.5 feet
   n (stability) = 0.037, d = 1.5 foot, v = 4.3 feet/second, Froude Number = 0.62
   n (capacity) = 0.0375, d = 1.6 feet, v = 5.41 feet/second, Froude Number = 0.75

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 3-10, 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 3-11 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 feet/second.

4. Determine minimum d_{30} from Figure 3-12 or Equation 3-16 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 3-13, 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 upstream 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 3-9.

3.5 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, developed by the U.S. Army Corps of Engineers (1998), and SWMM-Extran block, 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 floodplain. 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.

3.5.1 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 3.4.3)
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:

$$S_f = \frac{n^2 v^2}{2.22 R^{4/3}} \tag{3-20}$$

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.

Determine the difference between the bottom slope, S_o, and the average friction slope, S_f, from column 9 (not applicable for row 1). Record the result in column 10.

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

$$\Delta x = \frac{\Delta E}{S_o - S_f} = \frac{column\,7}{column\,10} \qquad (3\text{-}21)$$

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.

9. 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.

3.5.2 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 3.4.3)
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 3-20 and record the result in column 9.

9. Determine the average friction slope, 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, 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:

$$h_e = k_e \frac{v^2}{2g}$$

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

13. 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.

14. 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.

3.5.3 Example Problems

Example 3-5. Direct Step Method

Use the direct step method (Section 3.5.1) 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
α = 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 3-20, with n = 0.025, v as given in column 4, and R as given in column 3.

9. Average friction slope between the steps, 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, S_f, in column 9.

11. Length of the reach. Δx, in feet, between the consecutive steps, computed by Equation 3-21 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 3.5.2) 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 $\propto$ 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 3-20, 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.

3.6 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: Section 8.2.5. Bridge hydraulic design is detailed in Chapter 7. The hydraulic jump is presented below.

3.7 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 supercritical 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:

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

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 3-14. Additional details on evaluating hydraulic jumps can be found in publications by USDOT, FHWA (HEC-14, 1983), Chow (1959), Peterska (1978), and French (1985).

3.8 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 3.

3.9 Chapter Equations

Continuity Equation

$$Q = VA \quad (3\text{-}1)$$

Where:

Q	=	discharge, (ft^3/s)
A	=	cross sectional area of flow, (ft^2)
V	=	mean cross sectional velocity, (ft/s)

Flow Energy

$$E = d + V^2/2g \quad (3\text{-}2)$$

Where:

d	=	depth of flow, (ft)
V	=	mean cross-sectional velocity, (ft/s)
g	=	acceleration due to gravity, (32.2 ft/sec^2)

Manning's n Values

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

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

Uniform Flow Equation (open channel with steady uniform flow)

$$V = \left(\frac{1.486}{n}\right) R^{0.667} S^{0.5} \tag{3-4}$$

Where:

V	=	mean cross sectional velocity, (ft/s)
n	=	Manning's coefficient of channel roughness, (dimensionless)
A	=	cross sectional area of flow, (ft^2)
P	=	wetted perimeter (the cross section length touched by water), (ft)
R	=	hydraulic radius, (ft), (R = A/P)
S	=	energy grade line slope, (ft/ft)

Open-Channel Flow Rate

$$Q = \left(\frac{1.486}{n}\right) A R^{0.667} S^{0.5} \tag{3-5}$$

Where:

Q	=	Flow Rate (ft^3/sec)
n	=	Manning's coefficient of channel roughness, (dimensionless)
A	=	cross sectional area of flow, (ft^2)
P	=	wetted perimeter (the cross section length touched by water), (ft)
R	=	hydraulic radius, (ft), (R = A/P)
S	=	energy grade line slope, (ft/ft)

Energy Grade Line Slope of an Open Channel Flow Rate

$$S = \left(\frac{Qn}{1.49 A R^{2/3}}\right)^2 \tag{3-6}$$

Where:

S	=	energy grade line slope, (ft/ft)
Q	=	Flow Rate (ft^3/sec)
n	=	Manning's coefficient of channel roughness, (dimensionless)
A	=	cross sectional area of flow, (ft^2)
P	=	wetted perimeter (the cross section length touched by water), (ft)
R	=	hydraulic radius, (ft), (R = A/P)

Trial-and Error-Solutions

$$AR^{2/3} = \frac{Qn}{1.49S^{1/2}} \tag{3-7}$$

Where:

A	=	Cross-sectional area, (ft^2)
R	=	Hydraulic radius, (ft)
Q	=	Discharge rate for design conditions, (ft^3/sec)
n	=	Manning's roughness coefficient, (dimensionless)
S	=	Slope of the energy grade line, (ft/ft)

The Trapezoidal Conveyance Factor

$$K_T = \frac{Qn}{b^{8/3}S^{1/2}} \tag{3-8}$$

Where:

K_T	=	rapezoidal open channel conveyance factor
Q	=	Discharge rate for design conditions, (ft3/sec)
n	=	Manning's roughness coefficient, (dimensionless)
b	=	Bottom width, (ft)
S	=	Slope of the energy grade line, (ft/ft)

Critical Depth

$$\frac{Q^2}{g} = \frac{A^3}{T} \tag{3-9}$$

Where:

Q	=	Discharge rate for design conditions, (ft3/sec)
g	=	Acceleration due to gravity, (32.2 ft/sec2)
A	=	Cross-sectional area, (ft2)
T	=	Top width of water surface, (ft)

Critical Depth With The Critical Flow Section Factor, Z

$$Z = Q/\sqrt{g} \tag{3-10}$$

Where:

Z	=	Critical flow section factor
Q	=	Discharge rate for design conditions, (ft3/sec)
g	=	Acceleration due to gravity, (32.2 ft/sec2)

Froude Number

$$Fr = V/(gd)^{0.5} \tag{3-11}$$

Where:

Fr	=	Froude Number
V	=	mean cross sectional velocity, (ft/s)
g	=	acceleration due to gravity, (32.2 ft/sec^2)
D	=	hydraulic depth*, (ft), (D= A/T)
A	=	cross sectional area of flow, (ft^2)
T	=	channel top width at the water surface, (ft)

* For rectangular channels, hydraulic depth equals depth of flow.

Hydraulic Radius

$$R = \frac{(vR)}{V_m} \tag{3-12}$$

Where:

R	=	Hydraulic radius of flow, in feet
νR	=	Value obtained from Figure 3-3 in Step 3
V_m	=	Maximum velocity from Step 2

A form of Manning's Equation to calculate the value of vR

$$vR = \frac{1.49 R^{5/3} S^{1/2}}{n} \qquad (3\text{-}13)$$

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

Super Elevation of the Water Surface

$$\Delta d = \frac{v^2 T}{g R_c} \qquad (3\text{-}14)$$

Where:

Δd	=	Super elevation 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.

Manning's n Values for Riprap

$$n = 0.0395\,(d_{50})^{1/6} \qquad (3\text{-}15)$$

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

Required Minimum d_{30} Value

$$d_{30}/D = 0.193\ Fr^{2.5} \quad (3\text{-}16)$$

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 3-11), dimensionless
- v = Mean velocity above the stone, in feet/second
- g = Acceleration of gravity, 32.2 feet/second^2

Stone Weight for a Selected Stone Size

$$W = 0.5236\ \gamma_s\ d^3 \quad (3\text{-}17)$$

Where:

- W = Stone weight, in pounds
- d = Selected stone diameter, in feet
- γ_s = Specific weight of stone, in pounds/cubic foot

Manning's Equation, Conveyance (allowable flood elevation)

$$K = \frac{1.49}{n} AR^{2/3} \quad (3\text{-}18)$$

Where:

- K = Channel conveyance
- A = Cross-sectional area, in square feet
- R = Hydraulic radius A/P, in feet
- P = Wetted perimeter, in feet

$$Q = CS^{1/2} \quad (3\text{-}19)$$

Where:

- Q = Discharge, in cfs
- C = Constant
- S = Slope of the energy grade line, in feet/foot

Friction Slope

$$S_f = \frac{n^2 v^2}{2.22 R^{4/3}} \qquad (3\text{-}20)$$

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

Length of Channel Between Consecutive Rows of Depths of Flow

$$\Delta x = \frac{\Delta E}{S_o - S_f} = \frac{column\,7}{column\,10} \qquad (3\text{-}21)$$

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.

Relationship between Variable s for a Hydraulic Jump in Rectangular Sections

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

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

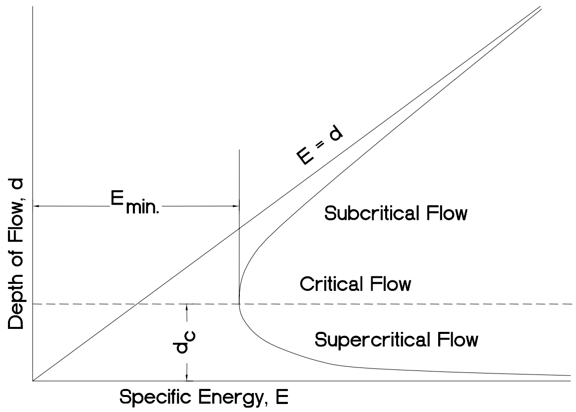

FIGURE 3-1
Specific Energy vs. Depth of Flow

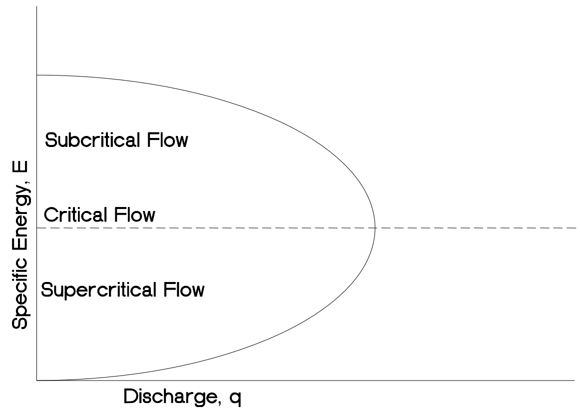

FIGURE 3-2
Specific Energy vs. Discharge

.

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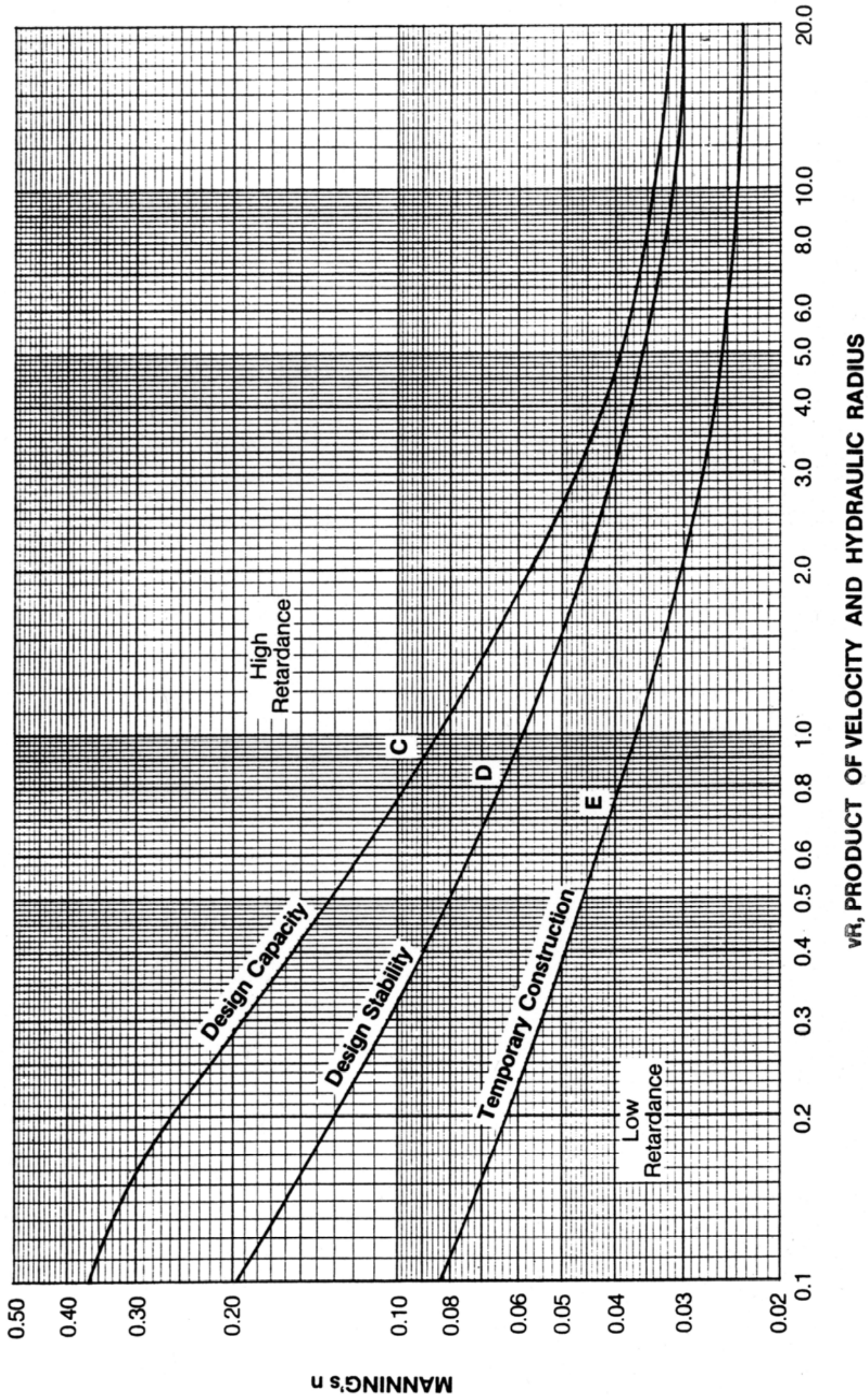

Reference: USDA, TP-61 (1947).

Figure 3-3
Manning's n Value for Vegetated Channels
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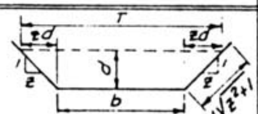
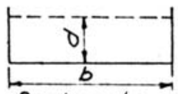
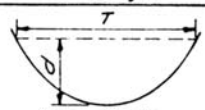
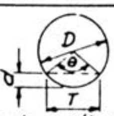
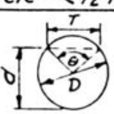
Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
 Trapezoid	$bd + zd^2$	$b + 2d\sqrt{z^2+1}$	$\frac{bd + zd^2}{b + 2d\sqrt{z^2+1}}$	$b + 2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
 Rectangle	bd	$b + 2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2} zd^{2.5}$
 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 $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D \sin \frac{\theta}{2}}}$

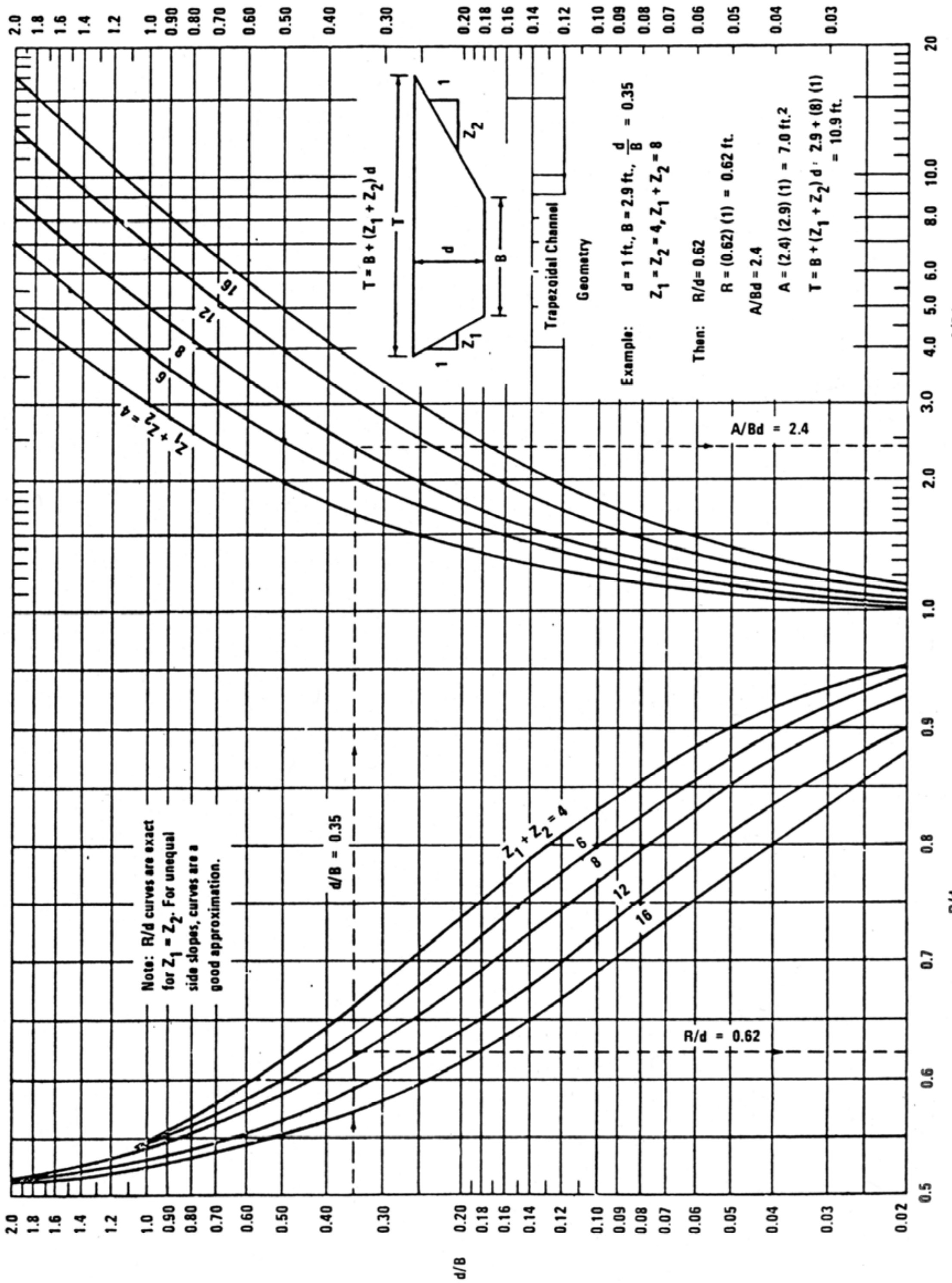
[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d} \sinh^{-1} \frac{4d}{T}$

[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

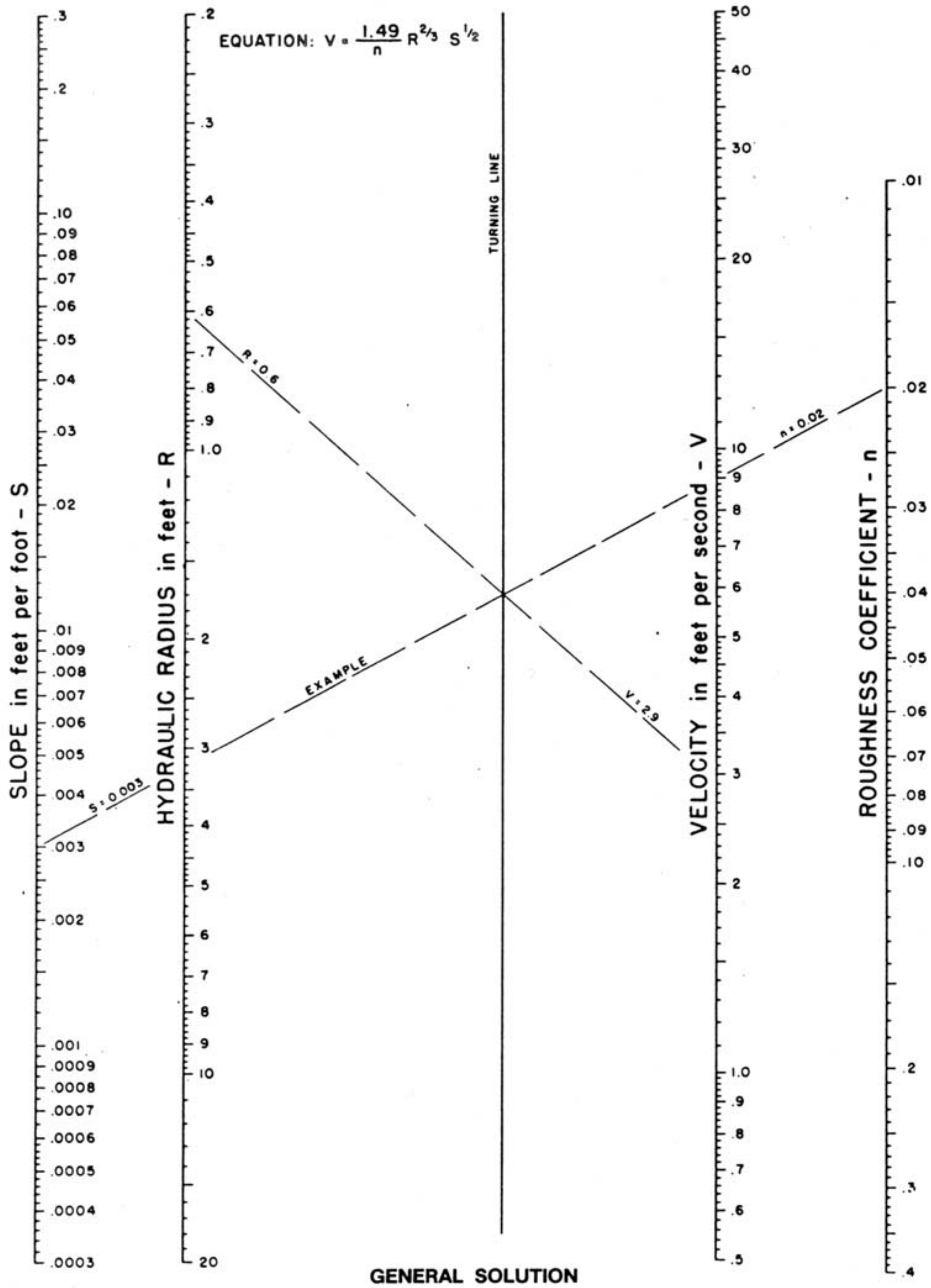
Reference: USDA, SCS, NEH-5 (1956).

Figure 3-4
Open Channel Geometric Relationships for Various Cross Sections
Back to Section 3.4.5, Section 3.4.8



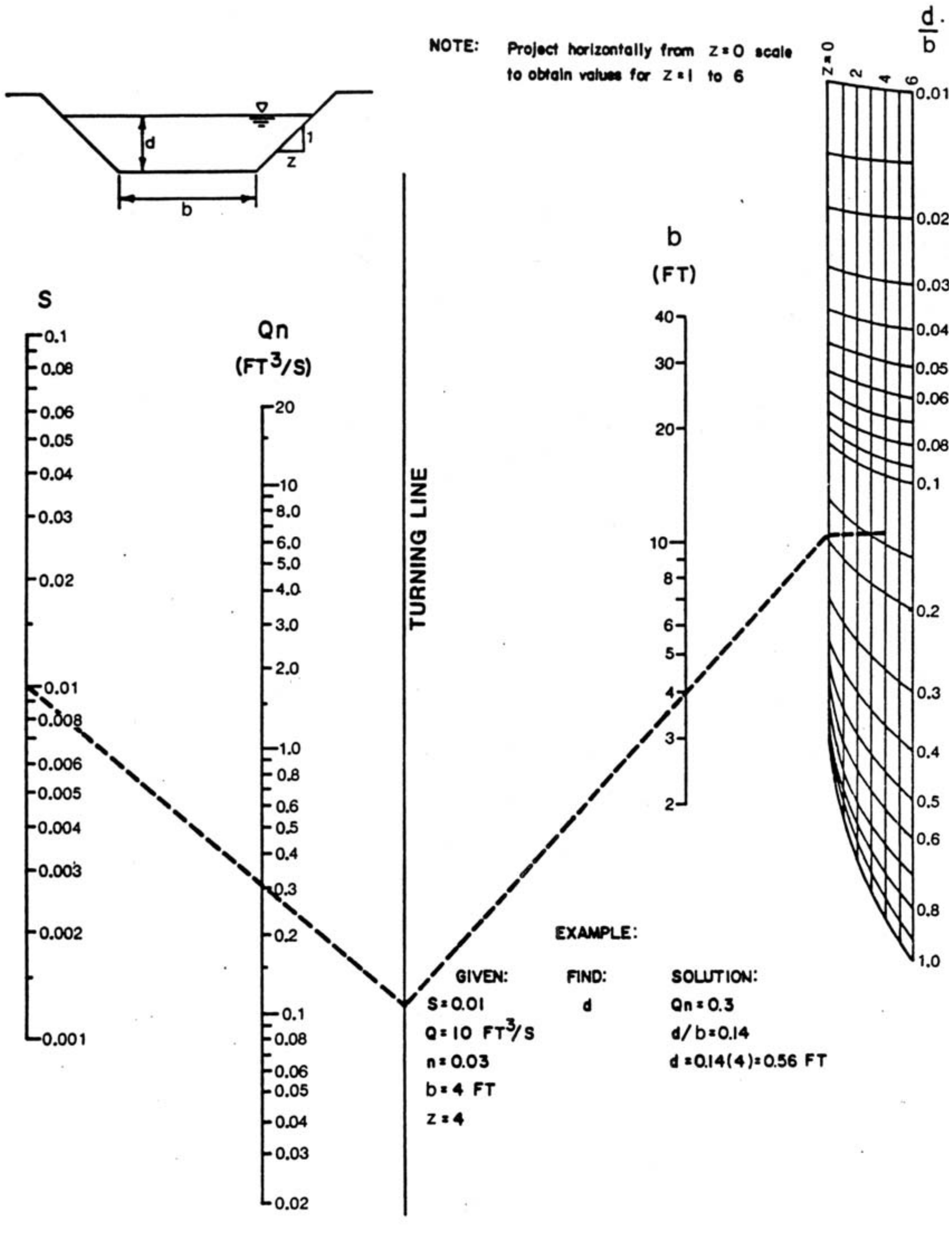
Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3-5
Trapezoidal Channel Geometry
Back to Section 3.4.5



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

Figure 3-6
Nomograph for the Solution of Manning's Equation
Back to Section 3.4.6



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

Figure 3-7
Solution of Manning's Equation for Trapezoidal Channels
Back to Section 3.4.6

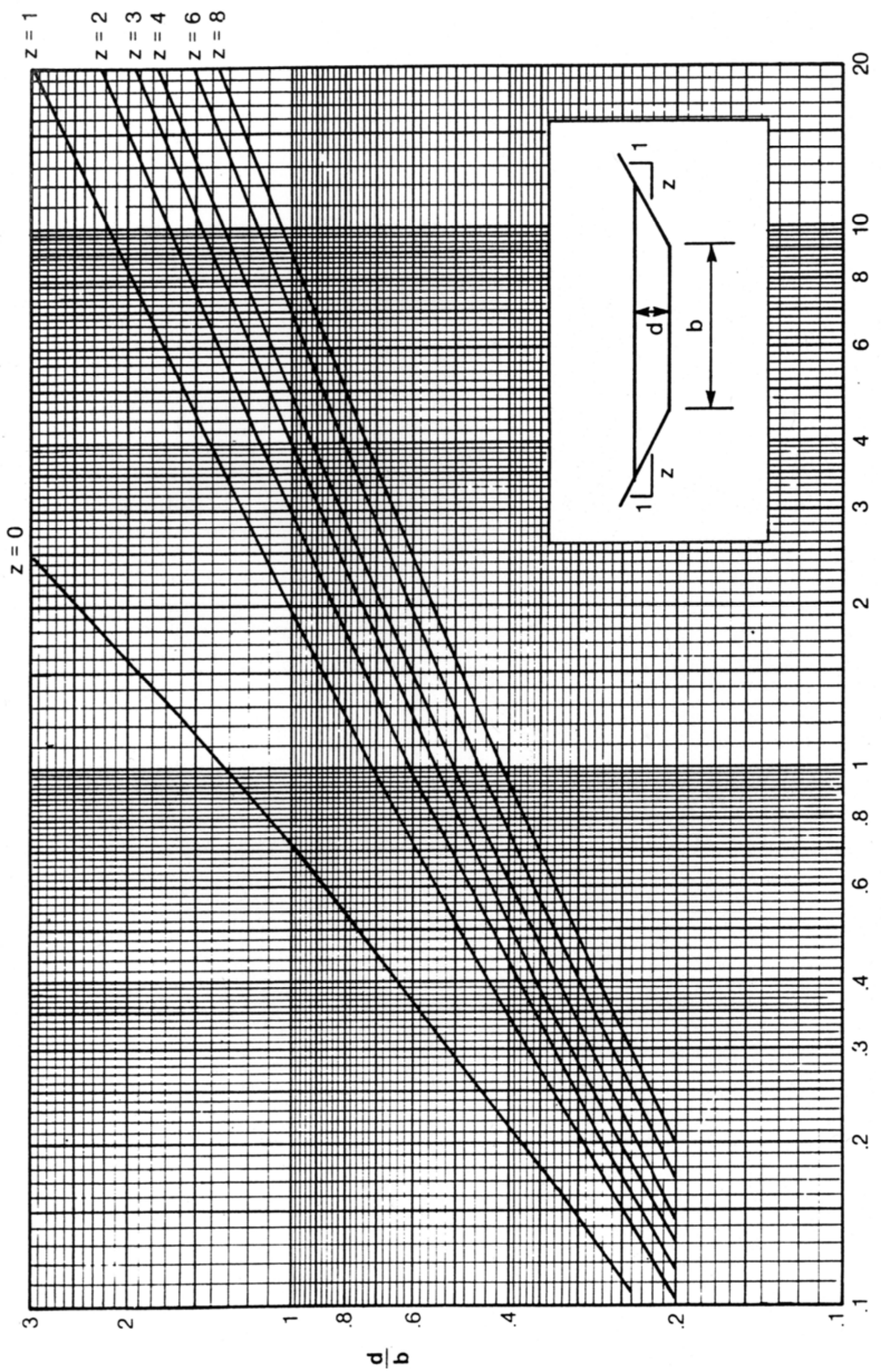
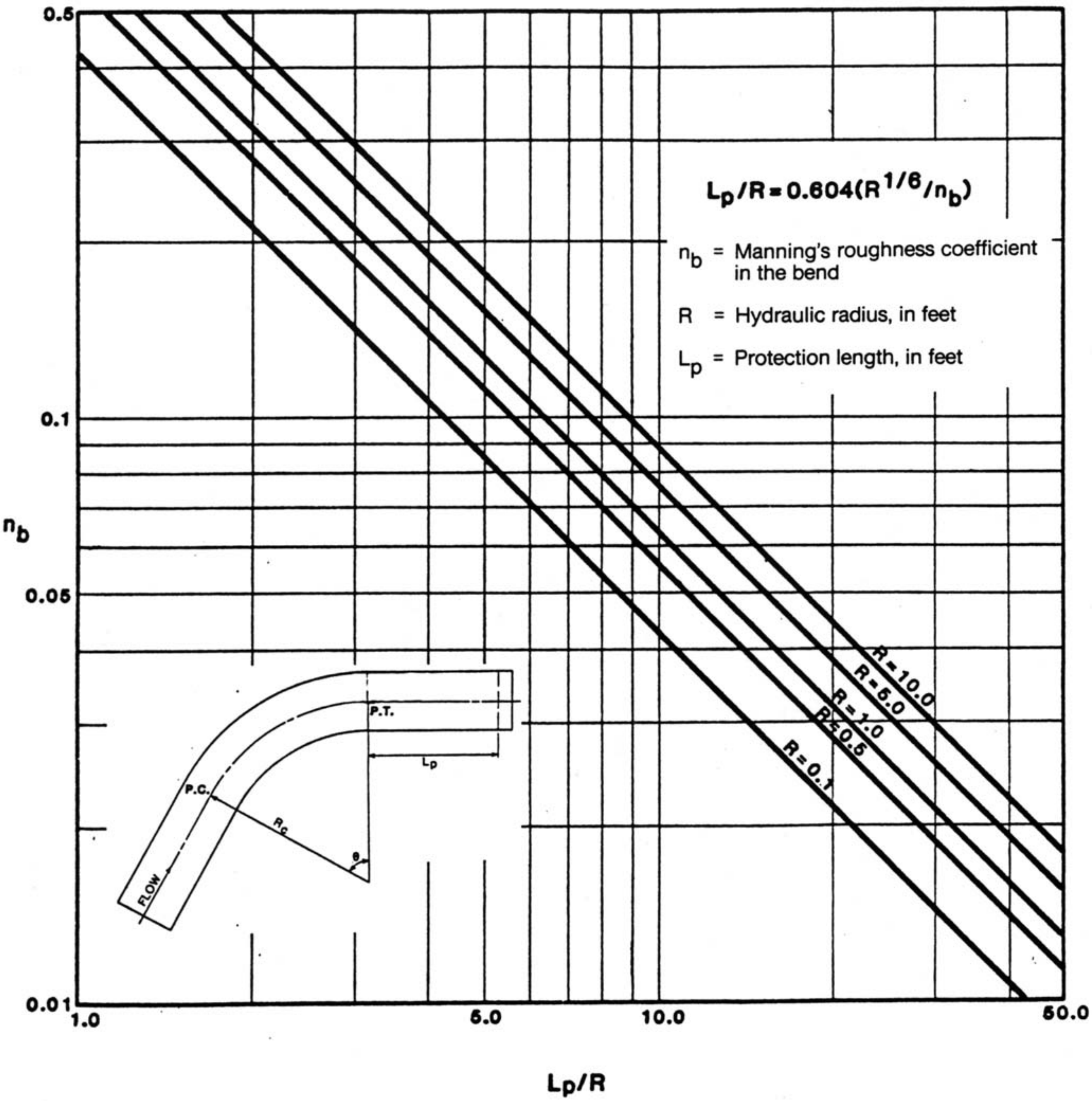


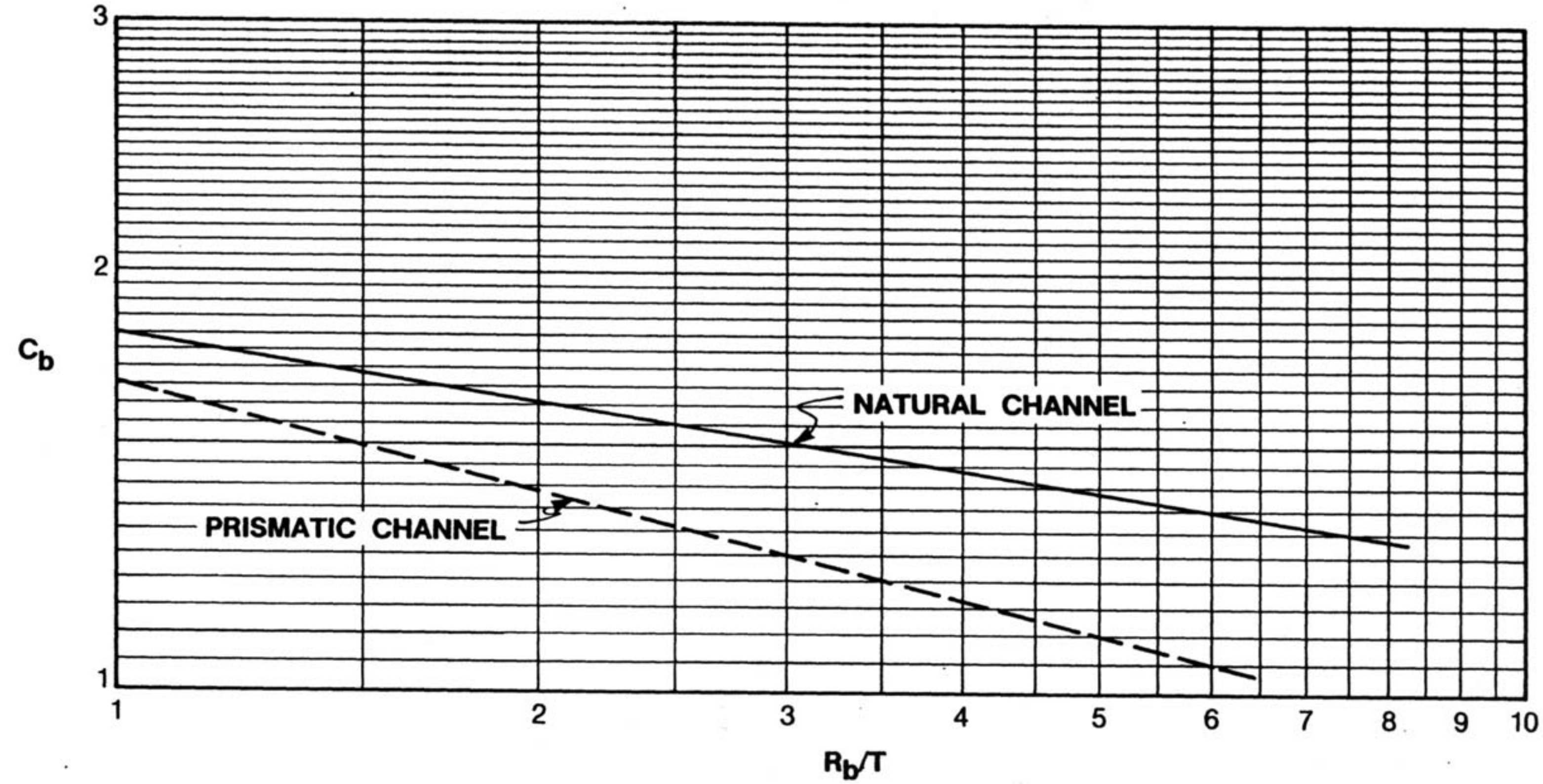
Figure 3-8
Trapezoidal Channel Capacity Chart
Back to Section 3.4.7



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

Figure 3-9
Protection Length, L_p, Downstream of Channel Bend

Back to Section 3.4.9

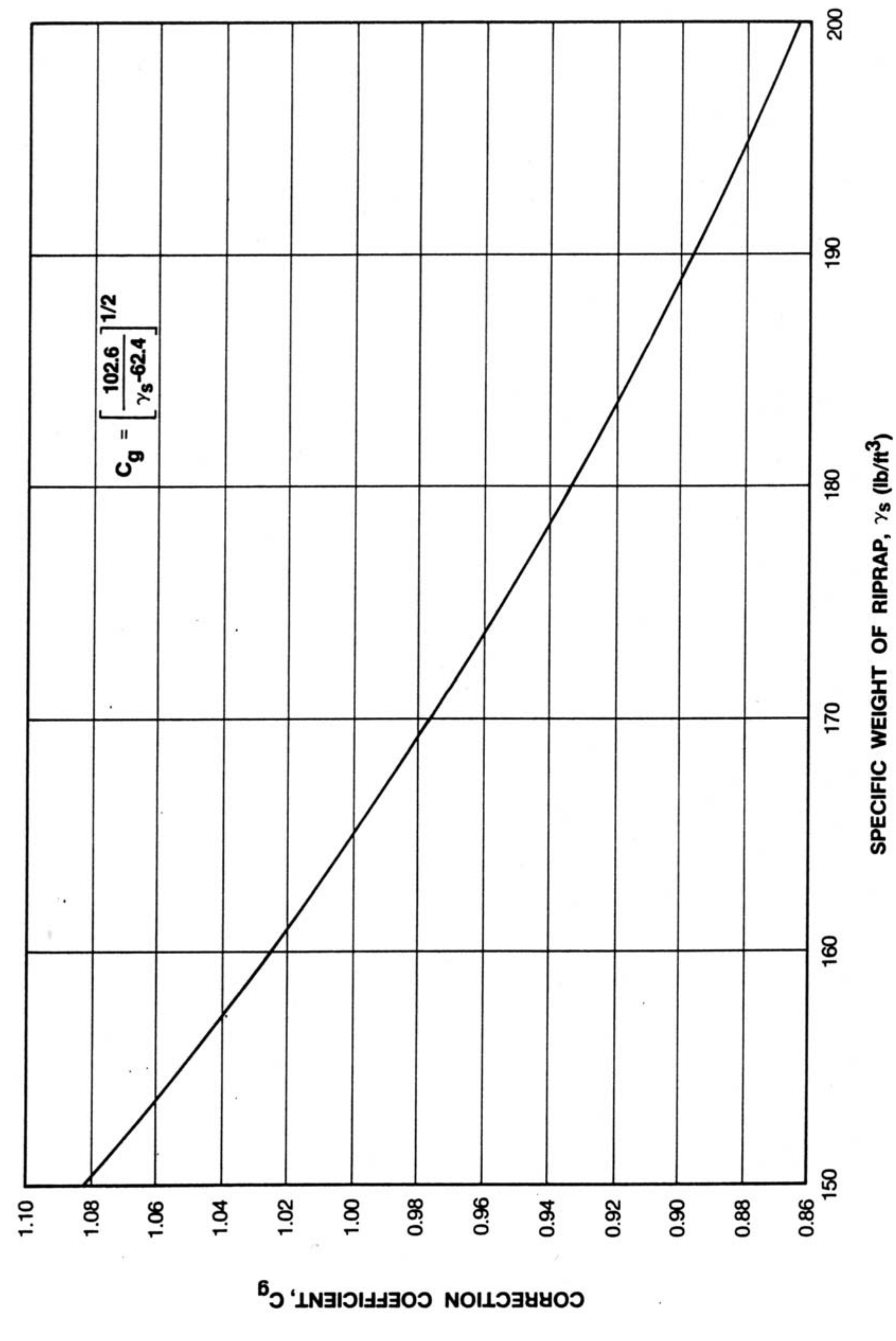


To obtain effective velocity, multiply known velocity by C_b.

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

Reference: Maynord (1987).

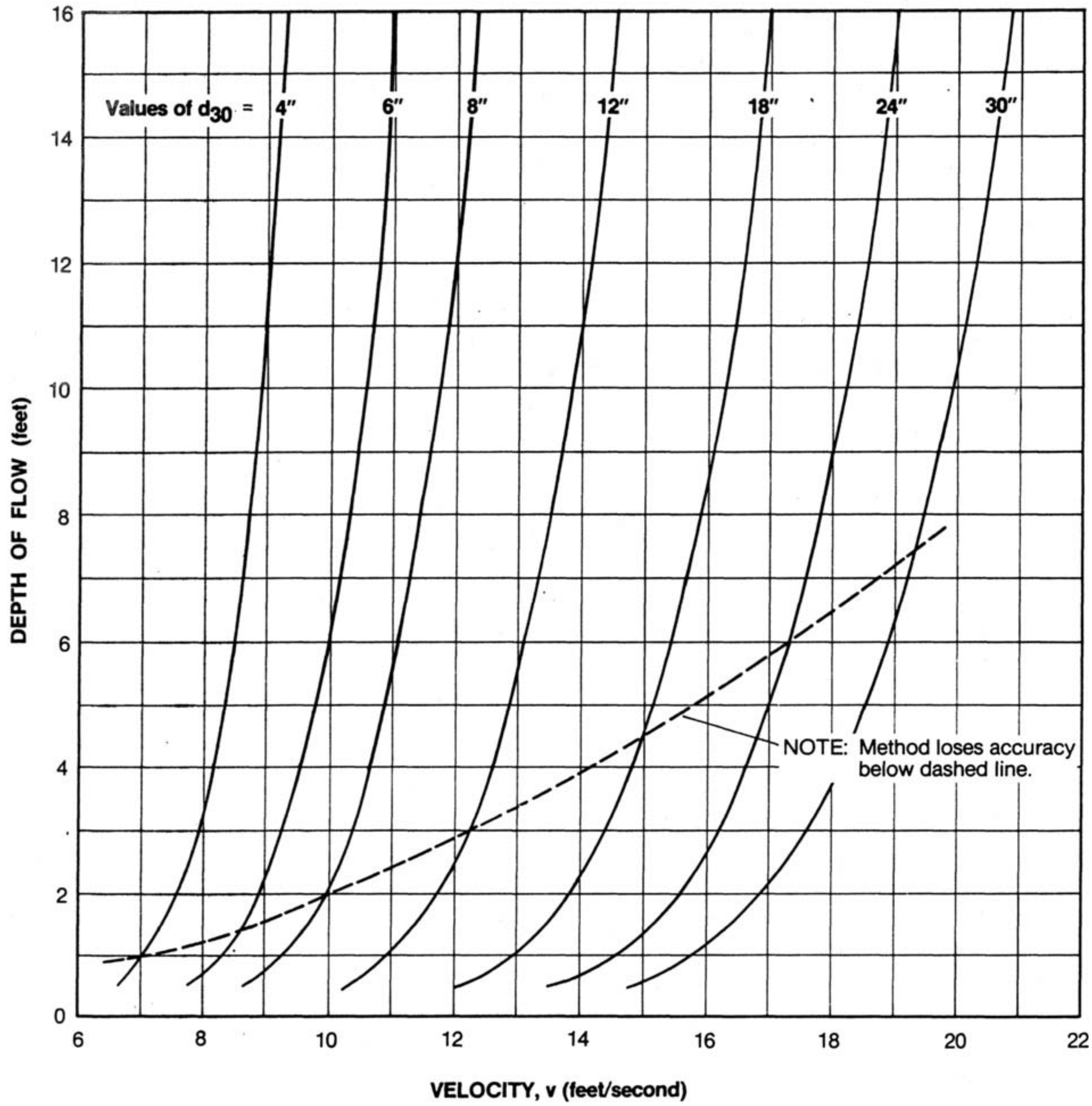
Figure 3-10
Riprap Lining Bend Correction Coefficient
Back to Section 3.4.10



C_g = Correction Coefficient

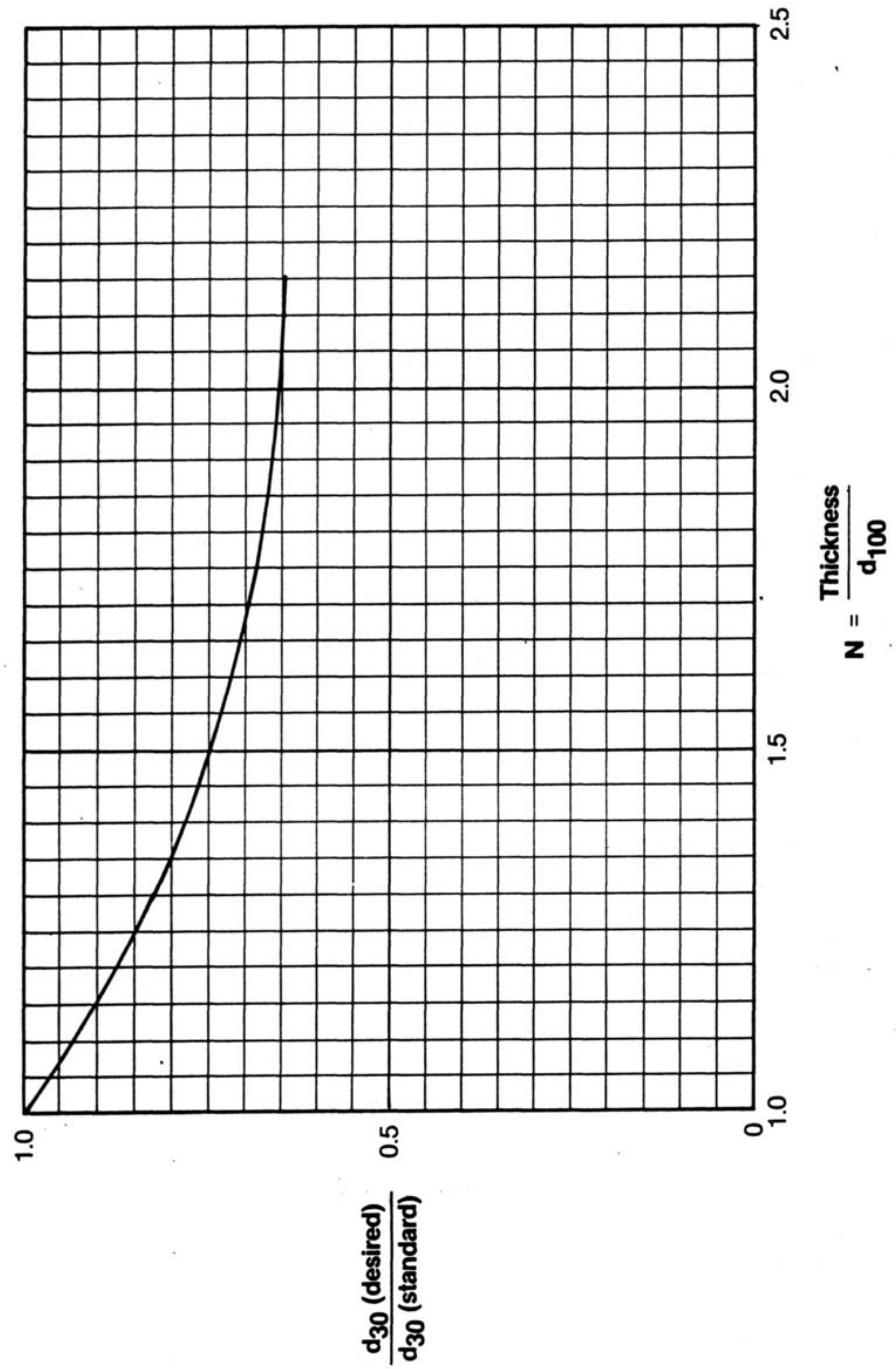
To obtain effective velocity, multiply known velocity by C_g.

Figure 3-11
Riprap Lining Specific Weight Correction Coefficient
Back to Section 3.4.10



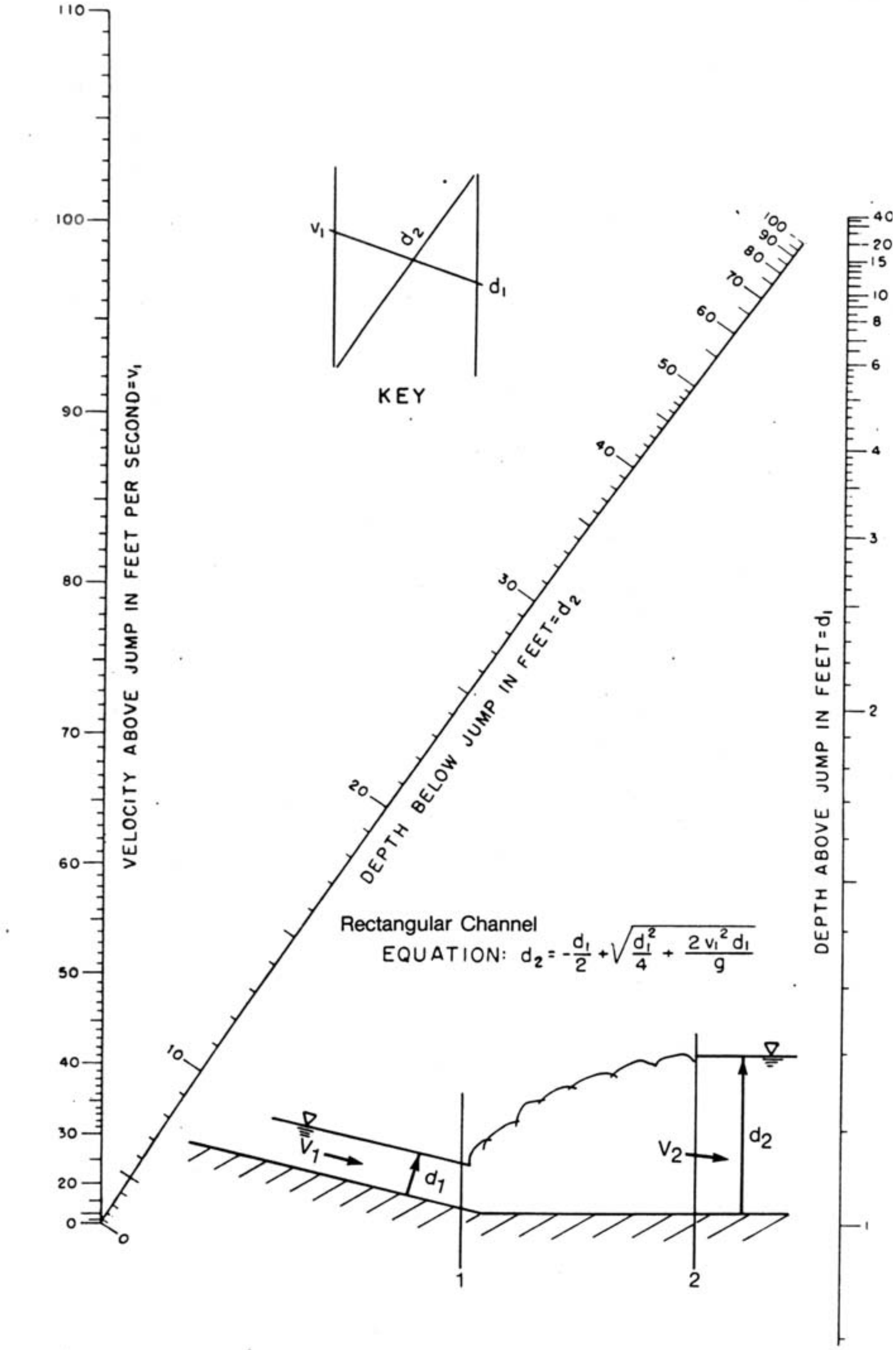
Reference: Reese (1988).

Figure 3-12
Riprap Lining d_{30} Stone Size as a Function
of Mean Velocity and Depth
Back to Section 3.4.10



Reference: Maynord (1987).

Figure 3-13
Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 2.0 to 2.3
Back to Section 3.4.10



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

Figure 3-14
Nomograph for Solving the Rectangular Channel Hydraulic Jump Equation
Back to Section 3.7

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

Note:
[a] = Higher velocities may be acceptable for rigid linings if appropriate protection is provided (see Chapters 9 and 10)
Back to Section 3.3.5

Table 3-2
Maximum Velocities for Vegetative Channel Linings

Vegetation Type	Slope Range (%)	Maximum Velocity [a] (feet per second)
Bermudagrass	0-5	5
	5-10	4
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

Note:
[a] = Based on erosive soils

Reference: USDA, TP-61 (1947)
Back to Section 3.3.5

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	0.015	0.015
	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		

Notes:
[a] = n values for vegetative linings should be determined using Figure 3-1
[b] = See Section 3.4.10

Reference: USDOT, FHWA, HEC-15 (1986)
Back to Section 3.4.3

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)
Back to Section 3.4.3, 3.4.9

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

Notes:
[a] = Cowan's Equation is presented as Equation 3-3
[b] = From Chow (1959), Table 5-5, page 109
Back to Section 3.4.3

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]	$d_c = 0.81 \left(\frac{Q^2}{gz^{0.75} \; b^{1.25}} \right)^{0.27} - \frac{b}{30z}$	< 0.5522 Q <0.4 b 2.5 For 0.5522 Q < 0.1, b 2.5 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

Notes:
[a] = See Figure 3-4 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)

Back to Section 3.4.8

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.											

Notes:

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

Back to Section 3.5.1

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)**	**$\bar{S}_f$ (9)**	**$\bar{S}_f$ (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.													

Notes:

(9) $S_f = \frac{n^2 v^2}{2.22 R^{4/3}}$ (12) $h_f = \Delta x \; \bar{S}_f$ (13) $h_e = k_e \frac{v^2}{2_g}$

Back to Section 3.5.2

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)	$\bar{S}_f$ (9)	$S_o - \bar{S}_f$ (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

Notes:
Q = 400 cfs   n = 0.025 S_o = 0.0016 α = 1.10 y_c = 2.22 ft. y_n = 3.36 ft.

Reference: Chow (1959)

Back to Section 3.5.3

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)	$\bar{S}_f$ (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

Notes:
Q = 400 cfs   n = 0.025 S_o = 0.0016 α = 1.10 h_e = 0 y_c = 2.22 ft. y_n = 3.36 ft.

Reference: Chow (1959)

Back to Section 3.5.3

City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 4: Gutter and Hydraulics

Volume 3: Best Management Practices Manual

Revision: 0
June 2006

EnSafe Inc.
5724 Summer Trees Drive
Memphis, Tennessee 38134
(901) 372-7962
www.ensafe.com

Acronym List
(Chapter 4)

a	Gutter depression
C	Weir coefficient, use 3.0 unless otherwise approved by City/County Engineer
cfs	Cubic feet per second
d	Depth of flow, (ft)
Eo	Gutter Flow Ratio
FHWA	Federal Highway Administration
H_m	Minimum height
MCC	Municipal Code Corporation
n	Manning's roughness coefficient
P	Perimeter of grate opening, (ft)
Q	Gutter flow rate
Q	Gutter flow upgradient (upstream) of inlet ($Q_{upstream}$), (cfs)
Q_i	Gutter flow intercepted ($Q - Q_{downstream}$), (cfs)
Q_i	Rate of discharge into grate opening, (cfs)
Q_p	Peak Runoff
S	Longitudinal street grade, (ft/ft)
S_o	Longitudinal slope
S_W	Gutter cross slope, (ft/ft)
S_x	Pavement cross slope, (ft/ft)
T	Width of flow or spread, (ft)
TDOT	Tennessee Department of Transportation
USDOT	United States Department of Transportation
V	Average velocity
W	Width of gutter
Z_C	Reciprocal of composite pavement cross slope ($1/S_c$), (ft/ft)
Z_W	Reciprocal of gutter cross slope ($1/S_W$), (ft/ft)
Z_x	Reciprocal of pavement cross slope ($1/S_x$), (ft/ft)

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4.0 GUTTER AND INLET HYDRAULICS

4.1 Introduction

This chapter describes the criteria and procedures used in the design of roadway storm drainage systems for the City of Memphis and Shelby County. To properly utilize this information, the designer should already be familiar with the hydraulic behavior of gutter and inlets and understand the basic concepts related to analyzing their hydraulic performance. The primary goal of the design parameters in this chapter is to limit the depth and spread of water in roadways, so at a minimum, the system will pass the 10-year 24-hour storm event runoff. In addition, roadway drainage systems shall provide that, during the 100-year 24-hour storm event, spread of water shall:

1. Be limited so that no more than one traffic lane is encumbered in either direction for arterial roadway

2. Must leave at least one lane free of water in each direction for collectors

3. Shall be limited as to maintain a minimum of one passable (8-foot) lane for local roads

See Table 4-1 for details.

The design standards and requirements outlined in this chapter represent a combination of standards set forth in the City of Memphis and Shelby County Division of Planning and Development's currently approved Subdivision Regulation (regulations for public streets are found in Section 404, Appendix B, Code of Shelby County) and design practices/standards required by the City of Memphis Division of Engineering.

4.1.1 Plans

The design engineer will submit all plans for approval by the City and/or County Engineer. Plans will include streets, bridges, culverts, drainage structures, and storm sewers for planned developments. Plans are to be submitted in accordance with the currently approved Subdivision Regulation (Appendix B, Code of Shelby County) and the City of Memphis Division of Engineering Design and Review Policy Manual.

4.1.2 Other Design Sources

While the guidance provided in this chapter is to be used for storm drainage design, it is not all-encompassing. There may be instances in which the designer may wish to consult an outside reference to address a particular design issue. Outside references include the

Tennessee Department of Transportation (TDOT) Design Division's Design Guidelines, Traffic Design Manual and Drainage Manual, Federal Highway Administration (FHWA) publications HEC-12 Drainage of Highway Pavements (USDOT, FHWA, 1984), HEC-21 Bridge Deck Drainage Systems (USDOT, FHWA, 1993), and HEC-22 Urban Drainage Design Manual (USDOT, FHWA, 2001).

4.1.3 Web Sites of Interest

City of Memphis and Shelby County Division of Planning and Development:

http://www.dpdgov.com

This Web site includes information on regulatory policies to be followed when designing planned developments in the City of Memphis and Shelby County. Contact information for the City of Memphis and Shelby County Division of Planning and Development can be found here as well.

Municipal Code Corporation (MCC):

http://www.municode.com

Municipal codes for many cities and counties in the United States can be found on this site. The Code of Shelby County can be viewed electronically on this Web site. The "Subdivision Regulation" for Memphis and Shelby County is found as Appendix B in the Code of Shelby County, with guidelines for public streets being found in Section 404.

City of Memphis Division of Engineering:

http://www.cityofmemphis.org/framework.aspx?page=18

Standard drawings for design of streets, culverts, drainage structures, storm sewers, and other general construction-related activities can be found on this site. Standard construction specifications and the Design and Review Policy Manual are also available. Contact information for the City of Memphis Division of Engineering can be located here as well.

Tennessee Department of Transportation Design Division:

http://www.tdot.state.tn.us/Chief_Engineer/assistant_engineer_design/design/index.htm

Useful publications such as TDOT's Design Guidelines, Traffic Design Manual, and Drainage Manual can be downloaded from this site.

Federal Highway Administration:

http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

An extensive library of publications addressing a wide variety of roadway-related drainage design subject areas is available for download at this site.

4.2 Design Criteria

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

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

4.2.1 Return Period

The design-storm return period for pavement drainage should be consistent with the frequency selected for other components of the drainage system. See Chapter 2 for details on return period and associated rainfall volumes and rates.

4.2.2 Classification of Streets

Streets are classified as defined in Section 404.2, Memphis and Shelby County Subdivision Regulation (Appendix B Code of Shelby County). There are three basic classifications of streets outlined in Section 404.2: Arterials, collectors, and locals. Each of these street classifications has sub-classifications, which are also described in Section 404.2. The classification of a street dictates criteria for design-storm return period, physical dimensions of the street, what types of drainage structures are available for use in design, and allowable flow-spread encroachment into the street. Table 4-1 briefly outlines different types of streets used in Memphis and Shelby County and various characteristics associated with varying sections.

4.2.3 Spread

For multi-lane 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). Flow spread encroachment for roadways in the City of Memphis and Shelby County is limited based on street classification described in Section 4.1.2:

1) Arterials: The permissible spread of water in gutters of arterials shall be limited so that no more than one traffic lane is encumbered on each side.

2) Collectors:

   a) The permissible spread of water in gutters of major collector streets shall be limited so 10 feet of pavement width in each direction will remain clear.

   b) The permissible spread of water in gutters of minor collector streets shall be limited so that 8 feet of pavement width will remain clear.

3) Locals:

   a) The permissible spread of water in gutters for major local streets shall be limited to allow 8 feet of pavement width to remain clear.

   b) The permissible spread of water in gutters for minor local streets shall be limited to allow 4 feet of pavement width to remain clear.

A summary of spread requirements can be found in Table 4-1. Figures 4-1 through 4-4 use standard street and gutter design cross slopes to define curb depth in inches as a function of spread of water from the face of the curb for a 6-30 curb and gutter or from the flow line for valley gutter. Velocity of flows for all curb and gutter configurations shall not exceed 10 feet per second.

In sag situations, the spread of water into the street is to be checked at the inlet as well as 50 feet upgradient (upstream) of the inlet in both directions to determine the maximum spread of water in the street.

4.2.4 Inlet Types and Spacing

A storm water inlet is an opening into a storm sewer system for the entrance of surface runoff. The City of Memphis utilizes three standard types of storm inlets:

1) Curb Opening Inlets
2) Grate Inlets (No. 10, No. 11, No. S-11, and Valley Gutter Inlets)
3) Combination Curb Opening and Grate (6-72 and 6-72M)

Civil standard drawings for approved inlets can be found on the City of Memphis Division of Engineering's Web site: http://www.cityofmemphis.org/framework.aspx?page=18. Other more efficient types of inlets may be used with the approval of the City and/or County Engineer.

Inlets shall be located or spaced in such a manner that the design curb flow does not exceed the spread criterion outlined in Section 4.2.3. With the maximum spread fixed and with a given pavement cross slope and longitudinal slope, the flow in the gutter and street is also fixed and can be calculated as explained in Section 4.2. The spacing of the inlets on a continuous grade is related to the drainage area needed to generate the discharge corresponding to the allowable spread on the pavement. The flow by-passing each inlet must be included in the flow arriving at the next inlet. Inlets shall be placed at all low points in the gutter grade.

Where a curbed roadway crosses a bridge, the gutter flow should be intercepted and not be permitted to flow onto the bridge.

At intersections, inlets shall be placed and sized so that encroachment into the intersection is no greater than that allowed on the street for the design storm.

4.2.5 Manning's n Values

Manning's n values for various pavement surfaces are presented in Table 4-2. A Manning's n value of 0.016 will be assumed for example design calculations and figures developed for this chapter.

4.2.6 Grade

Guidelines for minimum and maximum longitudinal curb and gutter grades are equal to those of pavement slopes outlined in Section 404.11 of the Memphis and Shelby County Subdivision Regulation. The minimum longitudinal grade for a public street is 0.4 percent except that cul-de-sacs without a drainage inlet at or near the cul-de-sac terminus and streets on fill shall have a 0.5 percent minimum longitudinal grade. The maximum longitudinal grade for the various street types are shown below:

Street Classification	Maximum Longitudinal Grade Percentage
Arterials	6%
Collector	10%
Local	12%

A minimum longitudinal gradient is more important for curbed pavements, which are susceptible to storm water spread. Flat longitudinal gradients on uncurbed pavements can lead to a spread problem if vegetation is allowed to persist along the pavement edge or debris is allowed to build up in or near the gutter.

4.2.7 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 design situations in Memphis and Shelby County, cross slopes will be defined by the standard pavement sections as defined in City of Memphis Design Division Civil Standard Drawings numbers 30 and 31. Cross slopes utilized in the City of Memphis and Shelby County are 3/16 inch (0.0156 feet per foot) and 1/4 inch (0.0208 feet per foot). Table 4-1 outlines acceptable cross slopes for various street sections used in Memphis and Shelby County. Special situations may require variation from use of standard cross slopes (i.e., super-elevated sections); theses variations require approval by the City and/or County Engineer.

4.2.8 Curb and Gutter Sections

Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities and is required within all urban subdivisions. Three basic types of curb and gutter are utilized in the City of Memphis and Shelby County:

1) 6-30 curb and gutter
2) 6-18 curb and gutter
3) Valley gutter

Curb and curb and gutter details are presented in the City of Memphis Design Division's Civil Standard Drawings numbers 1, 2, and 3. Typical sections of various approved road widths and recommended curb and gutter usage can be found in the City of Memphis and Shelby County Subdivision Regulation. Table 4-1 outlines acceptable curb and gutter applications for various street sections used in Memphis and Shelby County.

Memphis and Shelby County encourage the use of curb cuts allowing runoff from parking areas to be routed to biofilter swales or biofilter strips. Biofilters and this approach are discussed and illustrated in Volume 4 PTP-05.

4.2.9 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.

4.2.10 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 constructability usually requires a constant cross slope, so the guidelines in Section 4.2.7 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.

4.3 Gutter Flow Calculations

Gutter flow calculations are necessary to establish the spread of water encroaching on the roadway surface. Illustrations of typical gutter sections can be found on Figure 4-5. 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	=	Longitudinal street grade, in feet/foot
T	=	Width of flow or spread, in feet

Equation 4-1 utilizes width of flow, or spread in solving this modified Manning’s equation. It is also possible to utilize depth of flow at the face of curb to solve for gutter flow using the following relationship:

$$T = \frac{d}{S_x} \qquad (4\text{-}2)$$

Where:

T	=	Width of flow or spread, in feet
d	=	Depth of flow, in feet
S_x	=	Pavement cross slope, in feet/foot

When Equation 4-2 is substituted into Equation 4-1, the result is the following version of Manning's equation, which relates depth of flow to flow rate:

$$Q = \frac{0.56}{n}\left(\frac{1}{S_x}\right) S^{1/2}\, d^{8/3} \qquad (4\text{-}3)$$

let $$Z_x = \left(\frac{1}{S_x}\right) \qquad (4\text{-}4)$$

substituting,

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

Where:

Q	=	Gutter flow rate, in cfs
n	=	Manning's roughness coefficient
Z_x	=	Reciprocal of pavement cross slope ($1/S_x$), in feet/foot
S	=	Longitudinal street grade, in feet/foot
d	=	Depth of flow, in feet

4.3.1 Composite Gutter Sections Conditional Procedures

Composite cross slope situations (unequal gutter and pavement cross slope) require the calculation of a composite Z for use in Equation 4-5. Width of flow, T, for a composite section must first be calculated. For composite gutter sections, width of flow can be calculated in the following manner:

Depth of flow in composite sections can be expressed mathematically as:

$$d = S_W W + S_x (12T - W) \qquad (4\text{-}6)$$

Isolating T, width of flow (spread) for composite sections is expressed as:

$$T = \frac{\left(W + \frac{(d - W * S_W)}{S_x}\right)}{12} \qquad (4\text{-}7)$$

Where:

- T = Width of flow or spread, in feet
- W = Width of gutter, in inches
- d = Depth of flow, in inches
- S_W = Gutter cross slope, in feet/foot
- S_x = Pavement cross slope, in feet/foot

Composite Z for unequal cross slopes can then be expressed by the following equation:

$$Z_C = Z_W \left(1 + \left(\frac{Z_x}{Z_W} - 1 \right) \left(\frac{T - W}{\left(T + W\left(\frac{Z_x}{Z_W} - 1 \right) \right)} \right)^{\left(8/3\right)} \right) \qquad (4\text{-}8)$$

Where:

- Z_C = Reciprocal of composite pavement cross slope ($1/S_c$), in feet/foot
- Z_W = Reciprocal of gutter cross slope ($1/S_W$), in feet/foot
- Z_x = Reciprocal of pavement cross slope ($1/S_x$), in feet/foot
- T = Width of flow or spread, in feet
- W = Width of gutter, in feet

A nomograph for solving Equation 4-5 is presented in Figure 4-6. Using a composite Z in Equation 4-5 enables use of the nomograph in Figure 4-6 to solve for all gutter configurations and straight cross slopes.

Figures 4-1 through 4-4 illustrate the top width and area of flow as a function of depth in the gutter, and the discharge as a function of depth in the gutter and street slopes for all combinations of 6-30 curb and gutter and valley gutter cross-sections in combination with 1/4 inch per foot and 3/16 inch per foot street cross slopes. Gutter flow was calculated using the modified Manning's equation (Equation 4-5). A Manning's n value of 0.016 was used in all cases.

4.3.2 City of Memphis and Shelby County Standard Pavement Sections

There are a variety of typical roadway sections available for use in the City of Memphis and Shelby County. These typical sections can be found in the City of Memphis and Shelby County Subdivision Regulation. Table 4-1 outlines typical urban roadway pavement widths (from gutter flow line to gutter flow line) and corresponding recommended curb and gutter type, recommended cross slope, and allowable spread.

4.3.3 Nomograph Conditional Procedures

The nomograph in Figure 4-6 is used with the following procedures to find gutter capacity for all combinations of cross slopes. If composite section is being evaluated, a composite Z value (Section 4.3.1) must first be calculated for use in Figure 4-6.

Condition 1: Find depth, given gutter flow.

1) Determine input parameters, including grade, S, cross slope, S_x, reciprocal of cross slope, Z_x $(1/S_x)$, gutter flow, Q, and Manning's n

2) Draw a line between the Z_x/n and S 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 discharge scale

4) Read the value of the depth, d, at the intersection of the line from Step 3 and the depth at curb scale

Condition 2: Find gutter flow, given spread.

1) Determine input parameters, including grade, S, cross slope, S_x, reciprocal of cross slope, Z_x $(1/S_x)$, depth, d, spread, T, and Manning's n

2) Draw a line between the Z_x/n and S 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 depth at curb scale

4) Read the value of Q from the intersection of that line on the discharge scale

4.3.4 Example Problems

Example 4-1 Gutter Flow and Spread, given Depth of Flow

Find gutter flow and spread given the following conditions:

- Street section in question is a minor local road with ¼"/foot cross slope
- From Table 4-1, 6-30 curb and gutter is to be used
- From Table 4-2, for concrete gutter with asphalt pavement with moderately rough texture and small slopes (accumulated sediment), n = 0.014 + 0.002 = 0.016
- Longitudinal street grade is 0.01 feet/foot
- Depth of flow in gutter is 3.5 inches

This problem can be solved using the street flow characteristic figures at the end of this chapter, by using the Equations 4-5, 4-7, and 4-8 outlined in Section 4.3 and 4.3.1, or by using Equations 4-7 and 4-8 and Figure 4-6 as outlined in Section 4.3.3. It should be noted that if Manning's n had not been equal to 0.016, the street flow characteristic figures could not be used (Figures 4-1 through 4-4 are based upon n = 0.016).

Solution using the street flow characteristic figures:

1. Select the appropriate street flow characteristic figure. For 6-30 curb and gutter with ¼"/foot cross slope, Figure 4-2 should be used.

2. On the left side of the figure, find the appropriate longitudinal street grade line (0.01 feet/foot).

3. Draw a horizontal line from 3.5 inches (given flow depth) on the "Depth" axis until it intersects the 0.01 slope line.

4. Draw a vertical line down from the intersection point in Step 3 to the "Discharge" axis to find the flow rate, Q: **Q ≈ 3.0 cfs**.

5. On the right side of the figure, draw a horizontal line from 3.5 inches (given flow depth) on the "Depth" axis until it intersects the line labeled "Top Width".

6. Draw a vertical line down from the intersection point in Step 5 to the "Top Width" axis to find the spread, T: **T ≈ 10 feet**.

Solution using Equations 4-5, 4-7, and 4-8:

1. Find composite slope, Z_C, for unequal gutter and pavement cross slopes:

   a. From City of Memphis Division of Engineering Standard Civil Drawing number 2, for 6-30 curb and gutter, gutter cross slope, S_W, is 0.0652 feet/foot. Street cross slope, S_x, is ¼"/foot = 0.0208 feet/foot. Find the reciprocals of gutter and pavement cross slopes:

$$Z_w = \left(\frac{1}{S_w}\right) = \left(\frac{1}{0.0652}\right) = 15.34$$

$$Z_x = \left(\frac{1}{S_x}\right) = \left(\frac{1}{0.0208}\right) = 48.08$$

   b. From City of Memphis Division of Engineering Standard Civil Drawing number 2, for 6-30 curb and gutter, width of gutter, W, is 23 inches. The given depth of flow, d, is 3.5 inches. From Step a, S_W = 0.0652 Feet/foot and S_x, = ¼"/foot = 0.0208 feet/foot. Using Equation 4-7, width of flow, T, can be found:

$$T = \frac{\left(W + \frac{(d - W * S_W)}{S_x}\right)}{12} = \frac{\left(23 + \frac{(3.5 - 23 * 0.0652)}{0.0208}\right)}{12}$$

T = 9.93 feet

c. From Step b, width of flow, T, is 9.93 feet. From Step a, Z_W = 15.34 and Z_x = 48.08. From City of Memphis Division of Engineering Standard Civil Drawing number 2, for 6-30 curb and gutter, width of gutter, W, is 1.92 feet (23 inches). Substituting these values into Equation 4-8, solve for the composite slope:

$$Z_C = 15.34\left(1+\left(\frac{48.08}{15.34}-1\right)\left(\frac{(9.93-1.92)}{\left(9.93+1.92\left(\frac{48.08}{15.34}-1\right)\right)}\right)^{\left(8/3\right)}\right)$$

$$Z_C = 22.69$$

2. From step 1-c, the composite slope, Z_C, is 22.69. Manning's n is given as 0.016. The given longitudinal street grade, S, is 0.01 feet/foot. The given depth of flow, d, is 0.292 feet (3.5 inches). Substituting these values in Equation 4-5, solve for gutter flow, Q:

$$Q = \frac{0.56}{n} Z_x S^{1/2}\ d^{8/3} = \frac{0.56}{0.016} * 22.69 * 0.01^{1/2} * 0.292^{8/3}$$

$$Q = 2.98\ CFS$$

Solution using Figure 4-6 and steps in Section 4.3.3:

1. Find the composite Z value, Z_C, for unequal cross slopes as outlined above: Z_C = 22.69. Also note that in finding Z_C, width of flow was first calculated: **T = 9.93 feet.**

2. Using Z_C = 22.69 from Step 1 and given Manning's n = 0.016, find the ratio Z_C/n:

$$Ratio = \frac{Z_C}{n} = \frac{22.69}{0.016} = 1418.13 \approx 1425$$

3. Draw a line from 1425 on the "ratio" line (leftmost line on Figure 4-6) to 0.01 (given longitudinal grade) on the "slope of channel" line (fourth line from the left on Figure 4-6).

4. Draw a line from the intersection point of the line drawn in Step 3 and the "turning line" (second line from the left on Figure 4-6) to 0.292 (given depth of flow) on the "depth at curb" line (rightmost line on Figure 4-6).

5. Read gutter flow, Q, from the intersection point of the line drawn in Step 4 and the "discharge" line (third line from the left on Figure 4-6). **Q ≈ 2.95.**

Example 4-2 Find Maximum Allowable Gutter Flow

Find maximum allowable gutter flow for the street conditions in example 4-1.

1. Using Table 4-1, determine that the maximum allowable spread for a minor local road is 11 feet.

2. From Step 1, From Example 4-1: Z_W = 15.34 and Z_x = 48.08. From City of Memphis Division of Engineering Standard Civil Drawing number 2, for 6-30 curb and gutter, width of gutter, W, is 1.92 feet (23 inches). Substituting these values into Equation 4-8, solve for the composite slope at T = 11 feet:

$$Z_C = 15.34\left(1+\left(\frac{48.08}{15.34}-1\right)\left(\frac{(11-1.92)}{\left(11+1.92\left(\frac{48.08}{15.34}-1\right)\right)}\right)^{\left(\frac{8}{3}\right)}\right)$$

$$Z_C = 23.78$$

3. Using Equation 4-6, depth of flow at T = 11 feet, d_{11}, must be calculated:

$$d_{11} = S_W W + S_x(12T - W) = 0.0652 * 23 + 0.0208(12 * 11 - 23)$$

$$d_{11} = 3.77\ inches \quad (0.314\ feet)$$

4. Using known values for gutter slope, S_W, gutter width, W, street cross slope, S_x, and width of flow, T, as well as calculated values of Z_C from Step 2 and depth of flow, d, from Step 3, Equation 4-5 can be solved for maximum allowable gutter flow at T = 11 feet, Q_{11}:

$$Q_{11} = \frac{0.56}{n} Z_C \, S^{1/2} d^{8/3} = \frac{0.56}{0.016} * 23.78 * 0.01^{1/2} * 0.314^{8/3}$$

Q_{11} = 3.79 cfs

4.4 Curb Opening Inlets

Curb opening inlets are used in many locations because they offer little interference to traffic and are relatively free from clogging by debris.

4.4.1 Continuous Grade

Capacity of the curb opening inlet on continuous grade depends upon the length of the opening, L_i, the depression of the inlet lip, a, the depth of flow at the curb line in the gutter (d), and both the cross slope (S_x) and the longitudinal slope of the street/gutter (S_o). The manual, *Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12*, March 1969, Bureau of Public Roads, illustrates typical curb opening inlets as shown in Figure 4-7. Definitions of symbols used are also included.

Normally, curb openings in Memphis have a depression of 3 inches. Capacity charts, prepared by Hydraulic Research Division, Bureau of Public Roads, are reproduced. The general conditions for the figures are: W = 2 feet, a ≥ 2 inches, n = 0.016, and L_i = 5 feet, 10 feet, and 15 feet respectively for Figures 4-8, 4-9, and 4-10. These figures are used to find the amount of gutter flow intercepted by curb opening inlets. The rate at which gutter flow is intercepted by curb opening inlets is described as inlet interception rate:

$$Inlet\ Interception\ Rate = \frac{Q_i}{Q} \qquad (4\text{-}9)$$

Where:

Q_i = Gutter flow intercepted ($Q - Q_{downstream}$), in cfs
Q = Gutter flow upgradient (upstream) of inlet ($Q_{upstream}$), in cfs

The use of Figures 4-8, 4-9, and 4-10 is explained below:

1. Calculate width of flow (gutter flow spread), T, and gutter flow, Q, as described in Section 4.3.

2. Find gutter flow spread, T, from Step 1 on the x-axis, labeled "Gutter Flow Spread, T (ft.)", at the bottom left of the graph. Draw a vertical line upward from that point until it intersects the appropriate longitudinal slope, S_o, line.

3. From the intersection point from Step 2, draw a horizontal line to the right until it intersects the appropriate street cross slope, S_x, line.

4. From the intersection point from Step 3, draw a vertical line upward until it intersects the x-axis, labeled "Inlet Interception Rate, Q_i/Q", at the top right of the graph. Approximate the inlet interception rate based on the location of this point on the inlet interception rate scale.

It should be noted that not all longitudinal street grades, S_o, and street cross slopes, S_x, are represented on the figures. Approximation or interpolation may be required. See Example 4-3 for further details on the use of Figures 4-8, 4-9, and 4-10.

4.4.2 Sag Conditions

Capacity of the curb opening inlet in a sag depends upon the depth of water at the inlet and the inlet geometry. The inlet operates as a weir until the water submerges the entrance. Figure 4-11 gives the minimum height, H_m, of opening for weir operation. If the opening height, h, equals or exceeds the minimum height, Figure 4-12 will give the depth of ponding measured at the curb just above the depressed area.

4.4.3 Example Problem

Example 4-3 Curb Opening Inlet on Grade

A curb opening inlet is to be installed in the section of street described in Example 4-1. The curb opening inlet will have a depressed opening with typical geometry described in Section 4.4.1 and illustrated in Figure 4-1. If gutter flow conditions are as described in Example 4-2 (gutter is carrying maximum allowable flow at the maximum allowable spread of 11 feet), what length of opening, L_i, should be used if spread, T, is to be reduced to 7 feet downgradient of the inlet?

Solution:

1. Calculate depth of flow, d, for a width of flow, T, of 7 feet. Use Equation 4-6 as described in Step 3 of Example 4-2:

$$d_6 = S_W W + S_x(12T - W) = 0.0652 * 23 + 0.0208(12 * 7 - 23)$$

$$d_6 = 2.77\ inches \quad (0.231\ feet)$$

2. Calculate a composite Z value, Z_C, for a width of flow, T, of 6 feet. From Step 1, From Example 4-1: Z_W = 15.34 and Z_x = 48.08. From City of Memphis Division of Engineering Standard Civil Drawing number 2, for 6-30 curb and gutter, width of gutter, W, is 1.92 feet (23 inches). Substituting these values into Equation 4-8, solve for the composite slope at T = 6 feet:

$$Z_C = 15.34\left(1 + \left(\frac{48.08}{15.34} - 1\right)\left(\frac{(7 - 1.92)}{\left(7 + 1.92\left(\frac{48.08}{15.34} - 1\right)\right)}\right)^{\left(8/3\right)}\right)$$

$$Z_C = 19.41$$

3. Using Equation 4-5, calculate the gutter flow rate, Q, for the depth of flow from Step 1 and the composite Z value from Step 2. This is the flow rate at which T = 6 feet (flow rate downgradient of the inlet):

$$Q_6 = \frac{0.56}{n} Z_C\ S^{1/2} d^{8/3} = \frac{0.56}{0.016} * 19.41 * 0.01^{1/2} * 0.231^{8/3}$$

$$Q_6 = 1.36\ CFS$$

4. From Example 4-2, it is known that gutter flow, Q_{11}, upgradient of the inlet is 3.79 cfs. If the gutter flow downstream of the inlet, Q_6, is to be 1.36 cfs, then the gutter flow intercepted, by the inlet, Q_i is:

$$Q_i = Q_{upstream} - Q_{downstream} = Q_{11} - Q_6 = 3.79 - 1.36$$

$$Q_i = 2.43\ CFS$$

5. If the inlet must intercept 2.43 cfs of the 3.79 cfs flowing upgradient (upstream) of the gutter, then, using Equation 4-9, the required inlet interception rate is:

$$Inlet\ Interception\ Rate = \frac{Q_i}{Q} = \frac{Q_i}{Q_{11}} = \frac{2.43}{3.79}$$

$$Inlet\ Interception\ Rate = 0.64$$

6. Using Figure 4-8 and applying T = 11, S_o = 0.01, and S_x = 0.0208, follow the steps described in Section 4.4.1 to check if a curb opening of 5 feet provides an adequate interception rate. The street cross slope, S_x, is not represented by a line on the figure, so values from lines representing cross slopes greater than (0.03 line) and less than (0.015 line) are interpolated. The inlet interception rate for a curb opening of 5 feet is approximately 0.42, which is lower than the required 0.64. A 5-foot curb opening inlet will not work.

7. Repeat Step 6 for Figure 4-9 to check if a curb opening of 10 feet provides an adequate interception rate. The inlet interception rate for a curb opening of 10 feet is approximately 0.66, which is higher than the required 0.64. A 10-foot curb opening inlet meets the requirement. Approximately 2.50 cfs of the 3.79 cfs flowing upgradient (upstream) of the inlet would be intercepted by the inlet.

   Note that a 15-foot curb opening inlet may also be used, as it exceeds the requirement with an interception rate of 0.81. Approximately 3.07 cfs of the 3.79 cfs flowing upgradient (upstream) of the inlet would be intercepted by the inlet.

4.5 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. For an efficient grate inlet, all rectangular bars should be parallel with the flow and the openings should cover at least 50 percent of the width of the grate. All standard City of Memphis grate inlets have at least 50 percent openings.

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

4.5.1 Continuous Grade

The capacity of an undepressed efficient grate inlet on a continuous grade can generally be determined by computing the flow in the curb section occupied by the grate width. The capacity of the valley gutter grate on a continuous grade, as a function of depth of flow and longitudinal slope, can be determined by referring to Figure 4-13 and dividing the result by two.

4.5.2 Sump (sag) Conditions

The capacity of a grate inlet in a sag can be determined by assuming the grate operates first as weir having a crest length roughly equal to the outside perimeter, P, of the clear opening along which the flow enters. Weir operation continues to a depth, d, of about 0.4 feet above the top of the grate. The discharge intercepted by the grate is:

$$Q_i = CP\,d^{3/2} \qquad (4\text{-}10)$$

Where:

- C = Weir coefficient, use 3.0 unless otherwise approved by City/County Engineer
- Q_i = Rate of discharge into grate opening, in cfs
- P = Perimeter of grate opening, in feet
- D = Depth of flow at grate, in feet

The equations for weir operation for grate inlets adjacent to curb (the side against the curb is not included in computing P) used in the City of Memphis and Shelby County are as follows:

No. 10 grate inlet:

$$Q_i = 11.69\, d^{3/2} \qquad (4\text{-}11)$$

No. 11 or S-11 grate inlet:

$$Q_i = 16.75\, d^{3/2} \qquad (4\text{-}12)$$

Valley gutter grate inlet:

Q_i = See Figure 4-14

These equations do not take into account clogging or debris buildup. Because of the vortices and tendency of trash to collect on the grate, the clear opening or perimeter of the grate inlet should be at least twice that required by the above equation. During design of grate inlets for capacity, values obtained in Equations 4-11 and 4-12 and Figure 4-14 shall be divided by 2 to account for trash and debris collection on the grate.

Since maximum depth of the water in a sag shall not exceed the top of the curb or the encroachment on the street equivalent to that allowed on the street for the design storm, only the weir equation will be used to determine grate capacities for inlets in streets. For inlets not in streets, i.e., yard inlets or parking lot inlets (not adjacent to curb), use Equations 4-13 and 4-14 for grate inlets with water depths less than or equal to 12 inches. For water depths greater than 12 inches, designs and calculations will be reviewed by the City Engineer's office. For city standard 3' x 3' inlets or S-11 inlets, use Equation 4-14. Assume the openings are 50 percent clogged with trash and debris; therefore, the actual flow equals one half the calculated flow.

The equations for weir operation for grate inlets not in streets (weir operation on all four sides) used in the City of Memphis and Shelby County are as follows:

No. 10 grate inlet:

$$Q_i = 16.88\, d^{3/2} \tag{4-13}$$

No. 11 or S-11 grate inlet:

$$Q_i = 27\, d^{3/2} \tag{4-14}$$

4.5.3 Example Problem

Example 4-4 Grate Inlet in Sump Condition

A local contractor wants to build a 2.0-acre parking lot. The contractor plans to configure the lot so that all surface runoff will be collected by four grate inlets in sump conditions at the center of each of four sections of the parking lot. The four sections consist of a 0.65-acre section, a 0.55-acre section, and two 0.4-acre sections. All four sections will be asphalt pavement and have cross slopes of 0.01 feet/foot. Peak runoff, Q_p, for the entire 2.0-acre parking lot is calculated at 10.25 cfs. The contractor would like to use No. 10 grate inlets if possible, and a No. 11 grate inlet if a No. 10 inlet does not provide sufficient capacity. Determine the type of inlet required for each section of the parking lot if the ponding depth is not to exceed 0.4 feet (4.8 inches).

Solution:

1. Determine runoff to be intercepted for each parking lot section size:

$$Q_{0.65} = 10.25\left(\frac{0.65}{2.0}\right) = 3.33\ CFS$$

$$Q_{0.55} = 10.25\left(\frac{0.55}{2.0}\right) = 2.82\ CFS$$

$$Q_{0.4} = 10.25\left(\frac{0.4}{2.0}\right) = 2.05\ CFS$$

2. Determine the capacity of a No. 10 grate inlet with d = 0.4 feet (4.8 inches). Since the inlet is to be located in the center of each parking lot section (flow will be intercepted from all sides), Equation 4-13 will be used. Per requirements in Section 4.5.2, capacity will be reduced by half to account for 50% clogged openings:

$$Q_{\#10} = 16.88\, d^{3/2} = 16.88 * 0.4^{3/2}$$

$$Q_{\#10} = 4.27\ CFS$$

$$Q_{\#10} = \frac{4.27\ CFS}{2} = 2.14\ CFS$$

3. The peak flow for two of the parking lot sections exceeds the calculated capacity for a No. 10 grate inlet. Determine the capacity of a No. 11 grate inlet with d = 4.5 inches (0.375 feet). Since the inlet is to be located in the center of each parking lot section (flow will be intercepted from all sides), Equation 4-14 will be used. Per requirements in Section 4.5.2, capacity will be reduced by half to account for 50% clogged openings:

$$Q_{\#11} = 27\, d^{3/2} = 27 * 0.4^{3/2}$$

$$Q_{\#11} = 6.83\ CFS$$

$$Q_{\#11} = \frac{6.83}{2} = 3.42\ CFS$$

4. A No. 10 grate inlet with 50% blockage is able to intercept flow at a rate of 2.14 cfs, making it sufficient for use in the 0.4-acre sections, but not able to handle the peak flows from the 0.65-acre and 0.55-acre sections. A No. 11 grate inlet with 50% blockage is capable of intercepting runoff at a rate of 3.42 cfs, which is greater than any of the individual section peak flow rates. In summary:

		Inlet Use	
Lot Section (acres)	**Section Peak Flow (cfs)**	**No. 10 (2.14 cfs)**	**No. 11 (3.42 cfs)**
0.65	3.33		**X**
0.55	2.82		**X**
0.40	2.05	**X**	
0.40	2.05	**X**	

4.6 Combination Inlets (6-72 and 6-72M)

4.6.1 Continuous Grade

The capacity of an unclogged combined inlet on a continuous grade, using an efficient grate, is not appreciably greater than the grate alone. The capacity of a 6-72 inlet on a continuous grade, as a function of depth of flow and longitudinal slope, can be determined by referring to Figure 4-15. The capacity of a 6-72M inlet on a continuous grade can be determined by referring to Figure 4-16, where the depth of flow refers to the depth of flow in the standard 6-30 curb and gutter 20 feet upgradient (upstream) of the 6-72M inlet.

4.6.2 Sump Conditions

All debris carried by storm water runoff that is not intercepted by upgradient (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. A combination inlet used in a sag adds safety from clogging by trash. The curb opening provides relief opening if the grate should become clogged. To compute the capacity, use Equation 4-12 for the No. 11 inlet grate. Do not divide by 2 due to curb opening.

4.6.3 Example Problem

Example 4-5 6-72 Inlet on Continuous Grade

A 6-72 inlet is to be installed in the section of street described in Example 4-1. The 6-72 inlet will be installed per City of Memphis Engineering Division Civil Standard Drawing No. 21. If gutter flow conditions are as described in Example 4-2 (gutter is carrying maximum allowable flow at the maximum allowable spread of 11 feet), what will the gutter flow downgradient, $Q_{downstream}$, of the inlet be?

Solution:

1. Find depth of flow upgradient (upstream) of inlet: From Example 4-2, the gutter flow upgradient (upstream) of the 6-72 inlet, Q_{11}, is approximately 3.79 cfs flowing at a depth, d_{11}, of 3.77 inches.

2. Applying upgradient (upstream) depth of flow, d_{11} = 3.77 inches and longitudinal street gradient, S_o = 0.01 to Figure 4-15, find the inlet capacity, Q_i:

   a. Locate upgradient (upstream) depth of flow, d_{11} = 3.77 inches on the y-axis, labeled "Depth of Flow, inches."

   b. Draw a horizontal line to the right until it intersects the appropriate longitudinal street grade, S_o, line.

   c. Draw a vertical line downward from the intersection point from Step 2b until it intersects the x-axis, labeled "Q, Inlet Capacity, Cubic Feet Per Second." Approximate the 6-72 inlet capacity, Q_i, based on the location of this point on the inlet capacity scale: $Q_i \approx 2.05$ cfs.

3. Find the gutter flow downgradient (downstream), $Q_{downstream}$: If the flow upgradient (upstream), Q_{11}, from Step 1 is 3.79 cfs, and the inlet capacity, Q_i, from Step 2 is 2.05 cfs, then the flow downgradient (downstream) of the 6-72 inlet is:

$$Q_{downstream} = Q_{11} - Q_i = 3.79 - 2.05$$

$$Q_{downstream} = 1.74\ CFS$$

4.7 Computer Programs

There are various computer programs available that aid in time-consuming hand calculations associated with gutter and inlet hydraulics. As illustrated in Example 4-6, computer programs are extremely useful when calculating gutter flow rates. However, caution should be exercised when using a computer program to calculate inlet capacities and interception rates. Many computer models are inconsistent with the methods used in Memphis and Shelby County in the manner they carry out these calculations or they do not accurately model the capabilities of inlet structures used in Memphis and Shelby County. The designer should seek approval from the City and/or County Engineer before using gutter and inlet hydraulic modeling software as a design tool.

4.7.1 Example Problem

Example 4-6 6-72 Inlet on Continuous Grade

In Example 4-5, the gutter flow downgradient (downstream), $Q_{downstream}$, was calculated to be 1.74 cfs based upon street conditions in Example 4-1 and upgradient (upstream) flow parameters from Example 4-2. Although, knowing the downstream flow rate can be useful, it leaves many unknowns: depth of flow, d, width of flow, T, and composite Z value, Z_C are the most important. These unknowns can be solved using hand calculation methods, however, it is often significantly simpler to use a computer program. For this example, the Federal Highway Administration's Urban Drainage Design Program, Visual Urban (HY-22), will be utilized. Visual Urban requires the user to input 7 of 8 gutter flow parameters:

1. Longitudinal Slope, S: 0.01 feet/foot
2. Pavement Cross Slope, S_x: 0.0208 feet/foot
3. Gutter Cross Slope, S_W: 0.0652 feet/foot
4. Manning's Coefficient, n: 0.016
5. Gutter Width, W: 1.92 feet
6. Gutter Depression, a: 0.00 inches
7. Discharge (Gutter Flow), Q: 1.74 cfs
8. Width of Spread, T: Unknown

Based upon these input parameters, Visual Urban calculates width of spread, T = , as well as gutter flow ratio, E_o, depth of flow, d, and average velocity, V. Visual Urban Generates an output table that can be found as Table 4-4.

4.8 General Guidance

Water tables will be permitted on minor streets and only paralleling water tables will be allowed along collector streets, provided the water tables are a minimum of 8 feet wide, have a minimum thickness of 8 inches of Portland Cement concrete, a minimum longitudinal slope of 0.5 percent grade, and an invert depth of 4 inches. Limit water table flow across intersections so as not to exceed a 2-inch depth. Maximum flows across water tables are listed in Table 4-3 as a function of depth and longitudinal street grade.

At the intersection of streets, the curb and gutter gradient around the radius is to be 0.5 percent or greater. If the flow is to split in the radius, the high point of the curb and gutter elevations are to be shown on plans.

Drainage pipe will be located generally along the center line of streets. For drainage pipes located in streets, a minimum cover of 2 feet is required between the top of curb elevation and outside top of the drainage pipe to ensure that the drainage pipe is deep enough in the roadbed to not damage the pipe and to ensure proper hydraulic head is provided in the inlet box. Drainage inlets are not acceptable in the corner radius of curb and gutter at intersections.

Inlets will be required on both sides of the street when the street gradient within 150 feet of the intersection is 5 percent or greater (to prevent intersection water overrun).

4.9 Chapter Equation

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

Where:

Q = Gutter flow rate, in cfs
n = Manning's roughness coefficient
S_x = Pavement cross slope, in feet/foot
S = Longitudinal street grade, in feet/foot
T = Width of flow or spread, in feet

$$T = \frac{d}{S_x} \qquad (4\text{-}2)$$

Where:

T = Width of flow or spread, in feet
d = Depth of flow, in feet
S_x = Pavement cross slope, in feet/foot

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

$$Z_x = \left(\frac{1}{S_x}\right) \qquad (4\text{-}4)$$

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

Where:

Q = Gutter flow rate, in cfs
n = Manning's roughness coefficient
Z_x = Reciprocal of pavement cross slope ($1/S_x$), in feet/foot
S = Longitudinal street grade, in feet/foot
d = Depth of flow, in feet

$$d = S_W W + S_x \left(12T - W\right) \tag{4-6}$$

$$T = \frac{\left(W + \frac{\left(d - W * S_W\right)}{S_x}\right)}{12} \tag{4-7}$$

Where:

T = Width of flow or spread, in feet
W = Width of gutter, in inches
d = Depth of flow, in inches
S_W = Gutter cross slope, in feet/foot
S_x = Pavement cross slope, in feet/foot

$$Z_C = Z_W \left(1 + \left(\frac{Z_x}{Z_W} - 1\right)\left(\frac{T - W}{\left(T + W\left(\frac{Z_x}{Z_W} - 1\right)\right)}\right)^{\left(8/3\right)}\right) \tag{4-8}$$

Where:

Z_C = Reciprocal of composite pavement cross slope ($1/S_c$), in feet/foot
Z_W = Reciprocal of gutter cross slope ($1/S_W$), in feet/foot
Z_x = Reciprocal of pavement cross slope ($1/S_x$), in feet/foot
T = Width of flow or spread, in feet
W = Width of gutter, in feet

$$Inlet\ Interception\ Rate = \frac{Q_i}{Q} \tag{4-9}$$

Where:

Q_i = Gutter flow intercepted ($Q - Q_{downstream}$), in cfs
Q = Gutter flow upgradient (upstream) of inlet ($Q_{upstream}$), in cfs

$$Q_i = CP\,d^{3/2} \tag{4-10}$$

Where:

C = Weir coefficient, use 3.0 unless otherwise approved by City/County Engineer.
Q_i = Rate of discharge into grate opening, in cfs
P = Perimeter of grate opening, in feet
d = Depth of flow at grate, in feet

$$Q_i = 11.69\,d^{3/2} \tag{4-11}$$

$$Q_i = 16.75\,d^{3/2} \tag{4-12}$$

$$Q_i = 16.88\,d^{3/2} \tag{4-13}$$

$$Q_i = 27\,d^{3/2} \tag{4-14}$$

Back to Section 4.2.3, 4.3.1

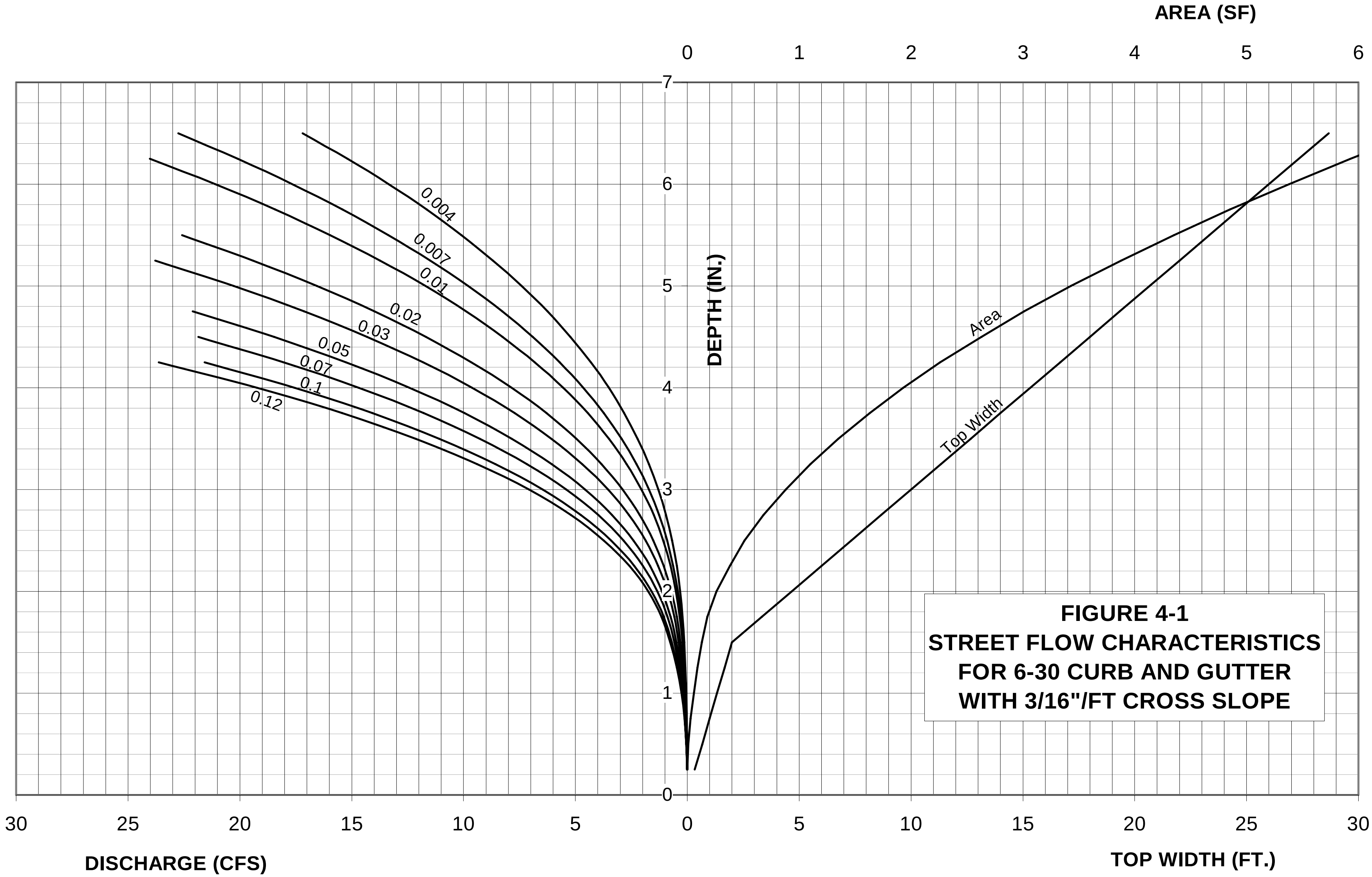


FIGURE 4-1
STREET FLOW CHARACTERISTICS
FOR 6-30 CURB AND GUTTER
WITH 3/16"/FT CROSS SLOPE

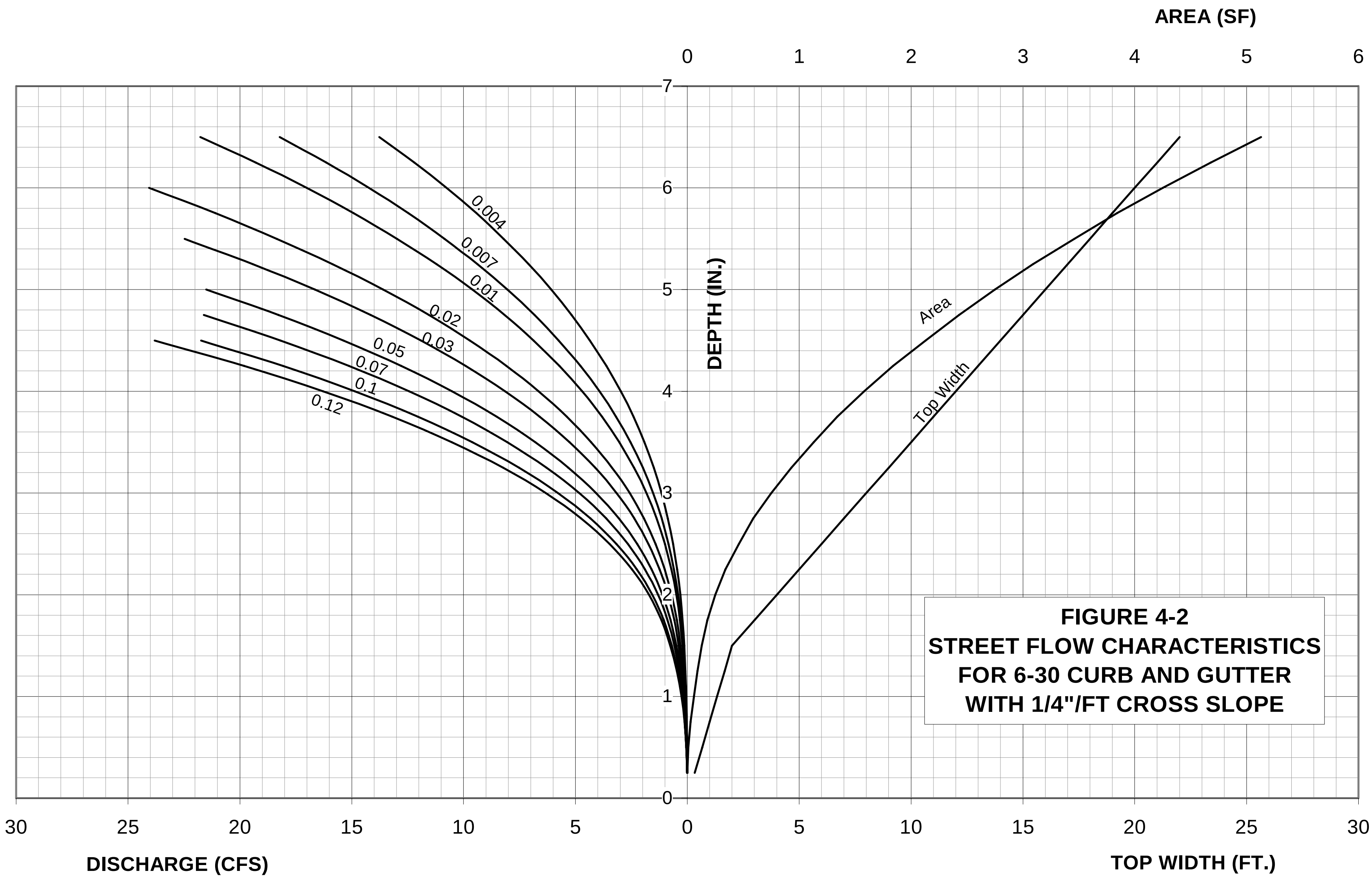


FIGURE 4-2
STREET FLOW CHARACTERISTICS
FOR 6-30 CURB AND GUTTER
WITH 1/4"/FT CROSS SLOPE

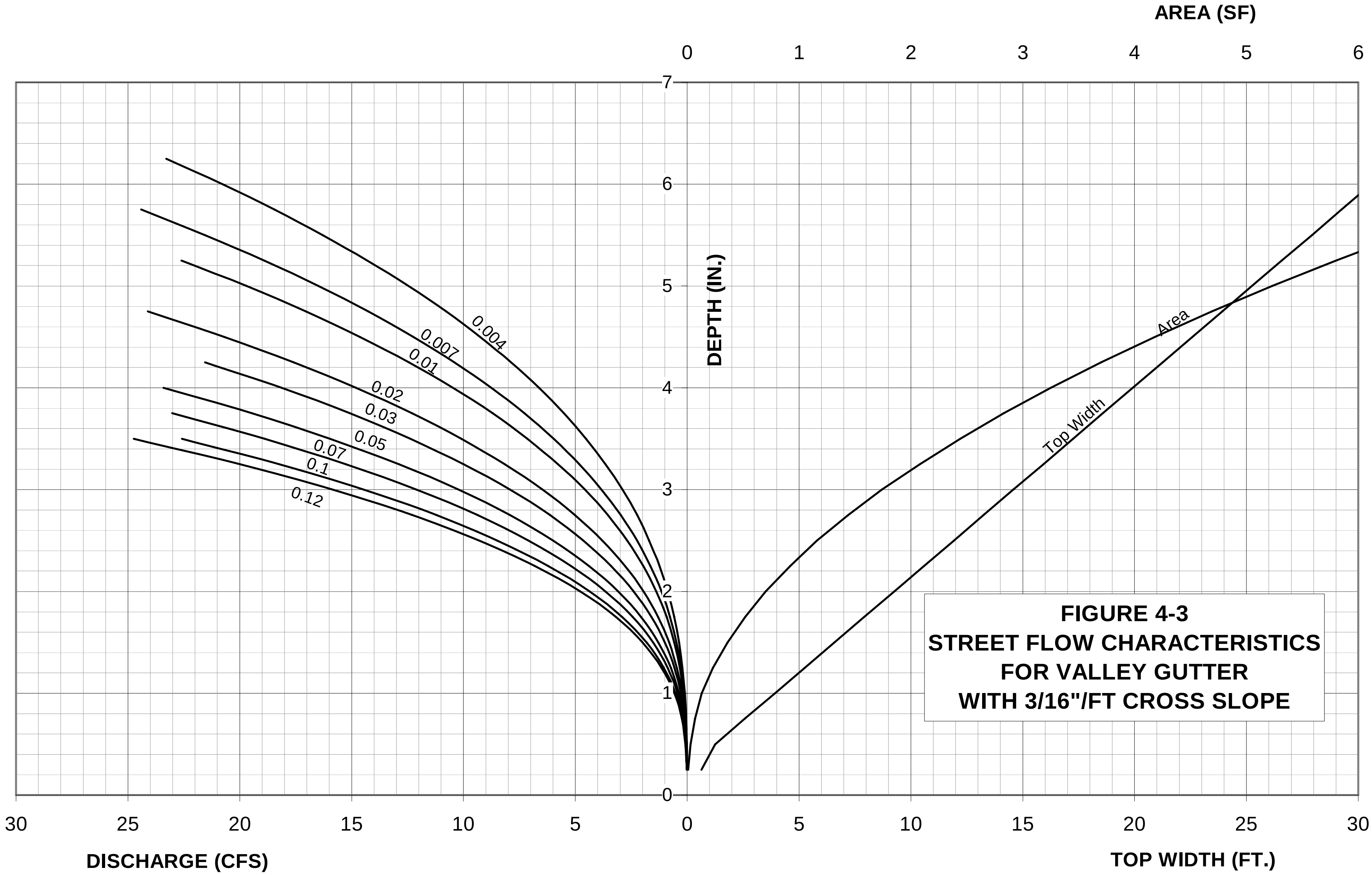


FIGURE 4-3
STREET FLOW CHARACTERISTICS
FOR VALLEY GUTTER
WITH 3/16"/FT CROSS SLOPE

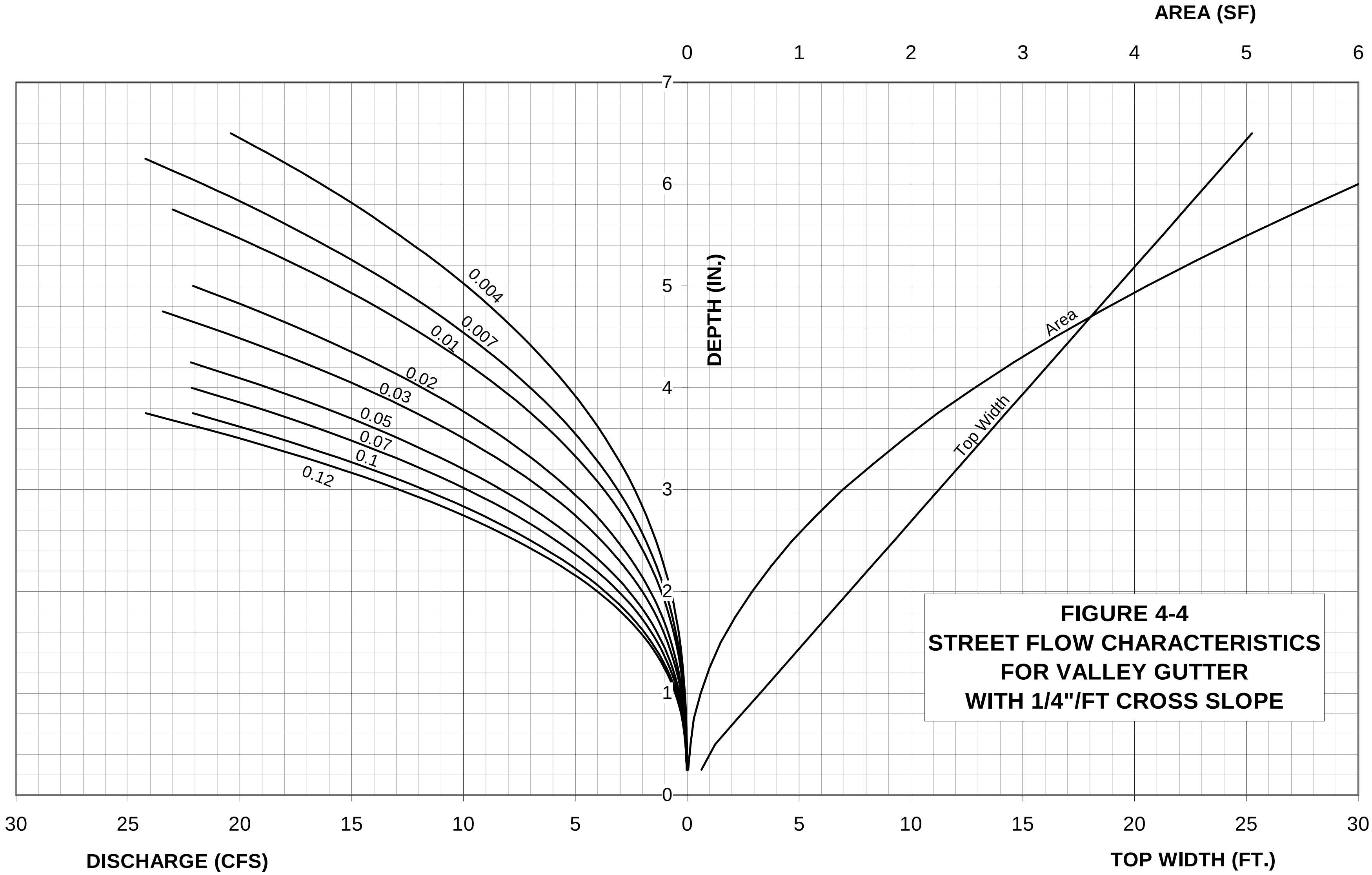
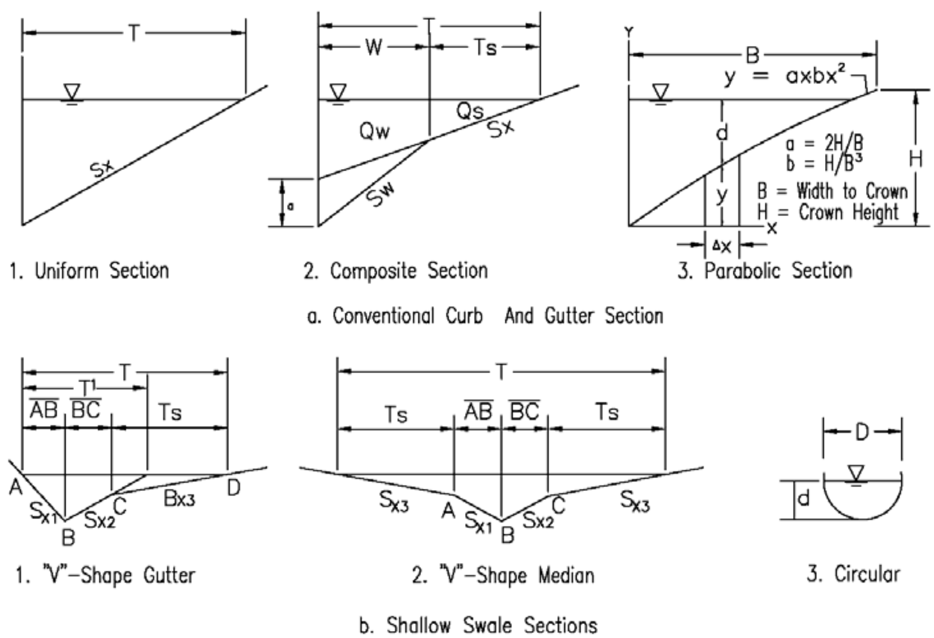


FIGURE 4-4
STREET FLOW CHARACTERISTICS FOR VALLEY GUTTER WITH 1/4"/FT CROSS SLOPE



Back to Section 4.3

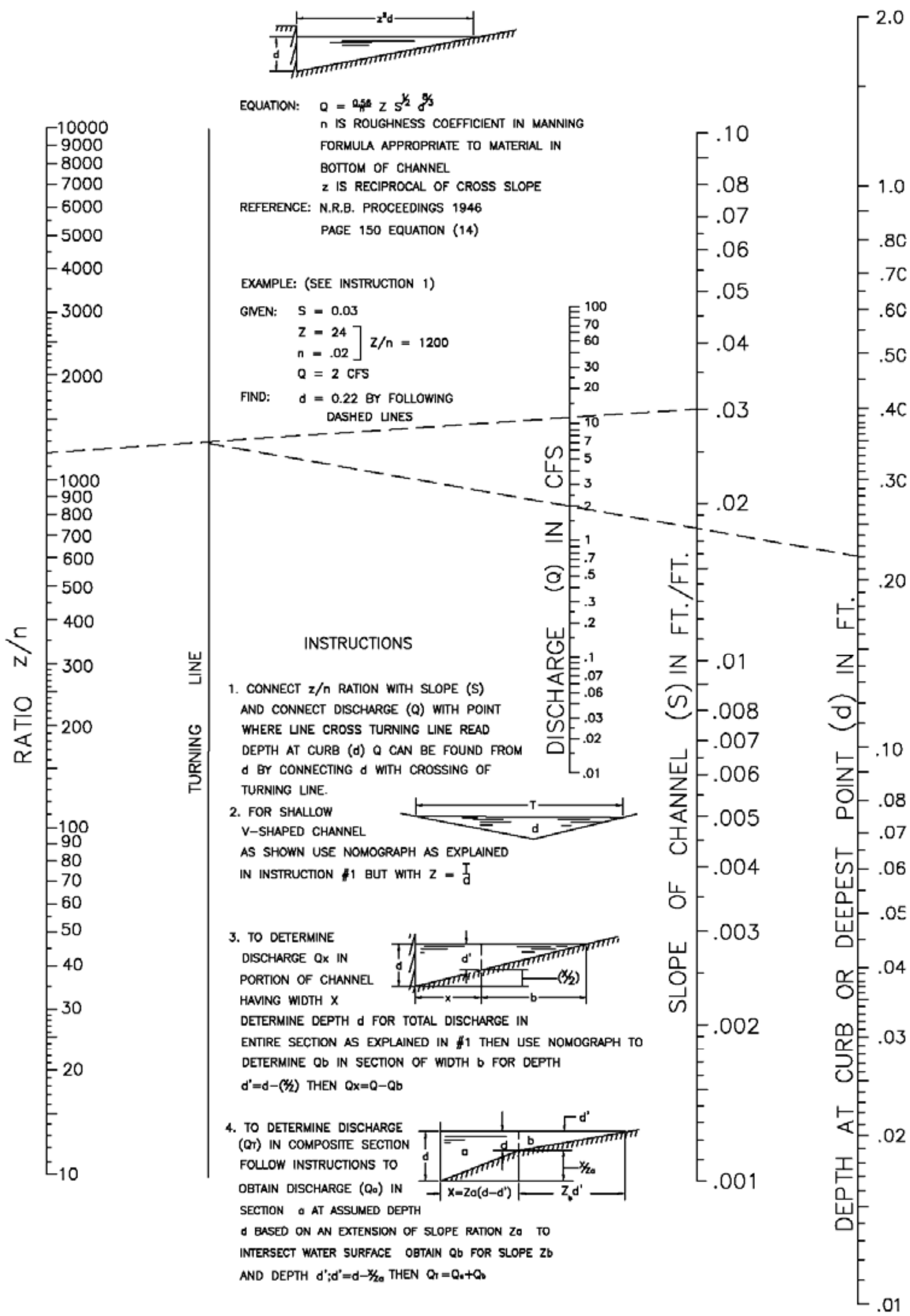


Figure 4-6
Nomograph for Flow in Triangular Gutter Sections

Back to Section 4.3.1, 4.3.3

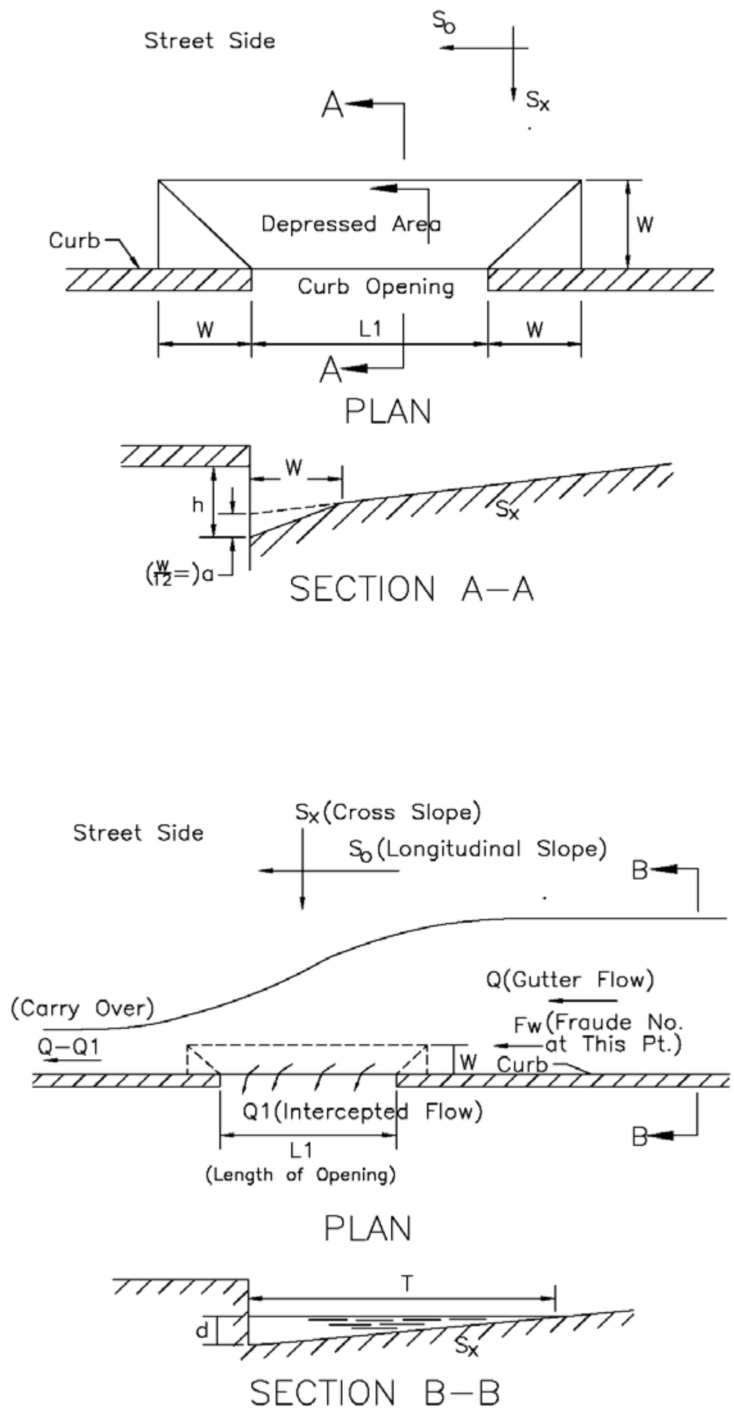


Figure 4-7
Typical Curb Inlet

Back to Section 4.4.1

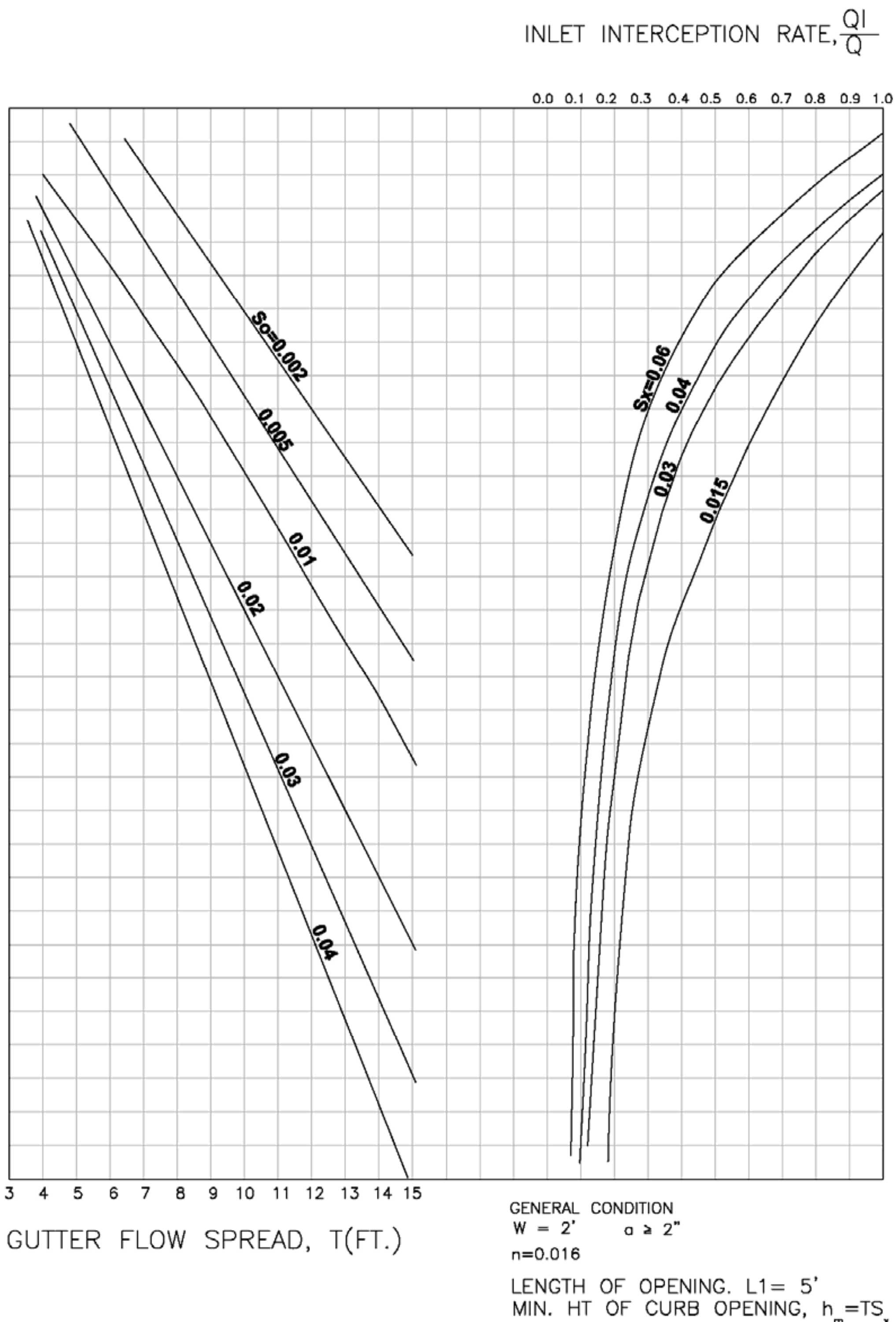


Figure 4-8
Capacity Chart for 5' Curb Opening

Back to Section 4.4.1

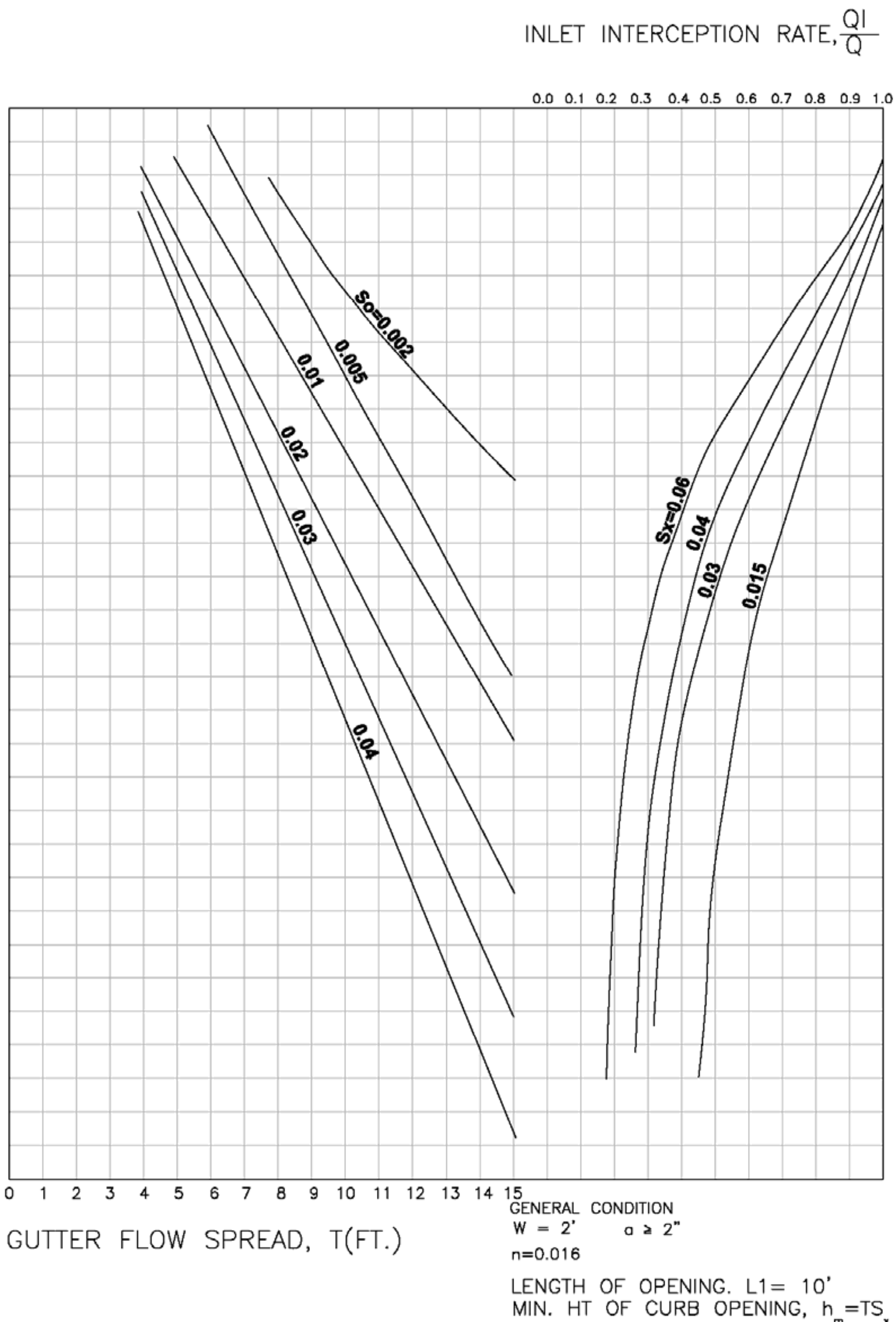


Figure 4-9
Capacity Chart for 10' Curb Opening

Back to Section 4.4.1

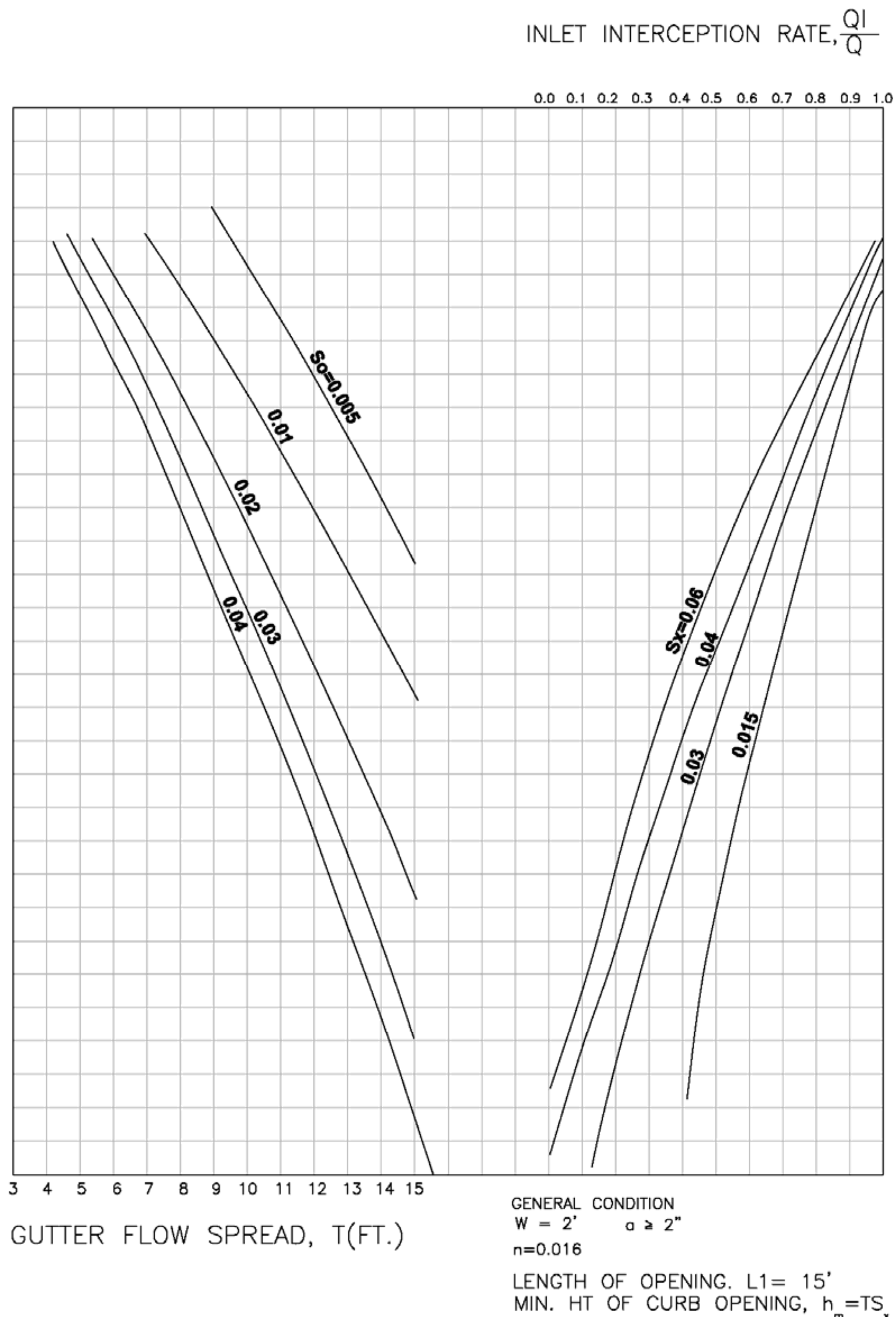


Figure 4-10
Capacity Chart for 15' Curb Opening

Back to Section 4.4.1

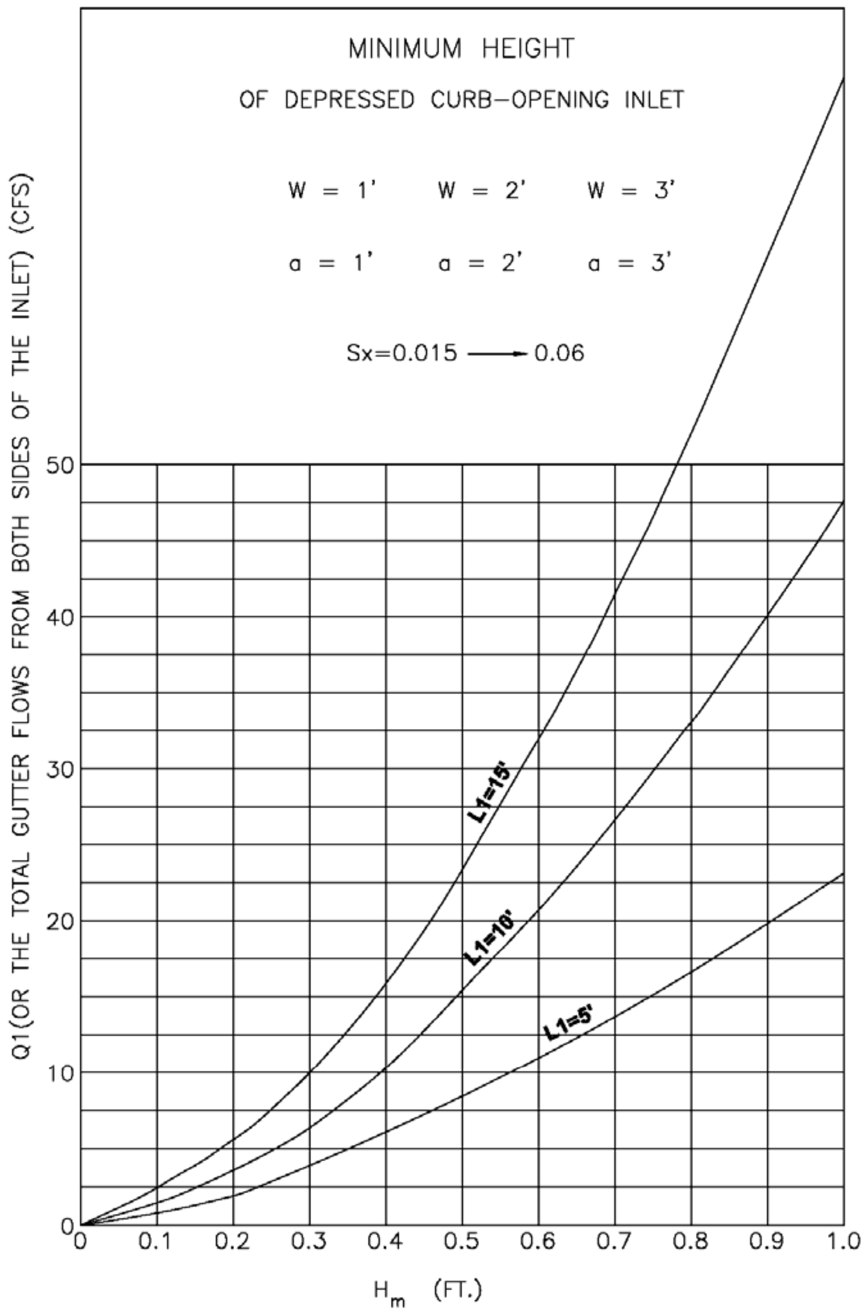


Figure 4-11
Minimum Height of Depressed Curb-Opening Inlet

Back to Section 4.4.2

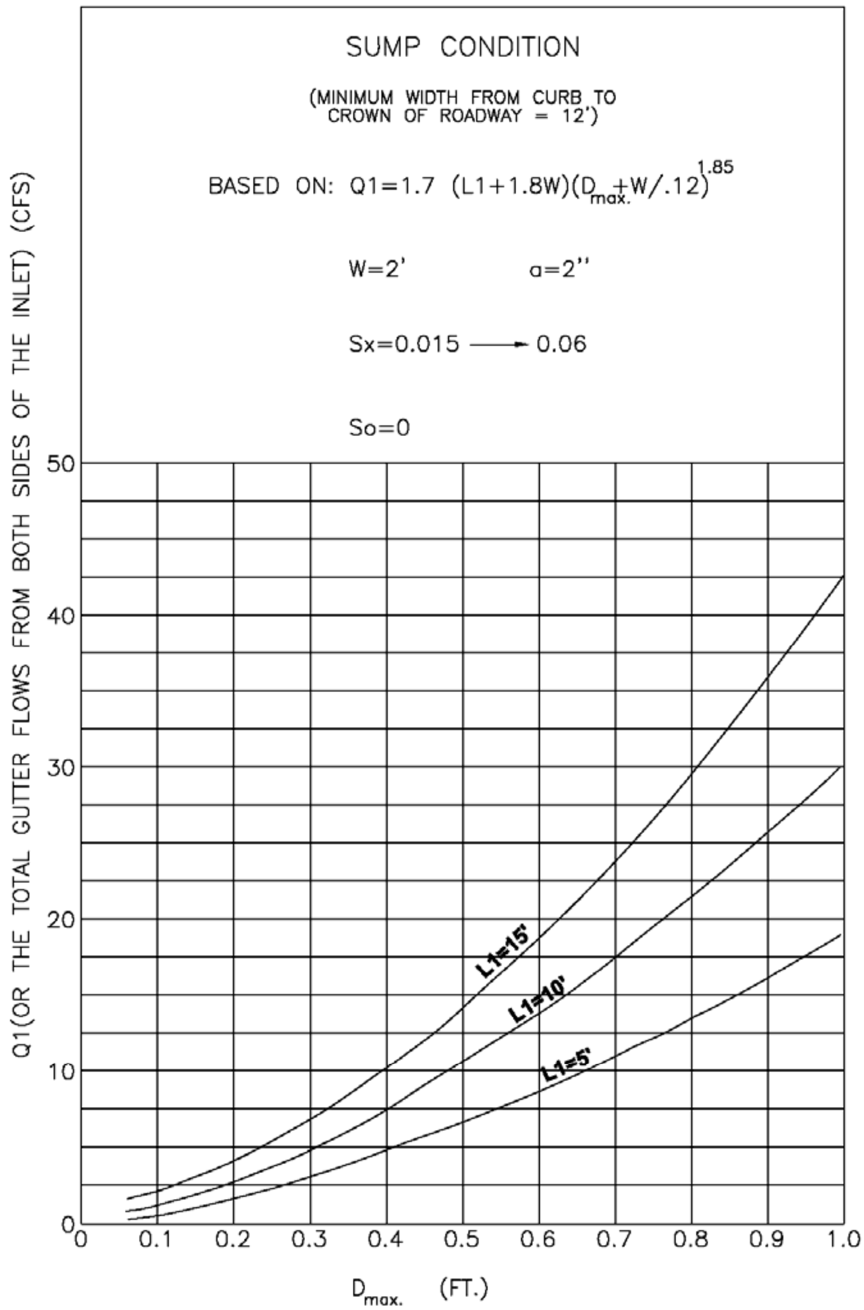


Figure 4-12
Depth of Ponding In Sump Condition

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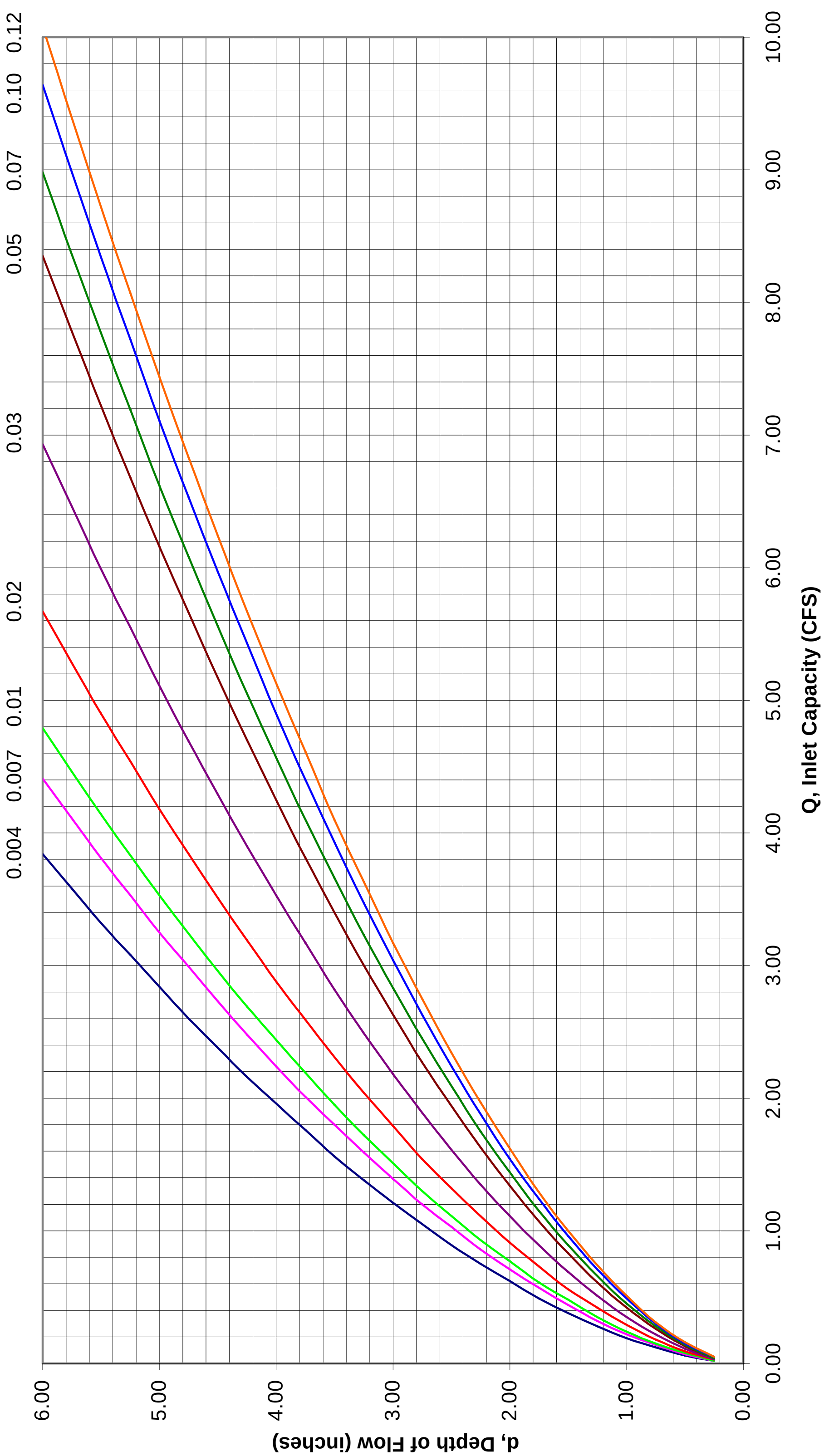
Figure 4-13
Inlet Capacity of Valley Gutter on a Continuous Slope



Back to [Section 4.5.1]

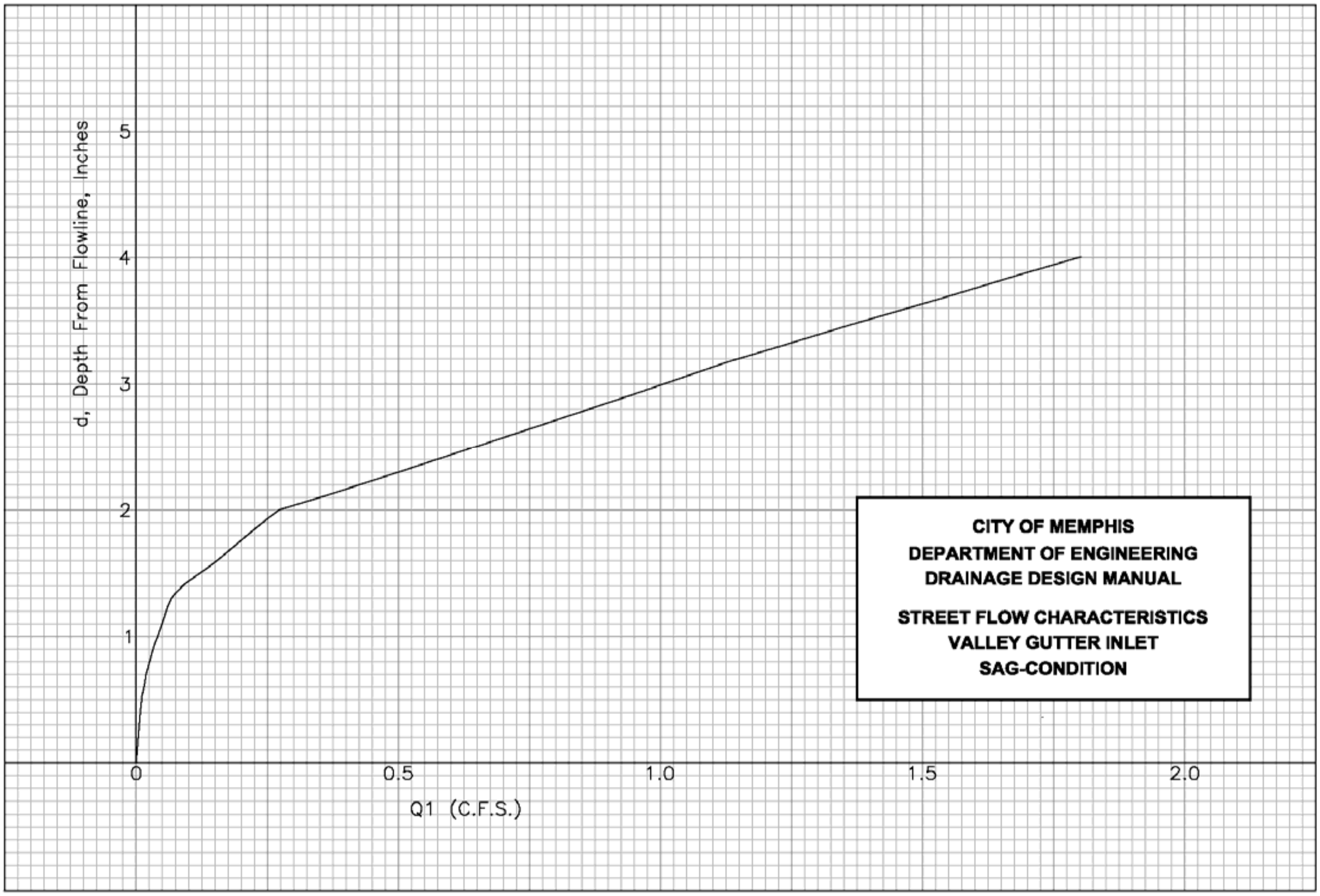
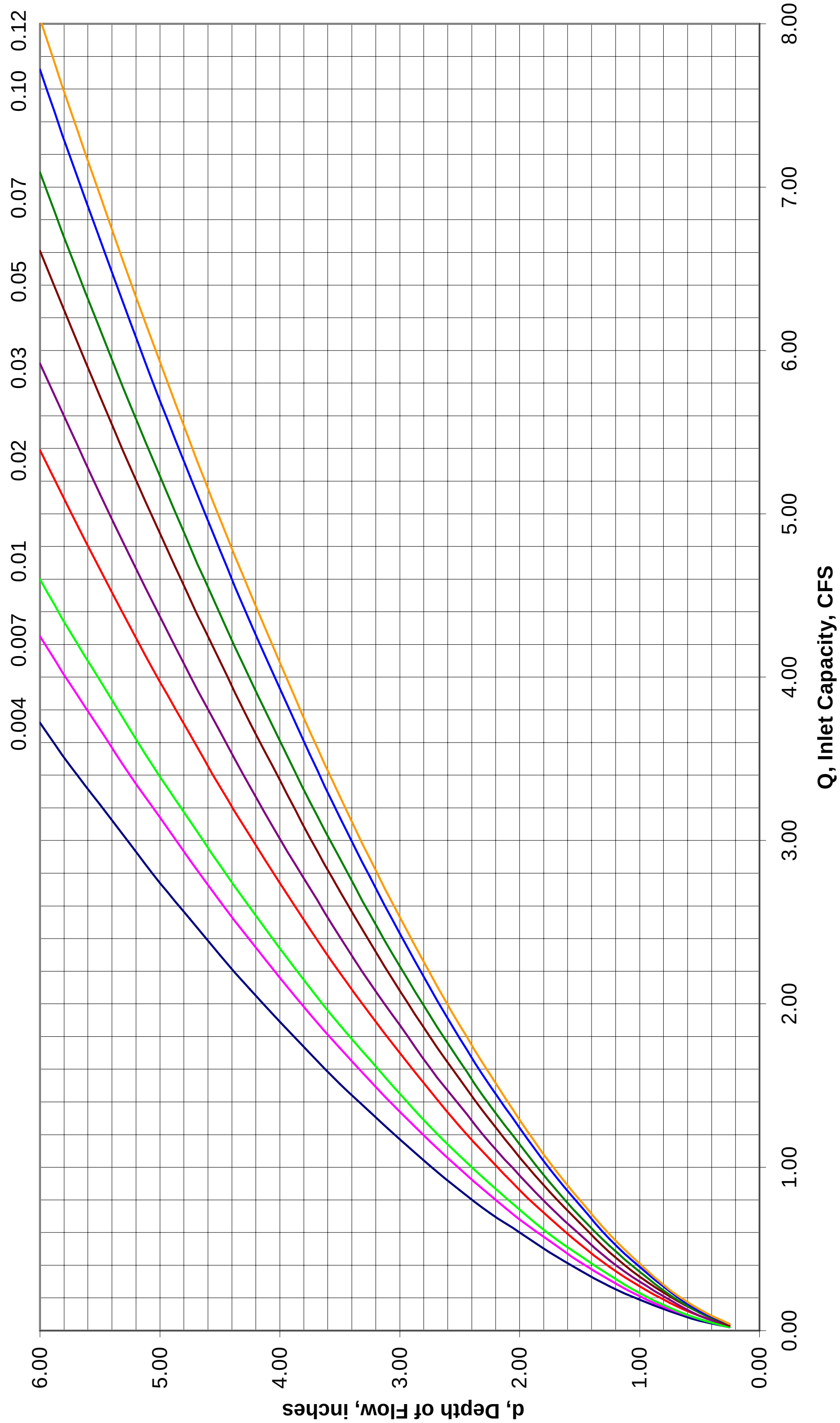


Figure 4-14
Street Flow Characteristics for Valley Gutter Inlet in Sag-Condition

Back to Section 4.5.2

Figure 4-15
Inlet Capacity of 6-72 Inlet on Continuous Grade



Back to Section 4.6.1

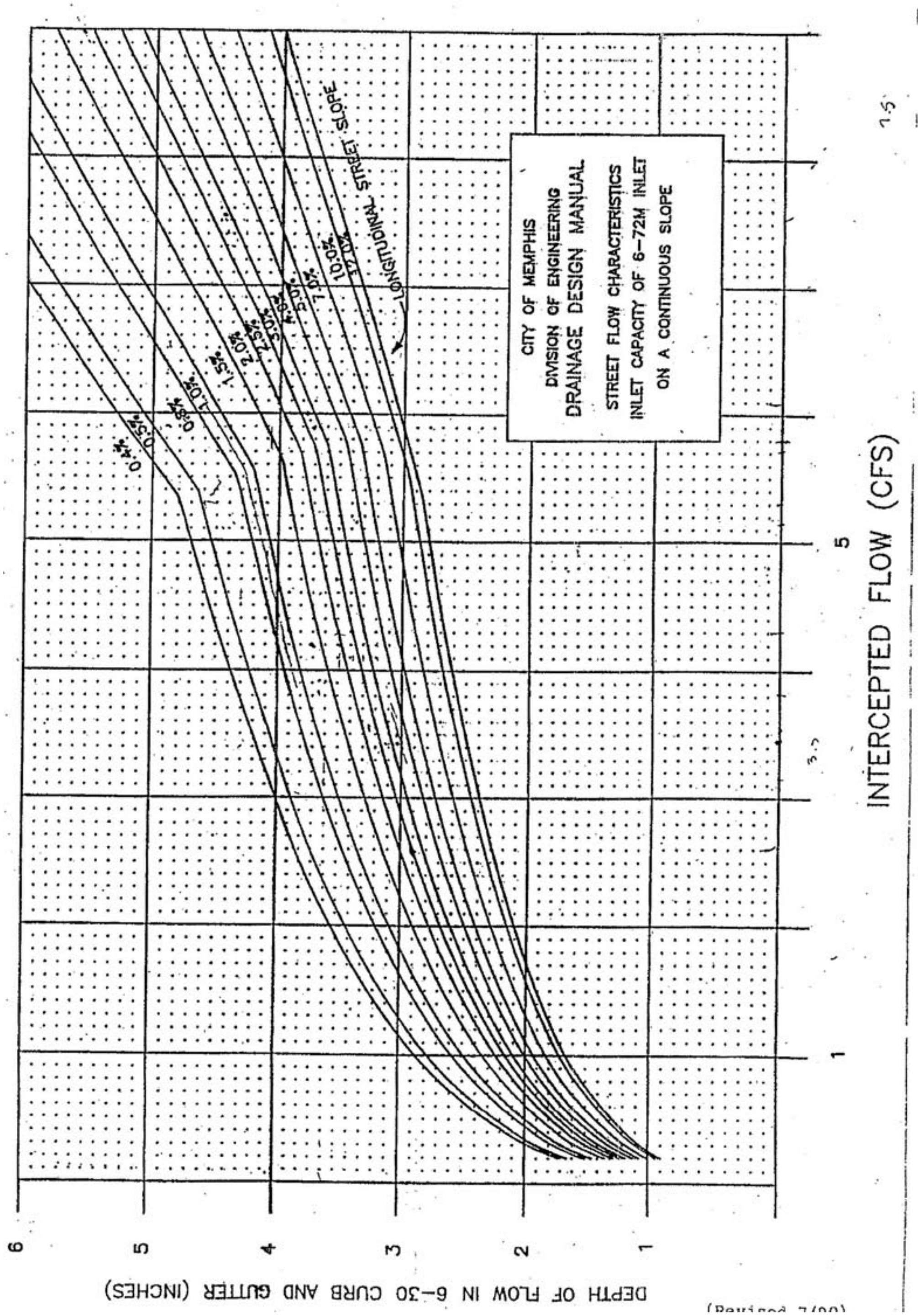


Figure 4-16
Street Flow Characteristics Inlet Capacity of 6-82M Inlet on a Continuous Slope

Back to Section 4.6.1

Table 4-1
Urban Street Design Requirements

Street Type	Pavement Width (feet)	Curb & Gutter	Cross Slopes	Max. Encroachment (each side of street)
ARTERIALS[a]				
Limited access	M.R.P.	N/A	3/16", 1/4", parabolic	12
Principal	M.R.P.	6-30	3/16", 1/4", parabolic	12
Minor	M.R.P.	6-30	3/16", 1/4", parabolic	12
COLLECTORS[b]				
Major	48	6-30	3/16", 1/4", parabolic	14
Minor	40	6-30	3/16", 1/4", parabolic	10
Cul-de-sac terminus (Radius)	65	6-30	3/16", 1/4", parabolic	10
LOCALS[c]				
Major	36	6-30	3/16", 1/4", parabolic	14
Minor	30	6-30	3/16", 1/4", parabolic	11
	30	Valley[d]	3/16", 1/4", parabolic	11
Loop				
50 dwelling units or less	28	6-30	3/16", 1/4", parabolic	10
	28	Valley[d]	3/16", 1/4", parabolic	10
Greater than 50 dwelling units	28	6-30	3/16", 1/4", parabolic	10
	28	Valley	3/16", 1/4", parabolic	10
	28	Valley	3/16", 1/4", parabolic	10
Cul-de-sac				
25 dwelling units or less	28	6-30	3/16", 1/4", parabolic	10
	28	Valley	3/16", 1/4", parabolic	10
Greater than 25 dwelling units	28	6-30	3/16", 1/4", parabolic	10
	28	Valley	3/16", 1/4", parabolic	10
	28	Valley	3/16", 1/4", parabolic	10
Cul-de-sac terminus (Radius)	40	6-30	3/16", 1/4", parabolic	10
	40	Valley	3/16", 1/4", parabolic	10

Notes:

M.R.P. = Major road plan
C.L. = Curbline
[a] = Spread of water in gutter shall be limited so that no more than one traffic lane is encumbered in either direction
[b] = Flow spread must leave at least one lane free of water in each direction
[c] = Flow may spread to within 4 feet of the parablolic of the street
[d] = Valley curbs only permitted where lots have 35 feet of frontage or less excluding corner lots and normal variances required for curvilinear lots

Back to Section 4.1, 4.2.2, 4.2.3, 4.2.7, 4.2.8, 4.3.2

Table 4-2
Manning's n for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete Pavement:	
Float finish	0.014
Broom finish	0.016
Concrete gutter, trowled finish	0.012
Concrete Gutter-Asphalt Pavement:	
Smooth	0.013
Rough	0.015
For gutters with small slope, where sediment may accumulate, increase above values of "n" by	0.002

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

Back to Section 4.2.5

Table 4-3
Flows in Water Table as a Function of Depth and Longitudinal Street Grade
(to the nearest 0.01 cfs)

	Slope (feet/foot)								
Depth (inches)	**0.004**	**0.007**	**0.01**	**0.02**	**0.03**	**0.05**	**0.07**	**0.10**	**0.12**
0.25	—	—	—	—	—	0.01	0.01	0.01	0.01
0.50	0.01	0.01	0.02	0.02	0.03	0.04	0.05	0.06	0.06
0.75	0.03	0.04	0.05	0.07	0.09	0.12	0.14	0.16	0.18
1.00	0.07	0.09	0.11	0.16	0.19	0.25	0.29	0.35	0.39
1.25	0.13	0.17	0.20	0.29	0.35	0.45	0.53	0.64	0.70
1.50	0.21	0.27	0.33	0.46	0.57	0.73	0.87	1.04	1.14
1.75	0.31	0.41	0.49	0.70	0.86	1.11	1.31	1.57	1.71
2.00	0.45	0.59	0.71	1.00	1.22	1.58	1.87	2.23	2.45
2.25	0.61	0.81	0.97	1.37	1.68	2.16	2.56	3.06	3.35
2.50	0.81	1.07	1.28	1.81	2.22	2.86	3.39	4.05	—
2.75	1.04	1.38	1.65	2.34	2.86	3.69	—	—	—
3.00	1.32	1.74	2.08	2.95	3.61	—	—	—	—
3.25	1.63	2.16	2.58	3.65	—	—	—	—	—
3.50	1.99	2.63	3.14	—	—	—	—	—	—
3.75	2.39	3.16	3.78	—	—	—	—	—	—
4.00	2.84	3.75	—	—	—	—	—	—	—

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Table 4-4
Visual Urban Output Table

FHWA Urban Drainage Design Program, HY-22
Drainage of Highway Pavements

Inlets on Grade
Date: 05/03/2006

Project No. : ______________________
Project Name: ______________________
Computed by : ______________________

Inlets on Grade: Gutter Flow Parameters

Roadway and Discharge Data

	Slope	**Composite**
S	Longitudinal Slope (ft/ft)	0.0100
Sx	Pavement Cross Slope (ft/ft)	0.0208
Sw	Gutter Cross Slope (ft/ft)	0.0652
n	Manning's Coefficient	0.016
W	Gutter Width (ft)	1.92
a	Gutter Depression (inch)	0.00
Q	Discharge (cfs)	1.740
T	Width of Spread (ft)	7.86

Gutter Flow

Eo	Gutter Flow Ratio	0.635
d	Depth of Flow (ft)	0.249
V	Average Velocity (ft/sec)	2.40

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City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 5: Culvert Hydraulics

Volume 3: Best Management Practices Manual

Revision: 0
June 2006

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Acronym List
(Chapter 5)

A	Culvert Area, (ft^2)
A	Average cross-sectional area of the tapered inlet
AHW	Allowable headwater depth
B	Span of box culvert
C_d	Discharge coefficient
C_r	Unsubmerged discharge coefficient
CMP	Corrugated Metal Pipe
d_c	Critical depth, (ft)
d_e	Equivalent hydraulic depth at outlet, (ft)
d_o	Depth at outlet, (ft)
D	Height of culvert opening, (ft)
EL_t	Elevation of the throat invert
EL_{hi}	Headwater elevation for inlet control, (ft)
EL_{hd}	Maximum design headwater
EL_{ho}	Headwater elevation for outlet control, (ft)
EL_i	Elevation of the culvert entrance, (ft)
EL_i	Inlet invert elevation
EL_{sf}	Elevation of the streambed at the face
EL_o	Elevation of the culvert outlet, (ft)
FHWA	Federal Highway Administration
g	Acceleration due to gravity, $(32.2\ ft/sec^2)$
h_o	Design tailwater, (ft)
H	Total head loss for outlet control, (ft)
H_1	Friction head loss in the tapered inlet, (ft)
HW	Headwater
HW_i	Headwater depth, for inlet control, (ft)
HW_r	Upstream depth
HW_o	Headwater depth for outlet control, (ft)
k_e	Entrance loss coefficients
K_tC_r	Discharge coefficient
L	Approximate length of curvet, (ft)
L	Length of roadway crest, (ft)
L	Barrel length, (ft)
L_1	Length of the tapered inlet, (ft)
L_1	Adjusted length
MDPW	Memphis Department of Public Works

n	Manning's roughness coefficient for sheet flow
N	Number of culvert
Q	Design Discharge
Q_o	Overtopping flow rate
R	Hydraulic radius of the culvert, (ft)
S_f	Fall slope
S_o	Barrel slope, (ft/ft)
S_o	Natural stream slope
TW	Tailwater depth, (ft)
USDOT	United States Department of Transportation
v	Average velocity, (ft/sec)

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5.0 CULVERT HYDRAULICS

5.1 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, 2001). Terminology is discussed in Volume 3.

5.2 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

5.2.1 Application

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

Cross Drain

A cross drain is a culvert placed transversely under roadway sections, usually with endwalls or some other type of end treatment. A cross drain is typically used where flow in an open channel water course (either natural or man-altered) intersects a roadway. After being confined and taken under the roadway, the flow is then returned to an open channel to proceed downstream. Because cross drain installations are normally under pavement, they should have soil-tight joints as a minimum to prevent soil migration. Leaking joints can cause uneven and differential settling of road surfaces or adjacent buildings.

Side Drain

This culvert is generally a pipe used in rural-type roadway projects without curb and gutter and running longitudinally (parallel to the roadway centerline) in roadway ditches under driveways or graded connections. Alignment joints rather than soil-tight joints are generally adequate for side drains.

5.2.2 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 most common materials for culvert barrels 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. Corrugated metal pipe may be allowed for use as a side drain, but only concrete pipe will be allowed for cross drains.

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 the City and/or County when other alternatives are available.

5.2.3 Size

Culverts should be sized using the nomographs presented in this chapter or available computer applications of them. 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.

5.2.4 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.

5.2.5 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 5.4.4) 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.

5.3 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

5.3.1 Return Period

The appropriate standard design-storm return periods (from Section 6.3.5 of Volume 1) are summarized below:

Cross Drains, residential collector and commercial	100 years
Cross Drains, all other residential	10 years
Side Drains, all streets-residential and commercial	10 years

For culverts designed for the 10 year storm, the cross-drain capacity should also be checked for the base flood (Q_{100}), to ensure that access is consistent with the roadway classification and that ponding at the entrance will not cause flooding of adjacent buildings. If ponding occurs at the entrance of the culvert, a reduction in the discharge may be appropriate. A reservoir routing procedure (see chapter 8) can be used to determine the discharge reduction attributable to storage.

5.3.2 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 depth for the design discharge should not exceed 1.5 times the diameter of a circular culvert, the height of a box culvert, or the rise of a non-circular pipe section.

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 or proposed drainage 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: Regulations).

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

5.3.3 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.

5.3.4 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.

5.3.5 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.

5.4 Design Calculations

The remaining portion of this chapter provides a detailed dialogue on the design of roadway culverts using the manual/graphical methods presented in HDS-5 (USDOT, FHWA, 2001). This dialogue is valuable in providing the designer a thorough understanding of the principles and techniques involved in roadway culvert design. Once this foundation is firmly established, the time and effort required to perform the manual calculations can be reduced significantly by the use of the computer software HY8 FHWA Culvert Analysis, available as a free download from the Federal Highway Administration Web site. While this software is a valuable tool, some of the input requirements tend to be somewhat cryptic to those inexperienced in the design of culverts using the manual methods. For that reason, a through discussion of these methods is given in the following sections of this chapter.

A flow chart for performing culvert design calculations is provided in Figure 5-1 (all figures are located at the end of this section) (see discussion in Section 5.4.1). 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, 2001) 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 Pip (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,2001).

5.4.1 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. The use of multiple barrel culverts is discouraged 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 5.4.2). A trial size is determined by assuming HW_i/D (e.g., $HW_i/D = 1.2$) 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 5.4.2 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 5.3.2 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:

$$EL_{hi} = HW_i + EL_i \tag{5-2}$$

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 the included Figures and procedures discussed in Section 5.4.3.

(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 5.3.3).

(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, 2001) 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:

$$HW_o \le D + (1 + k_e)\frac{v^2}{2g} \tag{5-5}$$

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:

$$EL_{ho} = EL_o + H + h_o \tag{5-6}$$

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 5.4.4.

9. If roadway overtopping occurs, capacity calculations can be performed following the procedure in Section 5.4.5.

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.

5.4.2 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.

5.4.3 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

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 MDPW discourages the use of multiple barrel culverts when other alternatives are available. Read head in feet (5-8) H scale located on the far right. For values beyond the limit of the printed scales, find H by solving the equation:

$$H = \left[1 + k_e + \frac{29n^2L}{R^{1.33}}\right]\frac{v^2}{2g}$$

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 5.4.1, 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, 2001), 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.

5.4.4 Tapered Inlets

Tapered inlet nomographs from HDS-5 (USDOT, FHWA, 2001) 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 5.4.1).

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) $D/4 \leq$ 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.

5.4.5 Roadway Overtopping (presented for information only, overtopping generally not permissible for design flows)

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 5.4.2 and 5.4.3).

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.

5.4.6 Example Problems

The following example problems demonstrate the procedure required to design culverts as presented in Section 5.4.1 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.

Example 5-1. Circular Pipe Culvert

A culvert at a new roadway crossing must be designed to pass the 50-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 2⅔ by one-half 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. ***Inlet Control Calculations:*** 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. ***Outlet Control Calculations:*** 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 5.4.1, 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 5-7. 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 50-year flow rate of 300 cfs. The following information is known:

Maximum design headwater = ELhd = 110.0 feet, based on adjacent structures
Roadway shoulder elevation = 113.5 feet
Elevation of streambed at culvert face = ELsf = 100 feet
Natural stream slope = So = 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 400, tail water = 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 5.4.5 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 as the product of the appropriate C_r and K_t values 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 a tapered inlet is described in Section 5.4.4. 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, 5-42 and 5-43.

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/D 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

Example 5-5. Concrete Box Culvert (design from Example 5-2. redone using HY8 software)

A culvert at a new roadway crossing will be designed to carry the 50-year flow rate of 300 cfs.

The following information is known:

Maximum design headwater = ELhd = 110.0 feet, based on adjacent structures
Roadway shoulder elevation = 113.5 feet
Elevation of streambed at culvert face = ELsf = 100 feet
Natural stream slope = So = 0.02 foot/foot
Tail water depth = TW = 4 feet
Approximate culvert length = L = 200 feet

Design a reinforced concrete box culvert for this installation. Assume square edges in a 90-degree headwall. The inlet is to be set at the existing channel elevation (no fall) of 100.

1. From the beginning menu screen, the user opts to either edit an existing file or create a new one. Once the new file is created and given a date, the menu asks for a "design" and a "maximum" flow. In this case, Q_{50} of 300 cfs and Q_{100} of 350 cfs are entered for these values. The software defaults to a minimum flow of zero, but this can be edited to a larger value if the user desires to limit the output to a more restricted range of discharges.

2. The next prompt asks whether data will be provided for the proposed culvert or embankment toes. In this example, culvert data is selected and the user provides stations and elevations for both ends of the culvert allowing the software to establish length and slope of the culvert being evaluated. From the assumptions given above, the inlet station is input as station 0, the inlet elevation as 100.0, the outlet station as 200 based on the assumed length, and the outlet elevation is calculated as 100.0 — (200 x 0.02 ft./ft.) = 96.0. The software defaults to the assumption of one barrel, but can be edited for multiple barrels.

3. On the next screen, the user selects the shape of the structure to be evaluated, such as circular or arch pipe, box culvert, etc. In this case a concrete box culvert is selected and the user provides the culvert span and rise in feet, in this case 6 feet and 5 feet, respectively.

4. The next step requires the selection of an inlet type (conventional), entrance treatment (square edge), and inlet depression (none). The next screen allows the user to verify input and modify it if necessary. The default Manning's "n" value is 0.012 for concrete, so this will need to be modified to 0.013 to comply with City/County standard assumptions.

5. The user must now identify the tail water conditions that exist at the culvert outlet. The choices include input of geometry for a rectangular, trapezoidal, triangular, or irregular shaped outlet channel for which the software will calculate tail water depth using Manning's Equation (following input of channel slope and friction coefficient). The user also has the option to input a known rating curve for the outlet having up to 11 pairs of discharge-elevation values. The final option for this screen is the specification of a known tail water depth shown as an elevation relative to culvert outlet elevation. The assumptions for this example call for a tail water depth of 4.0 feet, which would be input as an elevation of 100.0 (= 96.0 + 4.0).

6. The next input screen asks for definition of the roadway profile as either constant elevation or an irregular profile. This information is used to calculate overflow over the roadway surface (overtopping). Since this is a design exercise and overtopping will not be allowed, the "constant elevation" option is selected. The screen then prompts for input of the length of roadway to be overtopped and the overtopping elevation. Since overtopping will not be

allowed, an arbitrary length of 1 foot is used and the crest elevation is shown as 113.5, the shoulder elevation from the assumptions given above.

7. The user must then indicate the type of roadway pavement (paved or gravel) or specify a discharge coefficient for use in the broad-crested weir formula. For this example a paved surface is selected, which then requires the input of "top width" or the distance across which the overtopping flow would travel (normally the shoulder-to-shoulder width of the roadway embankment), in this case assumed to be 108 feet.

8. At this point the software allows the user to go back and review or edit any of the previous input by selecting the appropriate item or to save the input and proceed to making the calculations. The user is then returned to the main menu and, by selecting "Calculate," completes the calculation of headwater elevations for the range of discharges specified. The primary output table is shown below:

CURRENT TIME: 10:34:04 **FILE DATE: 02-13-2006**
FILE NAME: EXMPL5-5

PERFORMANCE CURVE FOR CULVERT 1-1 (6.00 (ft) BY 5.00 (ft)) RCB

Dis-Charge (Cfs)	Head-Water Elev. (Ft)	Inlet Control Depth (Ft)	Outlet Control Depth (Ft)	Normal Depth (Ft)	Crit. Depth (Ft)	Outlet Depth (Ft)	Tw Depth (Ft)	Outlet Vel. (Fps)
250.00	106.62	6.62	6.62	2.07	3.79	2.29	4.00	18.23
260.00	106.86	6.86	6.86	2.13	3.89	2.38	4.00	18.24
270.00	107.10	7.10	7.10	2.19	3.99	2.45	4.00	18.40
280.00	107.35	7.35	7.35	2.25	4.08	2.51	4.00	18.57
290.00	107.61	7.61	7.61	2.30	4.18	2.58	4.00	18.74
300.00	107.87	7.87	7.87	2.36	2.28	2.66	4.00	18.83
310.00	108.15	8.15	8.15	2.42	4.37	2.73	4.00	18.93
320.00	108.43	8.43	8.43	2.48	4.46	2.79	4.00	19.09
330.00	108.72	8.72	8.72	2.54	4.56	2.86	4.00	19.26
340.00	109.02	9.02	9.02	2.59	4.65	2.94	4.00	19.29
350.00	109.33	9.33	9.33	2.65	4.74	3.01	4.00	19.39

El. inlet face invert 100.00 ft El. outlet invert 96.00 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft.

Site Data	**Culvert Invert**
Inlet Station	0.00 ft
Inlet Elevation	100.00 ft
Outlet Station	200.00 ft
Outlet Elevation	96.00 ft
Number of Barrels	1
Slope (V/H)	0.0200
Culvert Length Along Slope	200.04 ft

Culvert Data Summary	
Barrel Shape	Box
Barrel Span	6.00 ft
Barrel Rise	5.00 ft
Barrel Material	Concrete
Barrel Manning's n	0.013
Inlet Type	Conventional
Inlet Edge and Wall	Square Edge (90-45 deg.)
Inlet Depression	None

9. Inspection of the output table shows that for the design flow (Q_{50}) of 300 cfs the headwater elevation is 107.87, which checks against the 107.9 from Example 5-2. As in Example 5-2, this is below the allowable elevation of 110.0, so the selection was made on the menu to “edit” the culvert data to use a 5-foot by 5-foot box and. When rerun with this change, the software computed a headwater elevation of 109.65, again checking against the hand version answer of 109.6.

For the convenience of the manual user, several of the figures containing information for the calculations discussed herein have been reproduced below.

5.5 Chapter Equations

$$A = \frac{Q}{v} \tag{5-1}$$

Where:

A	=	Culvert area, in square feet
Q	=	Design discharge, in cfs
v	=	Average velocity, in feet/second

$$EL_{hi} = HW_i + EL_i \tag{5-2}$$

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

$$HW_o = H + h_o - LS_o \tag{5-3}$$

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

$$d_e = \frac{d_c + D}{2} \tag{5-4}$$

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

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

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

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

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

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

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

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 5.4.1, 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

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

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

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

Where:

Q_o	=	Overtopping flow rate, in cfs
C_d	=	${}_tC_r$ = 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

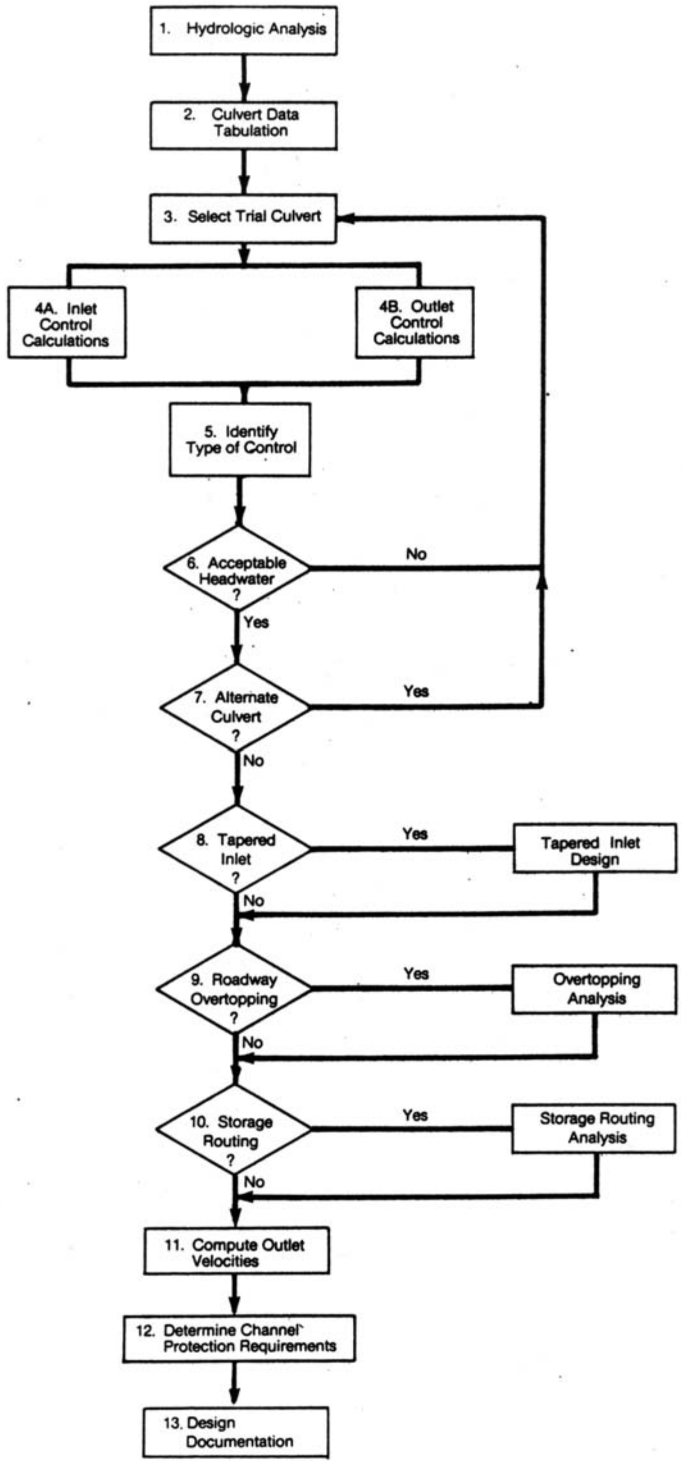


Figure 5-1
Culvert Design Flow Chart

Back to Section 5.4, 5.4.1, 5.4.6

PROJECT: ____________________

STATION: ____________ **SHEET** ____ **OF** ____

CULVERT DESIGN FORM

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

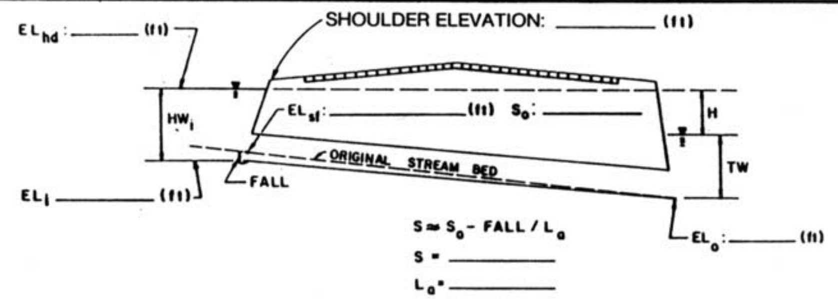
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: ____________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ____________
- ☐ ROUTING: ________ ☐ OTHER: ________

DESIGN FLOWS/TAILWATER

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



$S \approx S_o - FALL / L_a$
S = ______
L_a = ______

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

Back to Section 5.4, 5.4.1, 5.4.4, 5.4.6

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

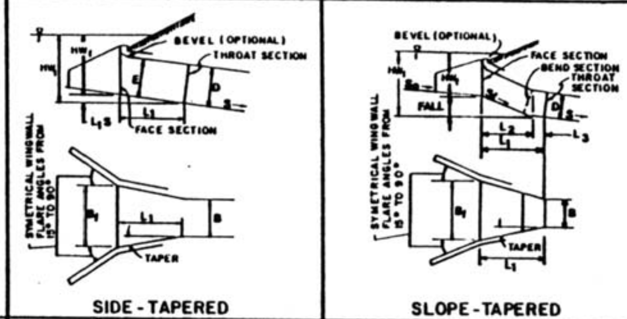
SLOPE OF BARREL , S = ________ ft/ft

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

BARREL SHAPE AND MATERIAL : ______________

N = __________ , B = __________ , D = __________

INLET EDGE DESCRIPTION ______________



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)	SLOPE-TAPERED ONLY: L_2 (7)	SLOPE-TAPERED ONLY: CHECK L_2 (8)	SLOPE-TAPERED ONLY: ADJ. L_3 (9)	SLOPE-TAPERED ONLY: ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	SIDE-TAPERED W/ FALL: HW_c (12)	SIDE-TAPERED W/ FALL: 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 $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot \text{TAPER} - L_3$

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

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

(11) SIDE - TAPERED : $L = \left[\frac{B_f - NB}{2}\right] \cdot \text{TAPER}$
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

Back to Section 5.4, 5.4.4, 5.4.6

PROJECT :	STATION :	MITERED INLET DESIGN FORM
	SHEET ____ OF ____	DESIGNER / DATE : ____ / ____
		REVIEWER / DATE : ____ / ____

DESIGN DATA : N ____ ; B ____ ; D ____

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

EL. THROAT INVERT ____ ft EL. STREAM BED AT CREST ____ ft

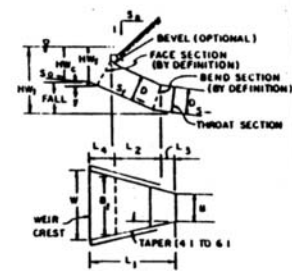
FALL ____ ft ; TAPER ____ : (4:1 TO 6:1)

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)	HW_f / E (4)	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

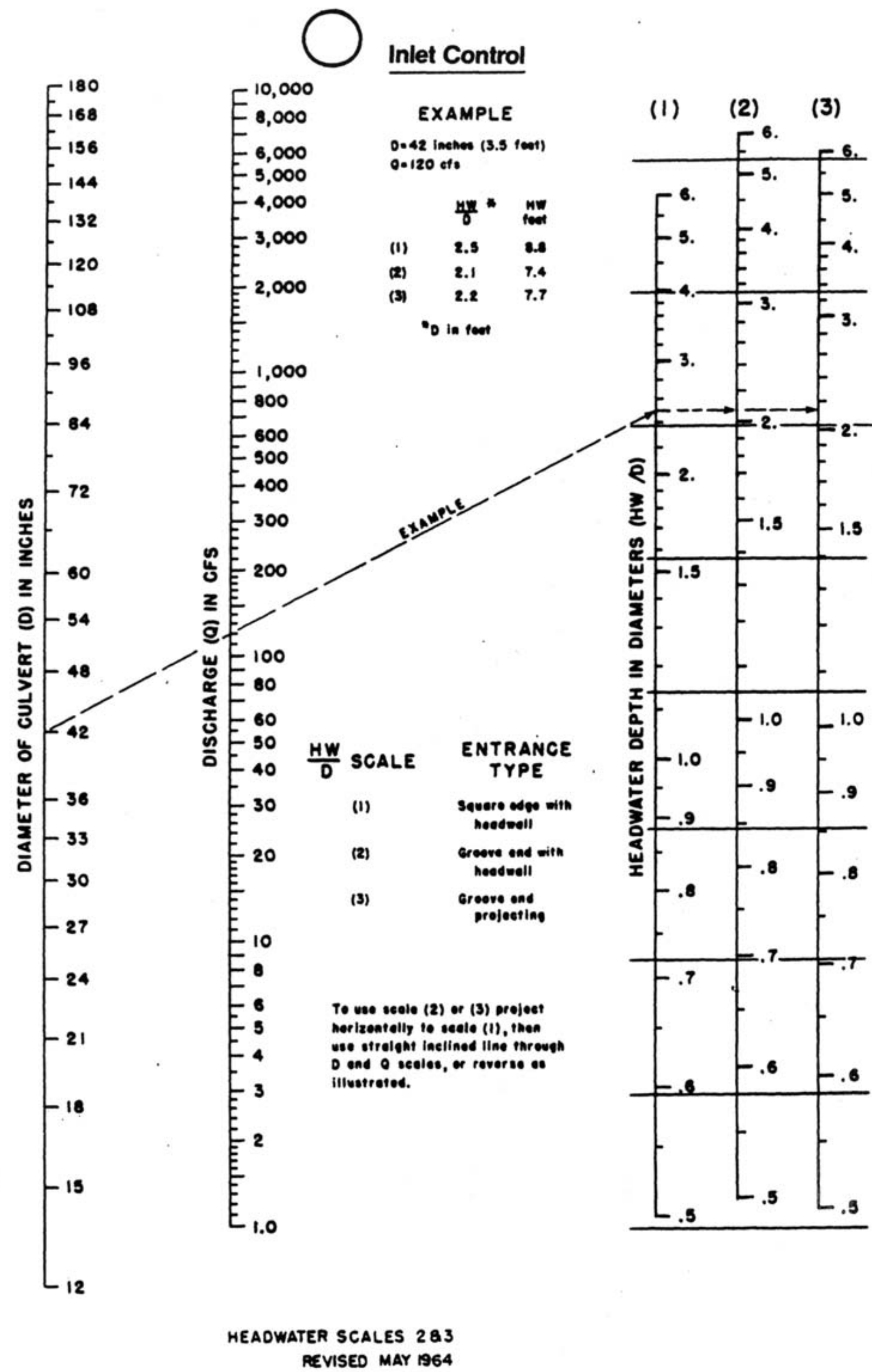
TAPER ____ : 1

S_f ____ : 1

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

Figure 5-4
Mitered Inlet Design Form

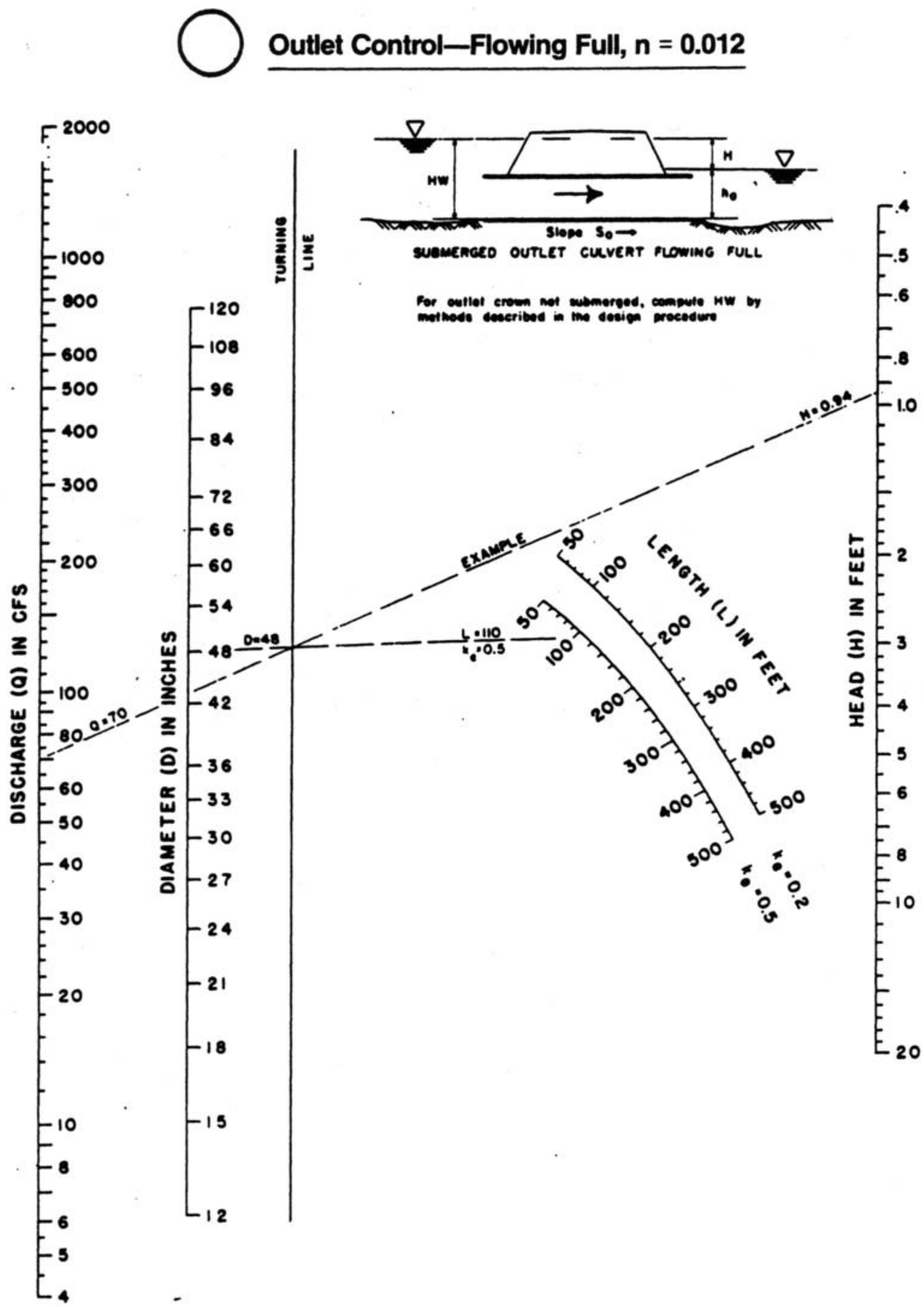
Back to Section 5.4, 5.4.4



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

Figure 5-5
Inlet Control Chart for Circular Concrete Pipe Culverts

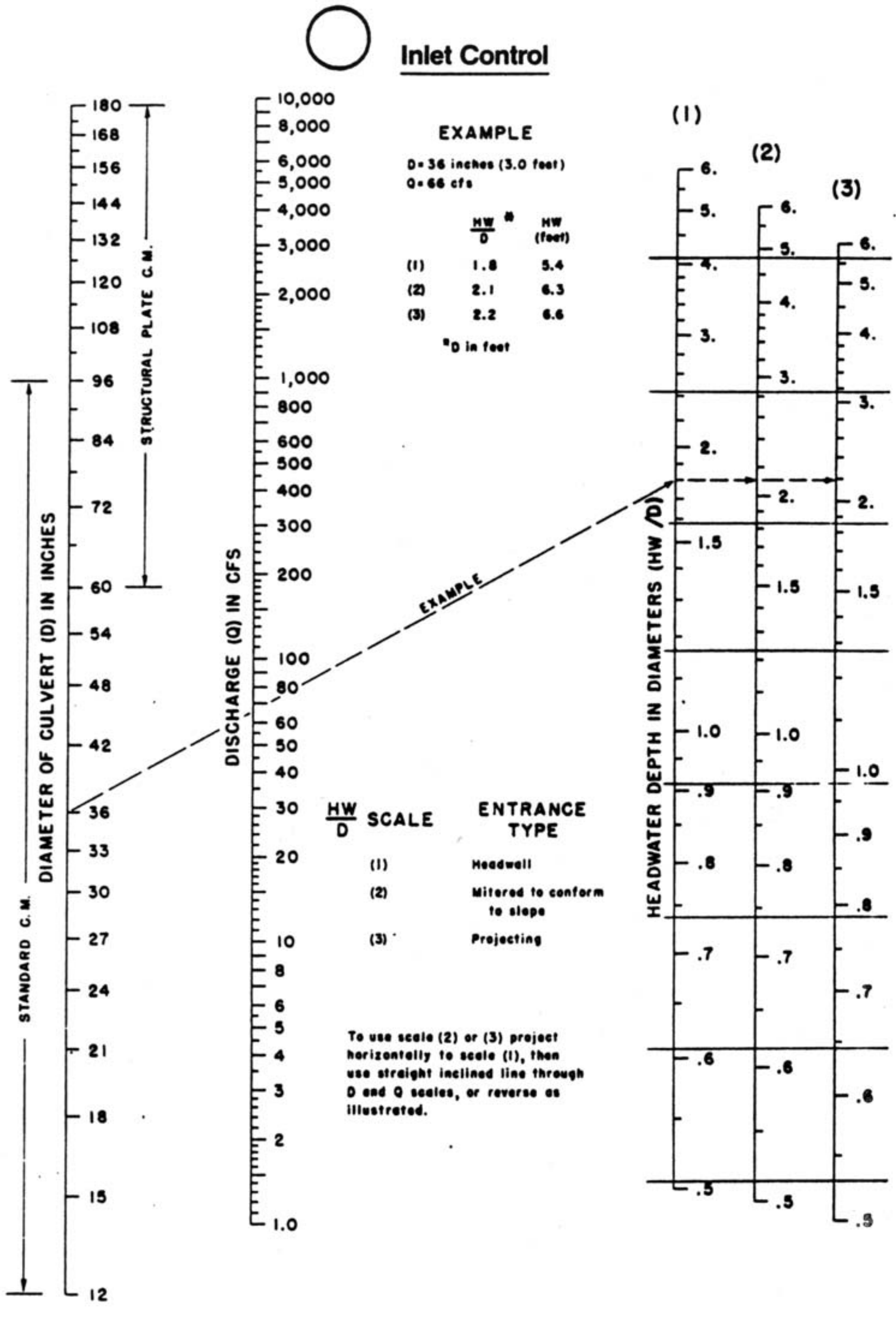
Back to Section 5.4.6



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

Figure 5-6
Outlet Control Chart for Circular Concrete Pipe Culverts Flowing Full

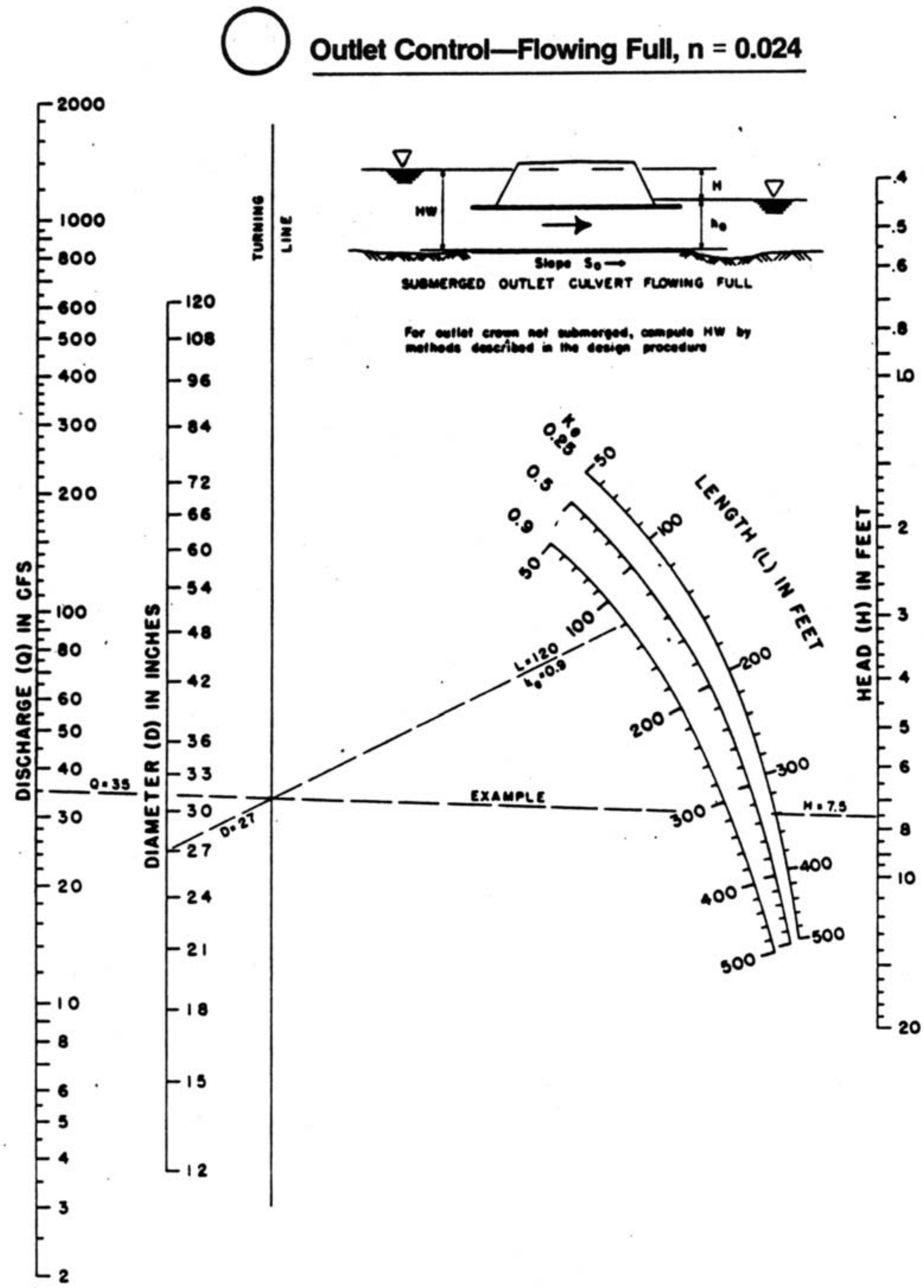
Back to Section 5.4.6



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

Figure 5-7
Inlet Control Chart for Circular CMP Culverts

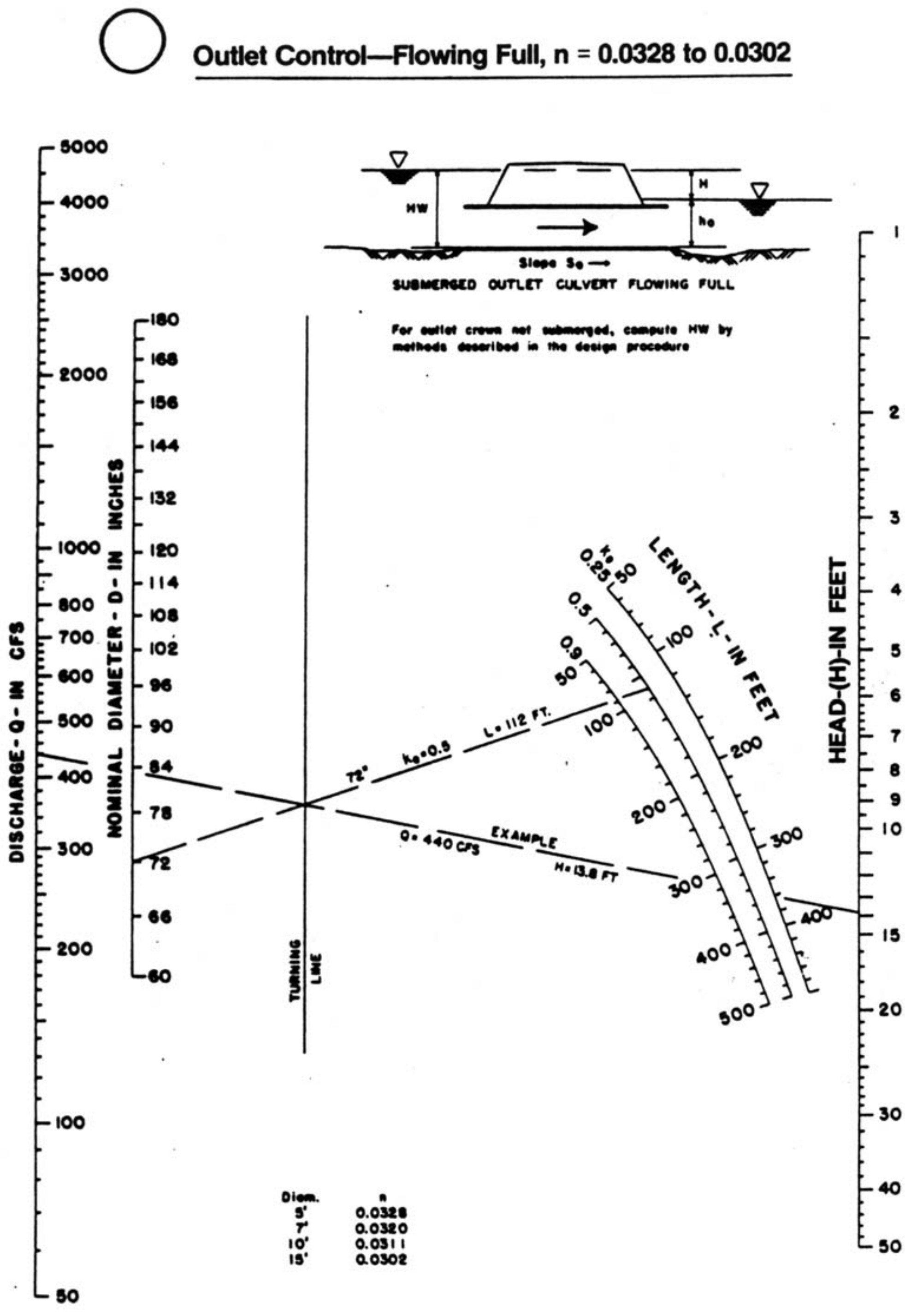
Back to Section 5.4.6



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

Figure 5-8
Outlet Control Chart for Circular CMP Culverts Flowing Full

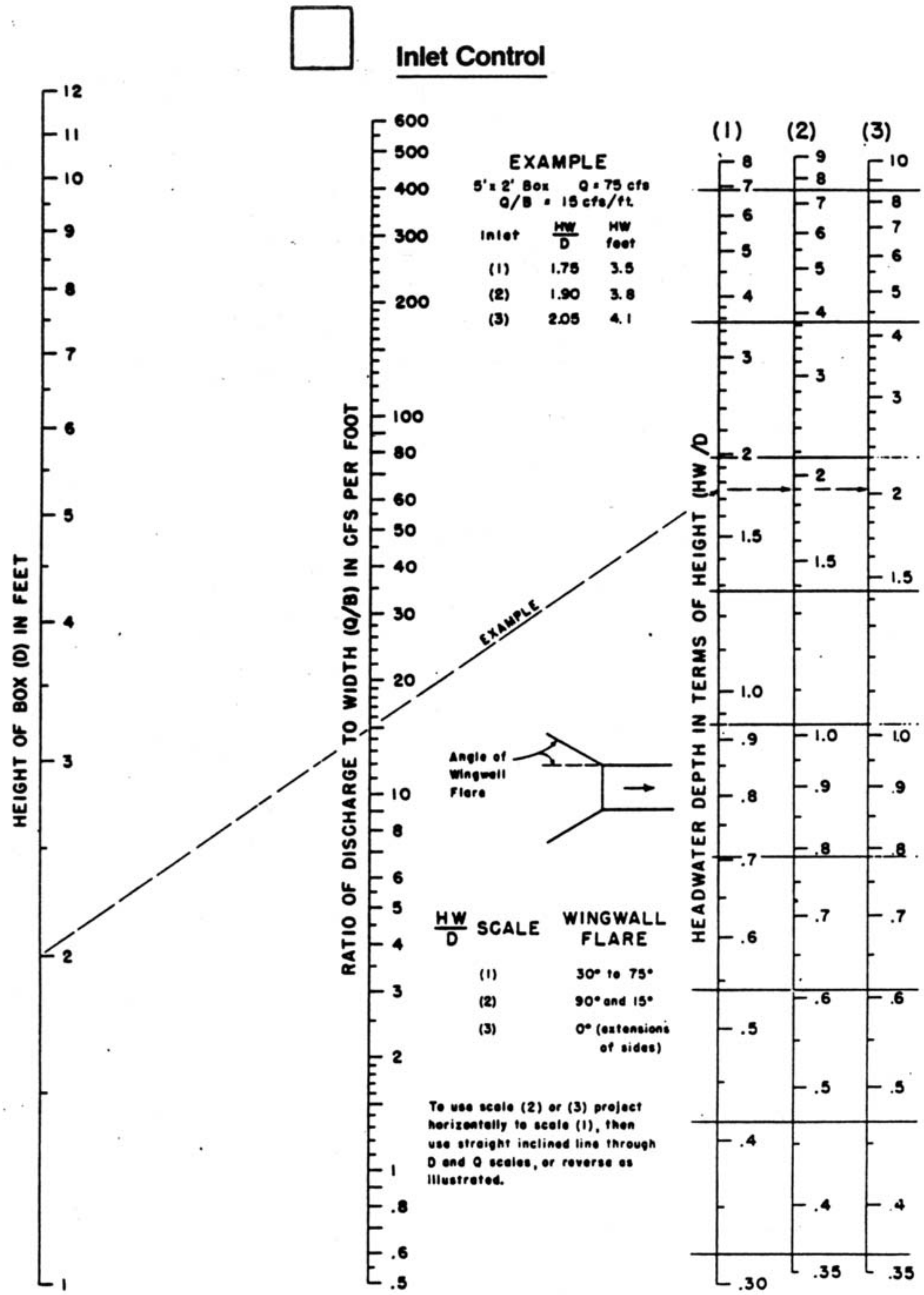
Back to Section 5.4.6



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

Figure 5-9
Outlet Control Chart for Structural Plate
CMP Culverts Flowing Full

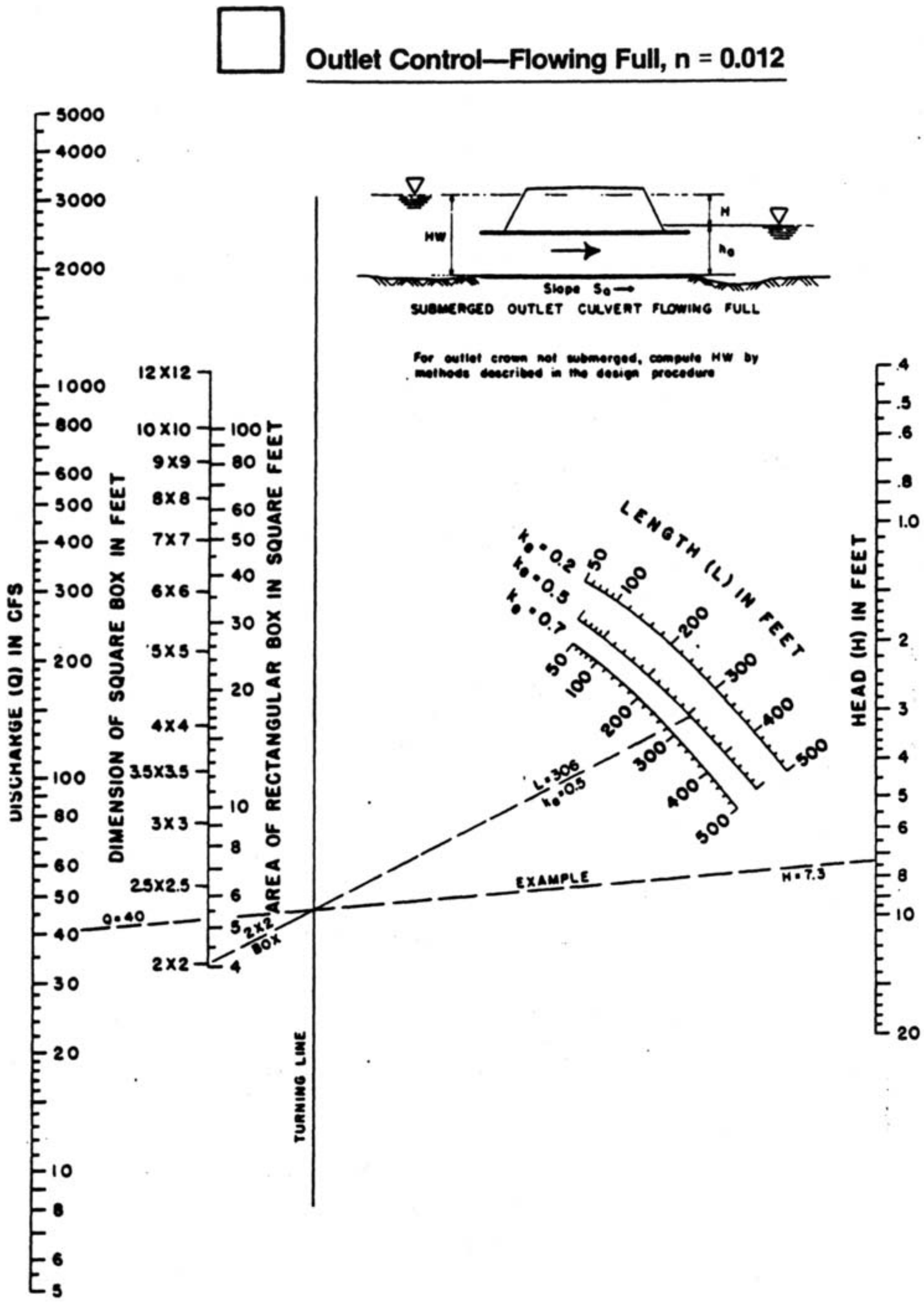
Back to Section 5.4



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

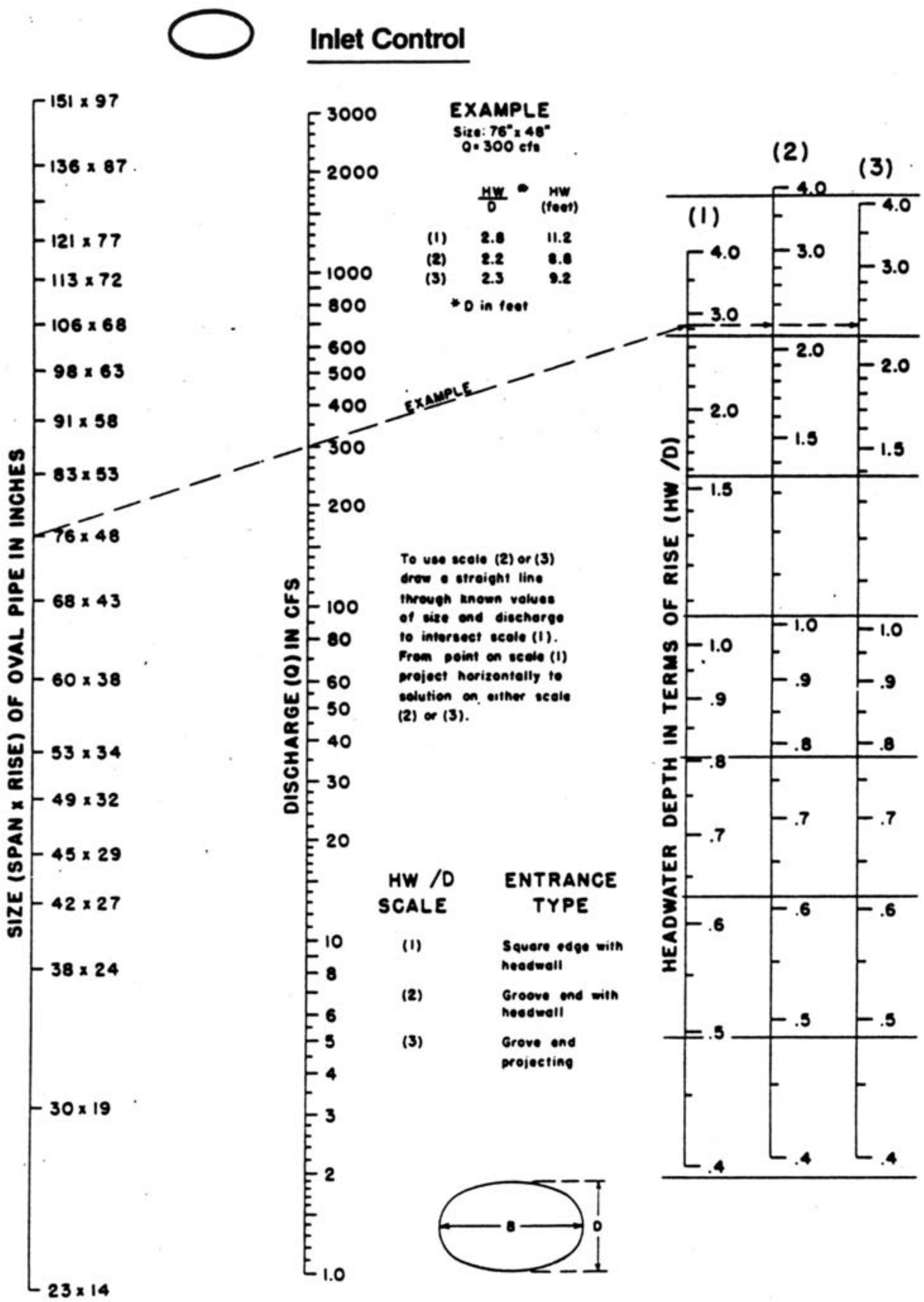
Figure 5-10
Inlet Control Chart for Concrete Box Culverts

Back to Section 5.4.6



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

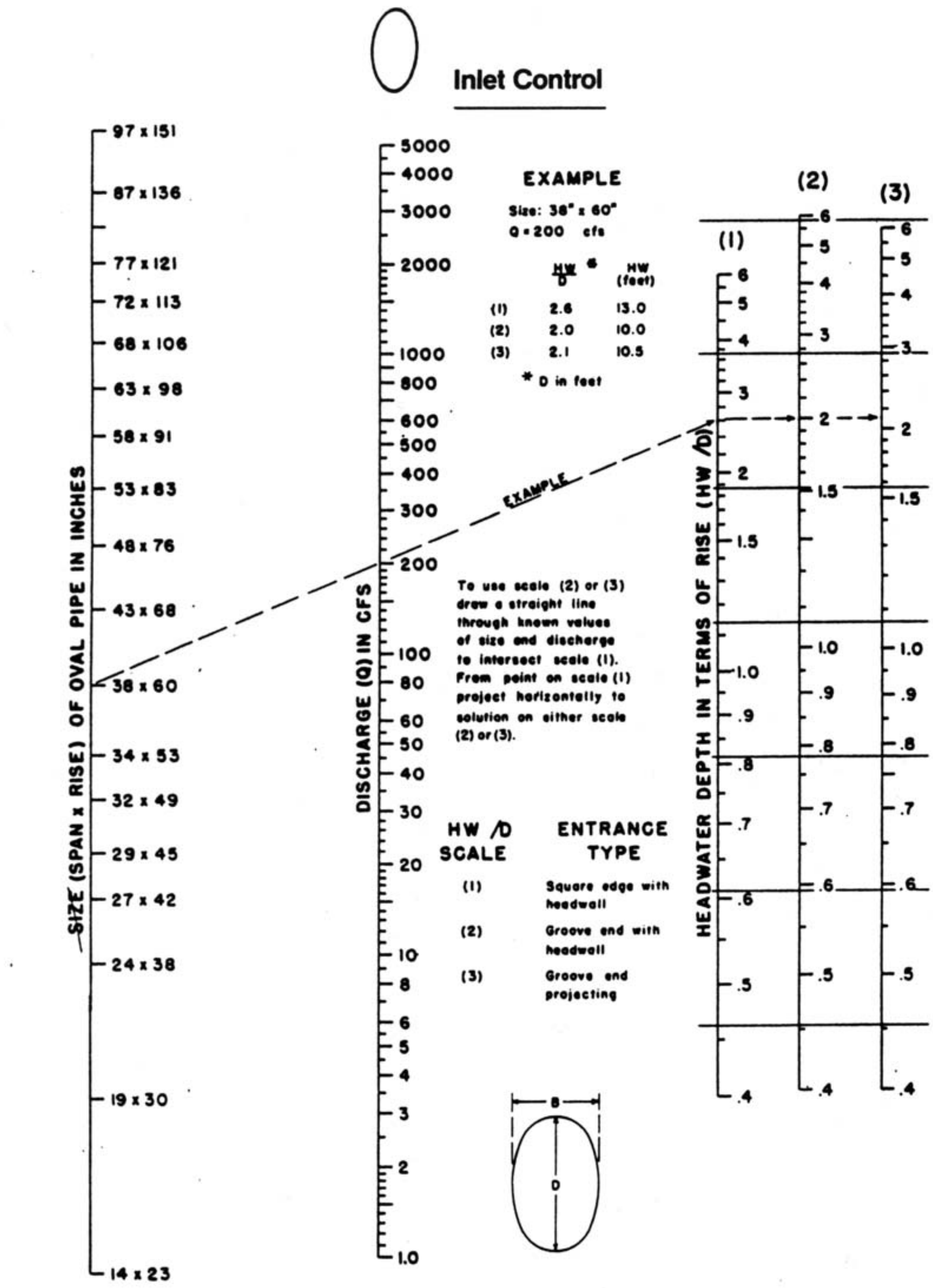
Figure 5-11
Outlet Control Chart for Concrete Box Culverts Flowing Full Back to Section 5.4.3 , 5.4.6



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

Figure 5-12
Inlet Control Chart for Oval Concrete Pipe Culverts – Long Axis Horizontal

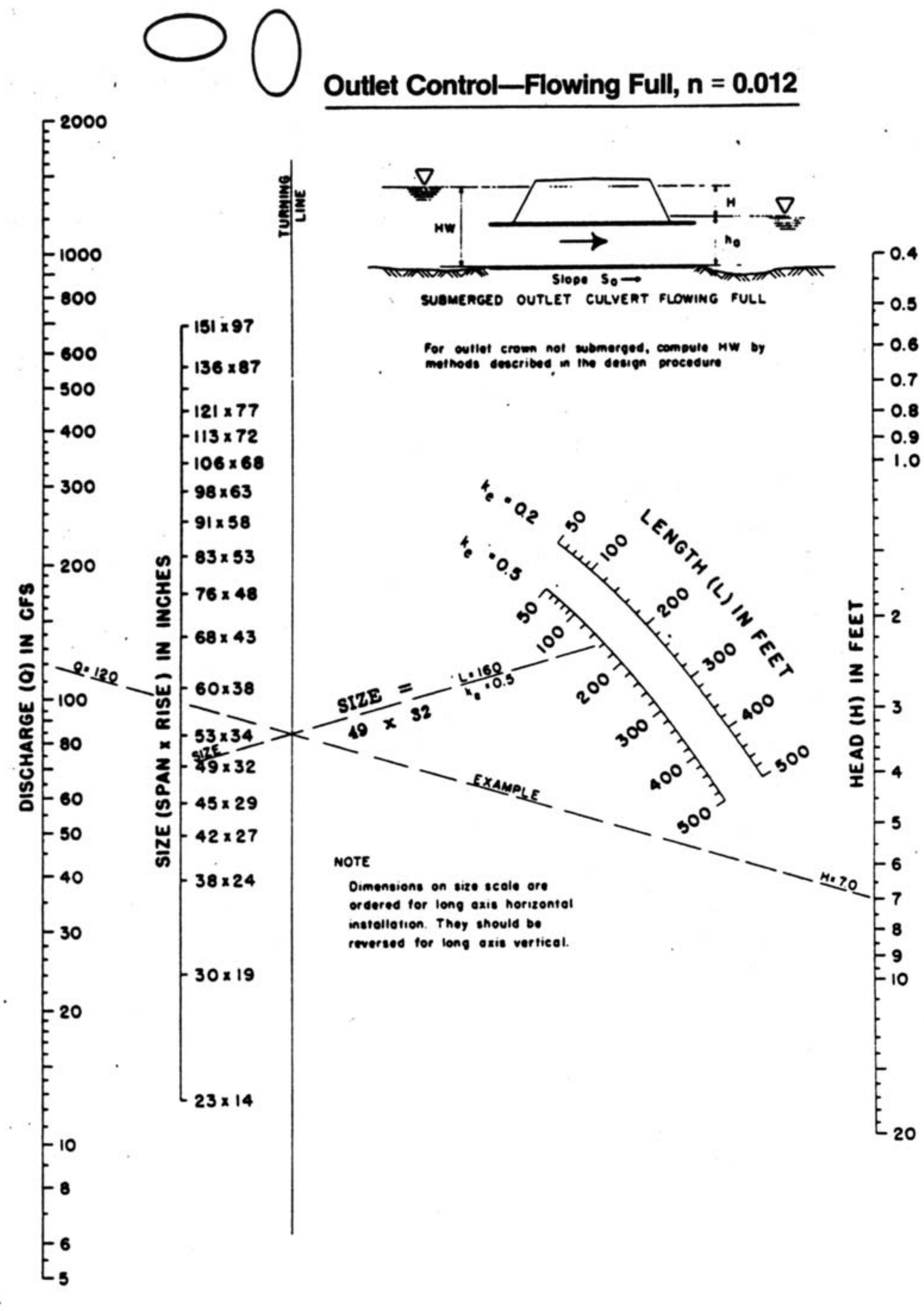
Back to Section 5.4



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

Figure 5-13
Inlet Control Chart for Oval Concrete Pipe Culverts – Long Axis Vertical

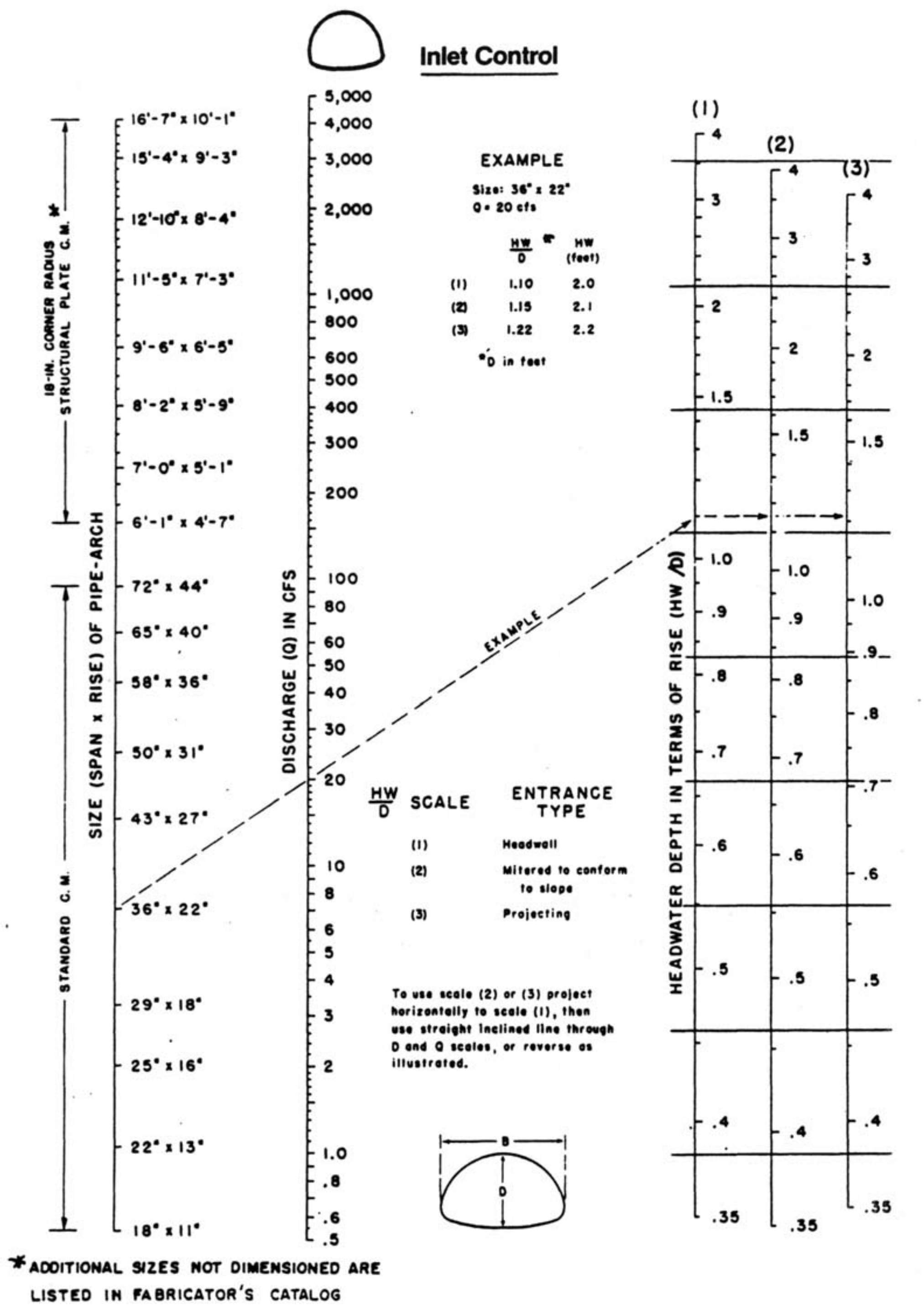
Back to Section 5.4



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

Figure 5-14
Outlet Control Chart for Oval Concrete Pipe Culverts Flowing Full – Long Axis Horizontal or Vertical

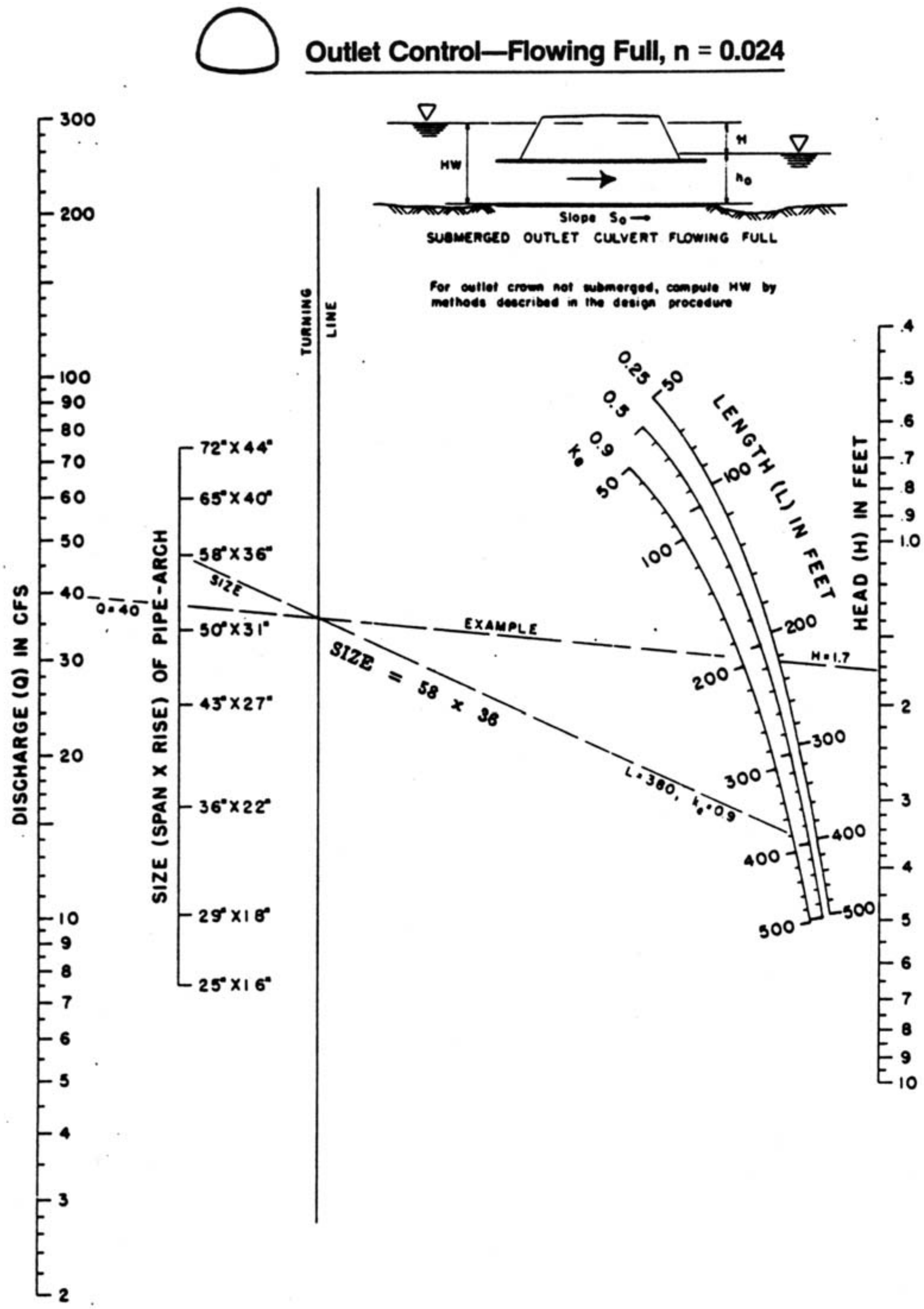
Back to Section 5.4



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

Figure 5-15
Inlet Control Chart for CMP Arch Culverts

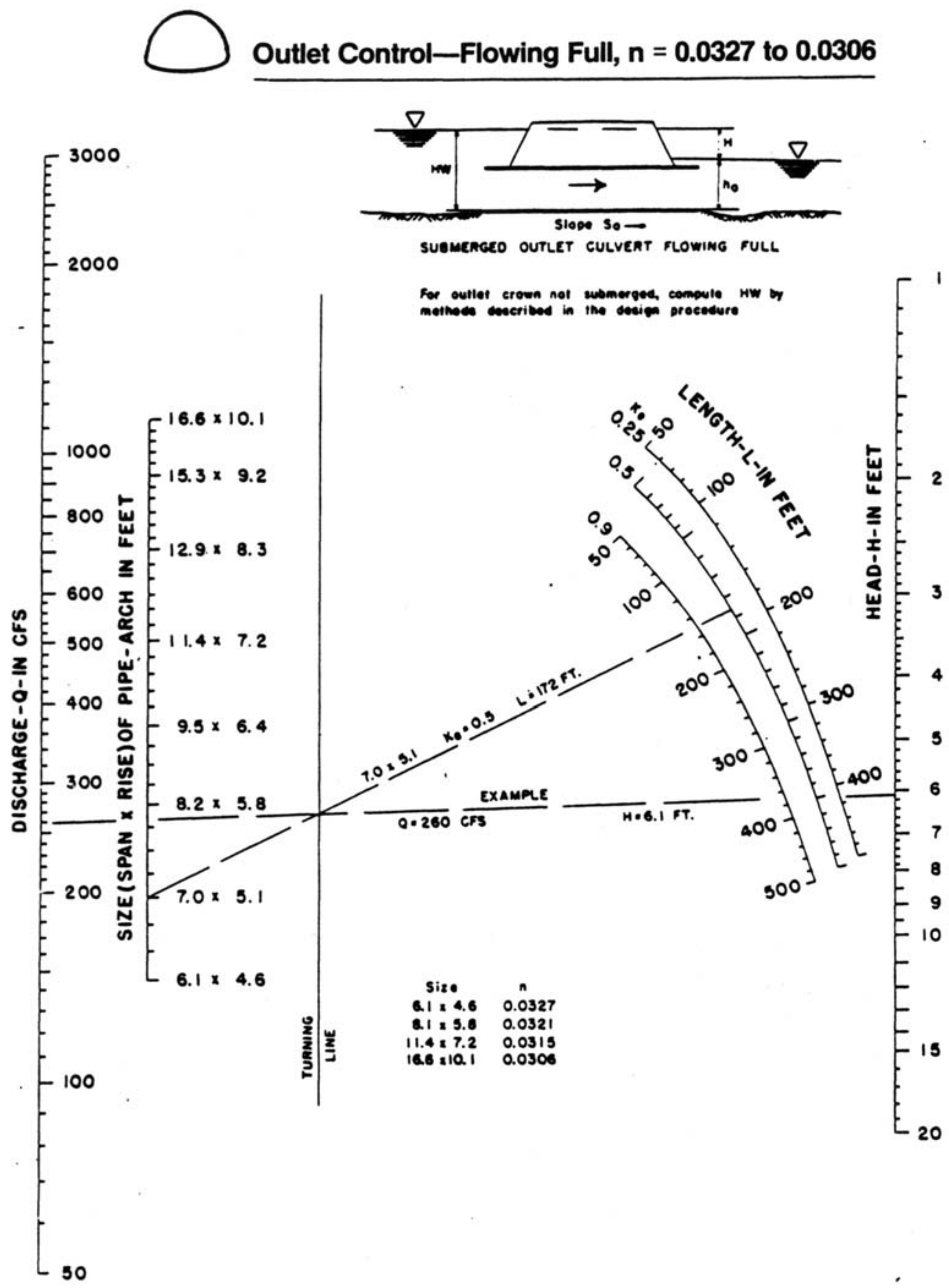
Back to Section 5.4



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

Figure 5-16
Outlet Control Chart for CMP Arch Culverts Flowing Full

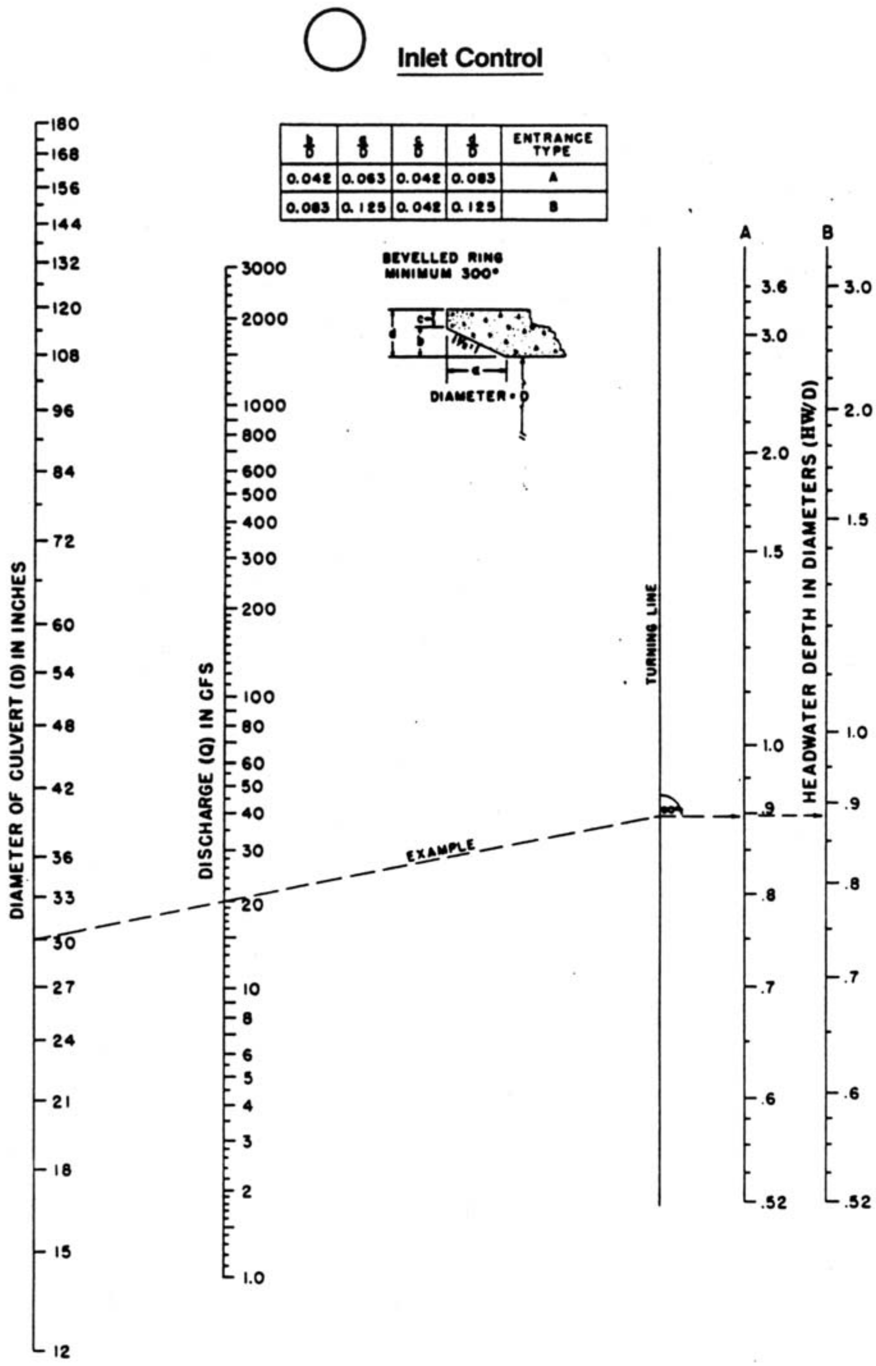
Back to Section 5.4



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

Figure 5-17
Outlet Control Chart for Structural Plate CMP Arch Culverts
(18-inch Corner Radius Flowing Full)

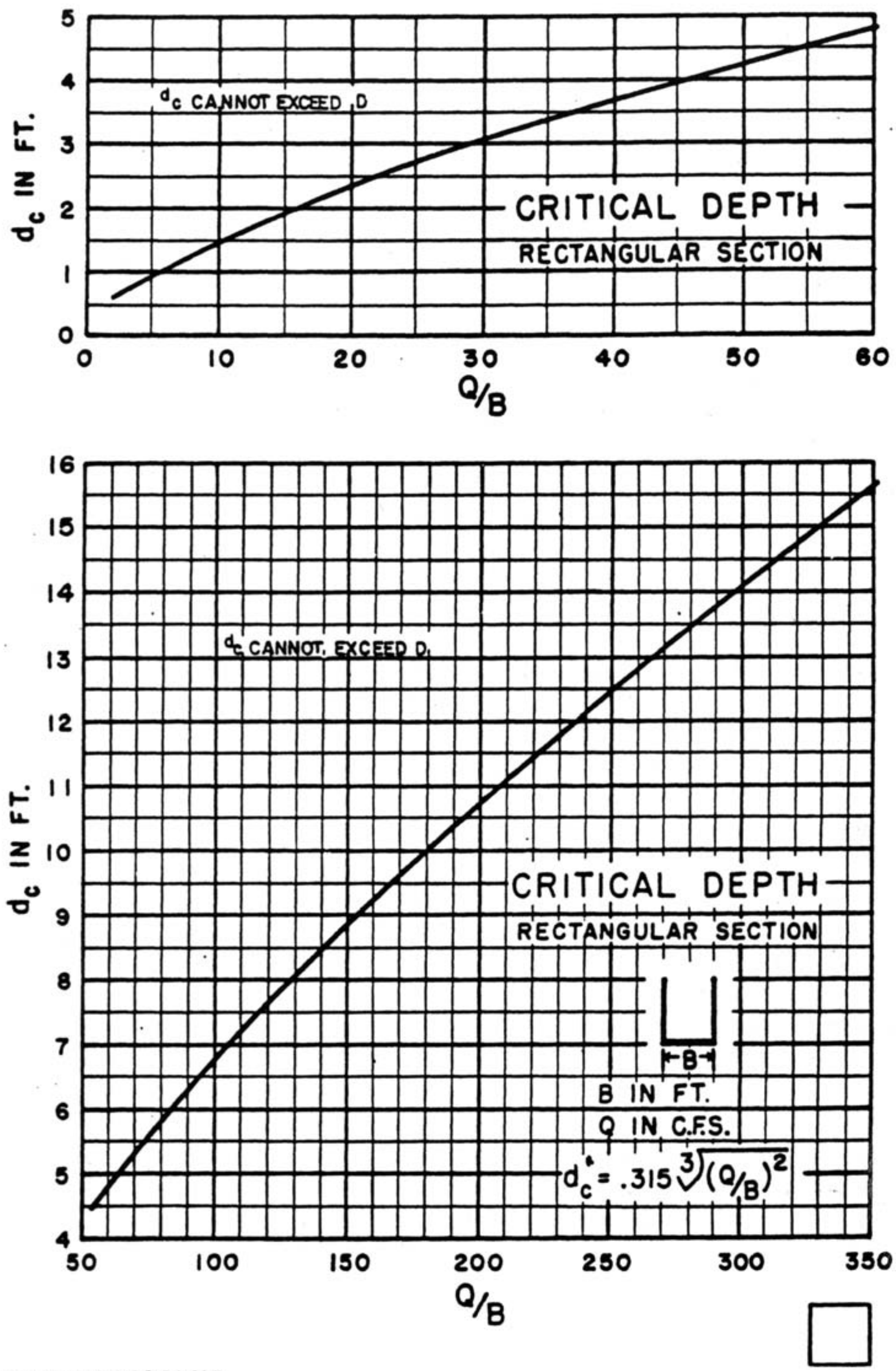
Back to Section 5.4



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

Figure 5-18
Inlet Control Chart for Circular Pipe Culverts with Beveled Ring

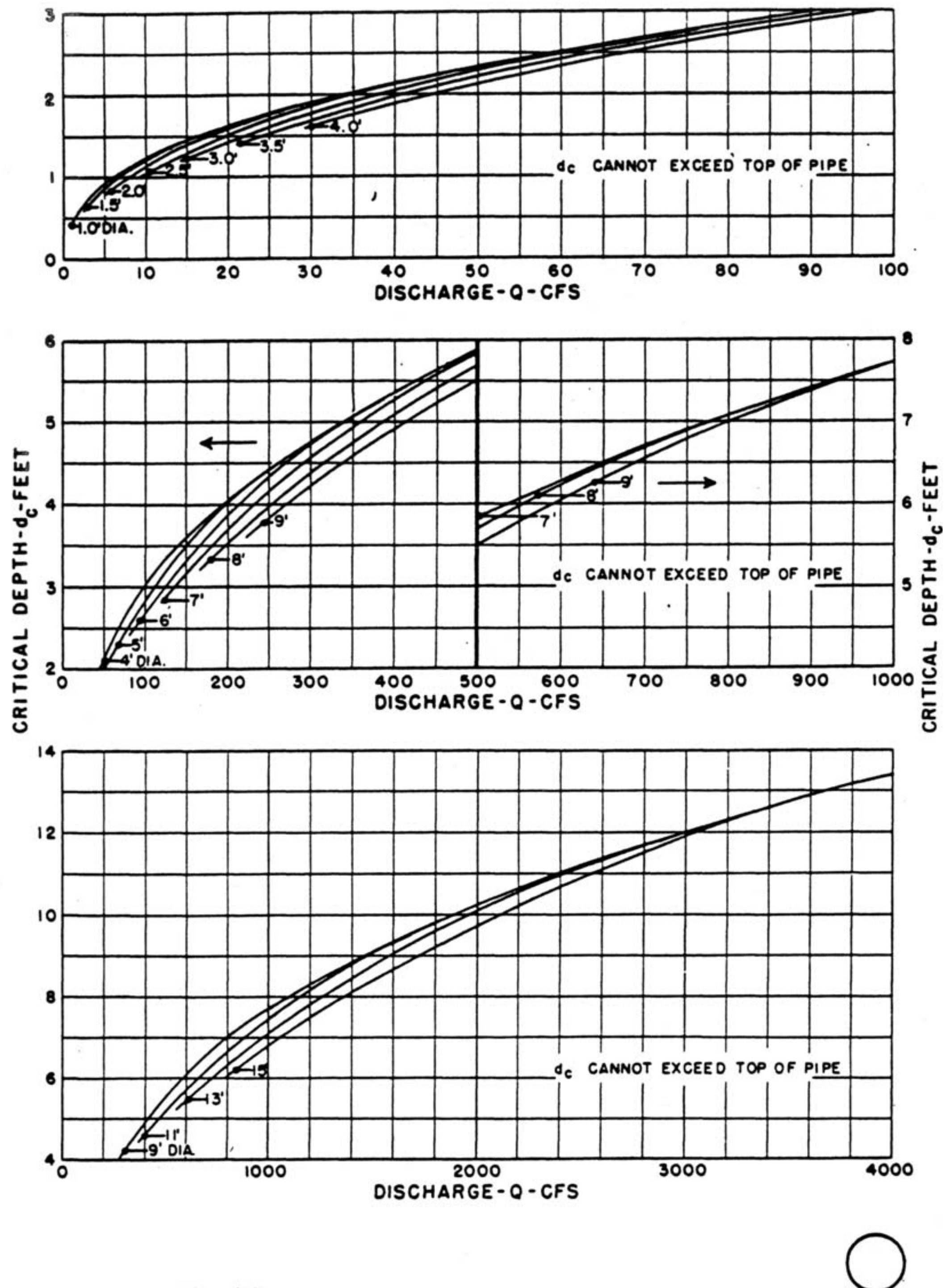
Back to Section 5.4.6



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

Figure 5-19
Critical Depth Chart for Rectangular Sections

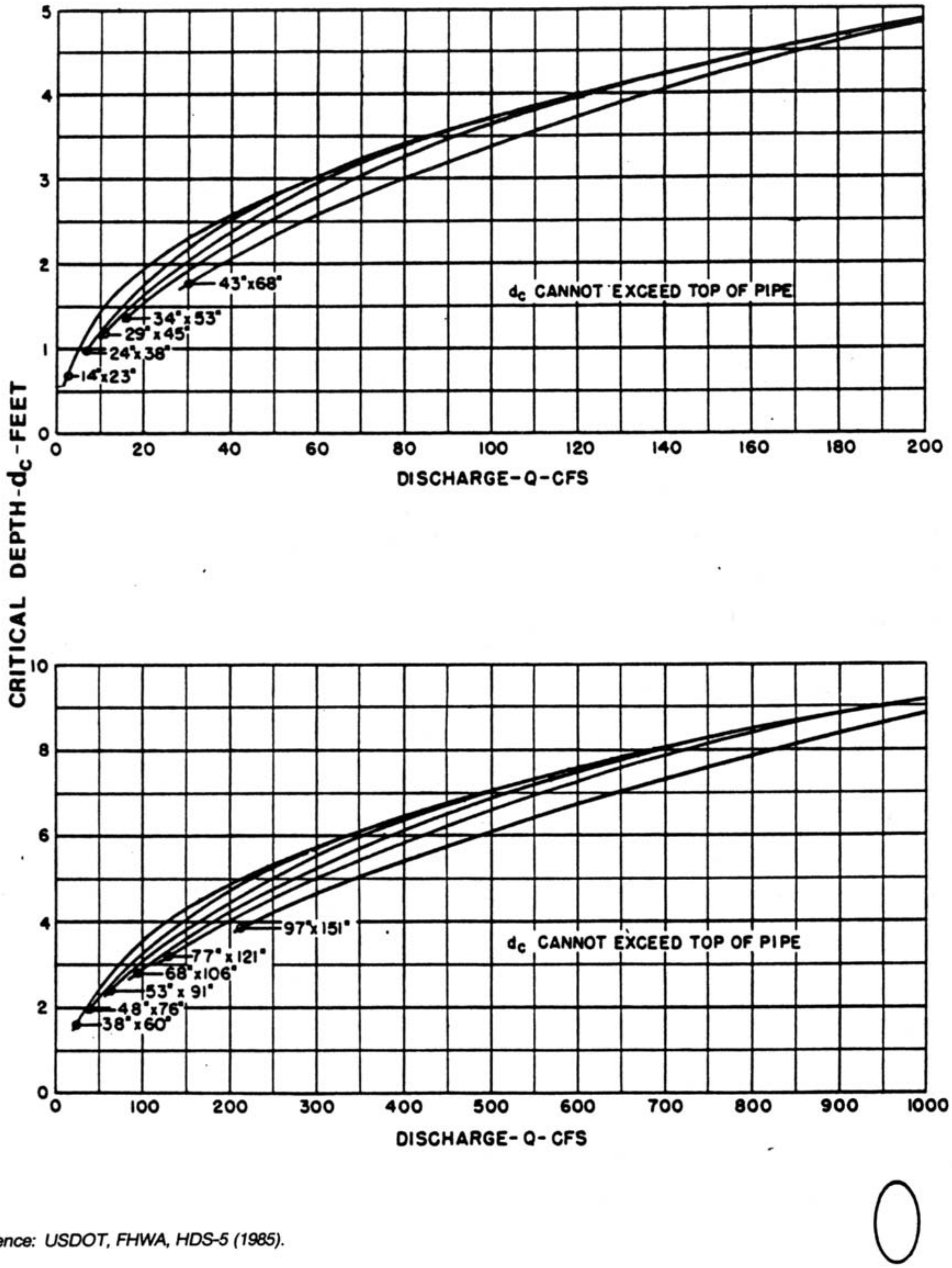
Back to Section 5.4.1, 5.4.6



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

Figure 5-20
Critical Depth Chart for Circular Pipe

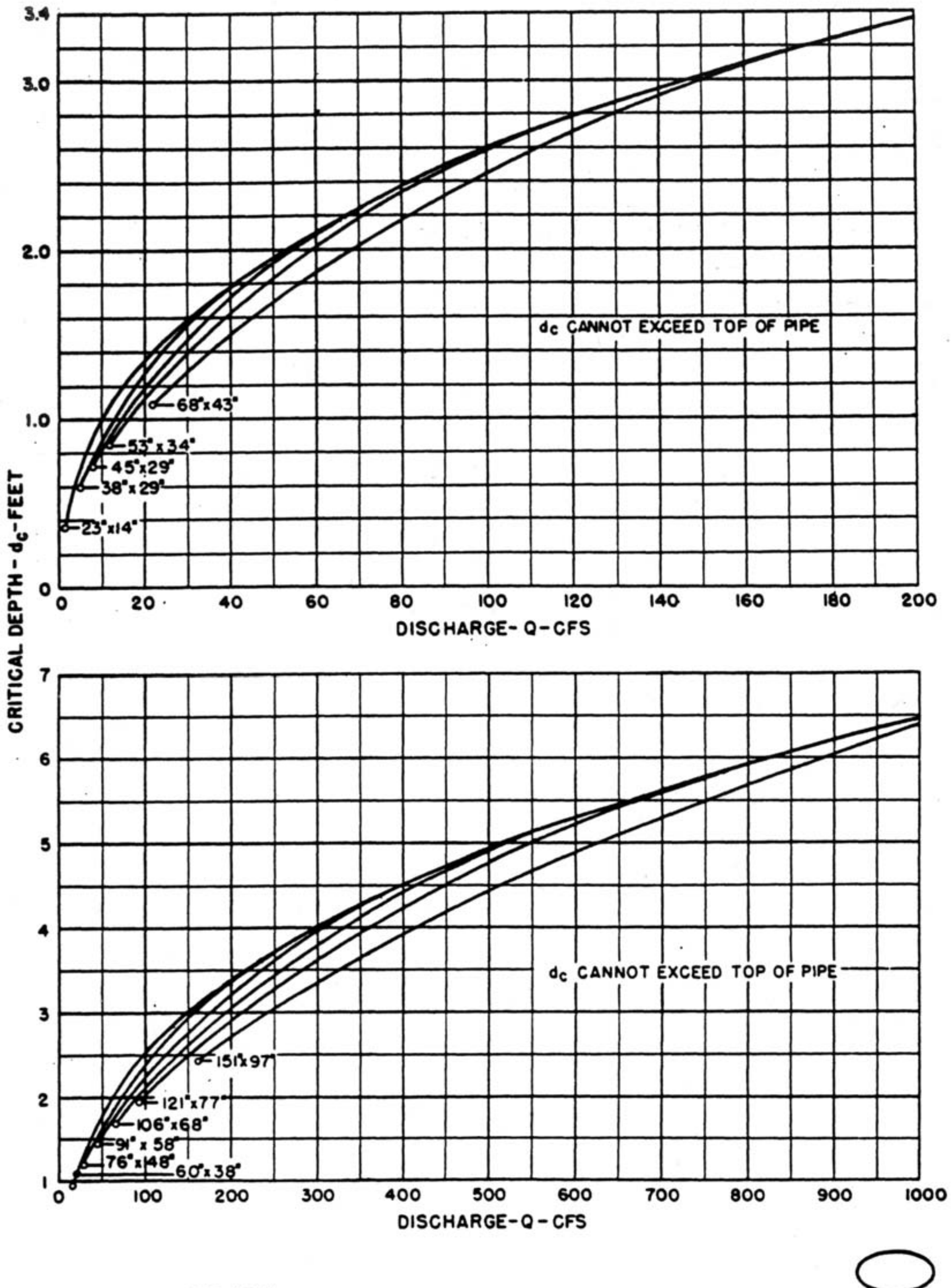
Back to Section 5.4.1, 5.4.6



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

Figure 5-21
Critical Depth Chart for Oval Concrete Pipe – Long Axis Vertical

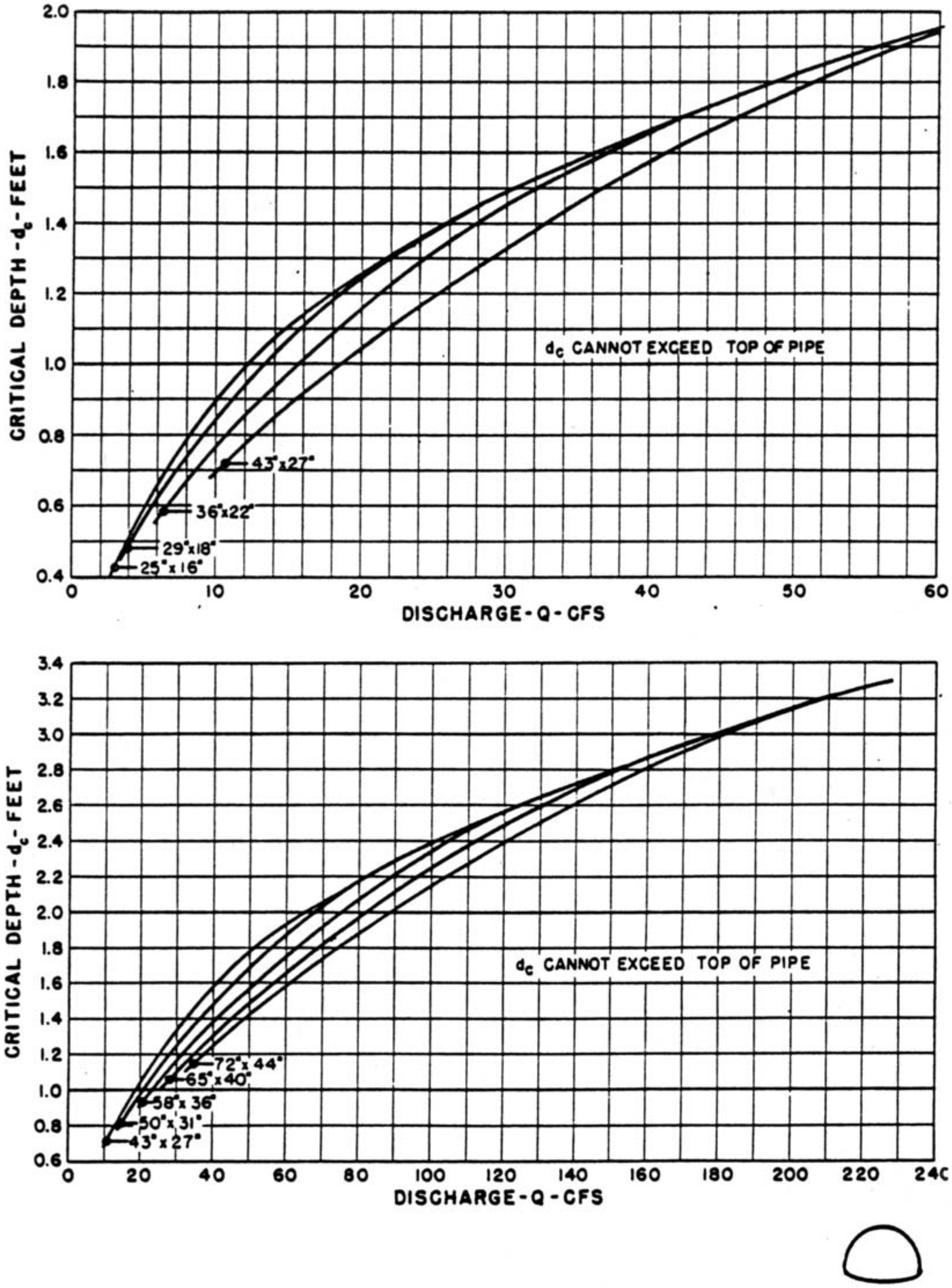
Back to Section 5.4.1



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

Figure 5-22
Critical Depth Chart for Oval Concrete Pipe – Long Axis Horizontal

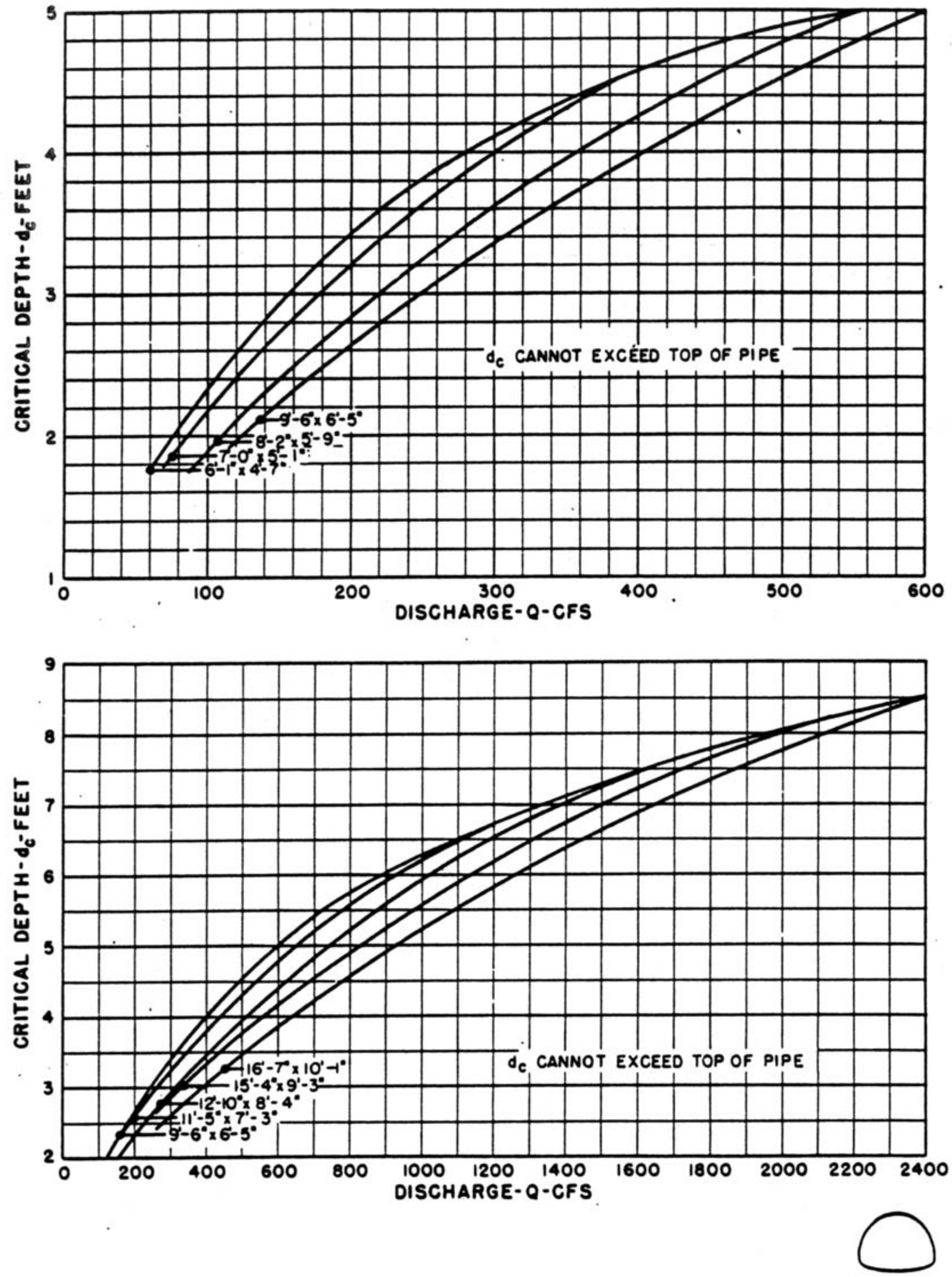
Back to Section 5.4.1



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

Figure 5-23
Critical Depth Chart for Standard CMP Arch

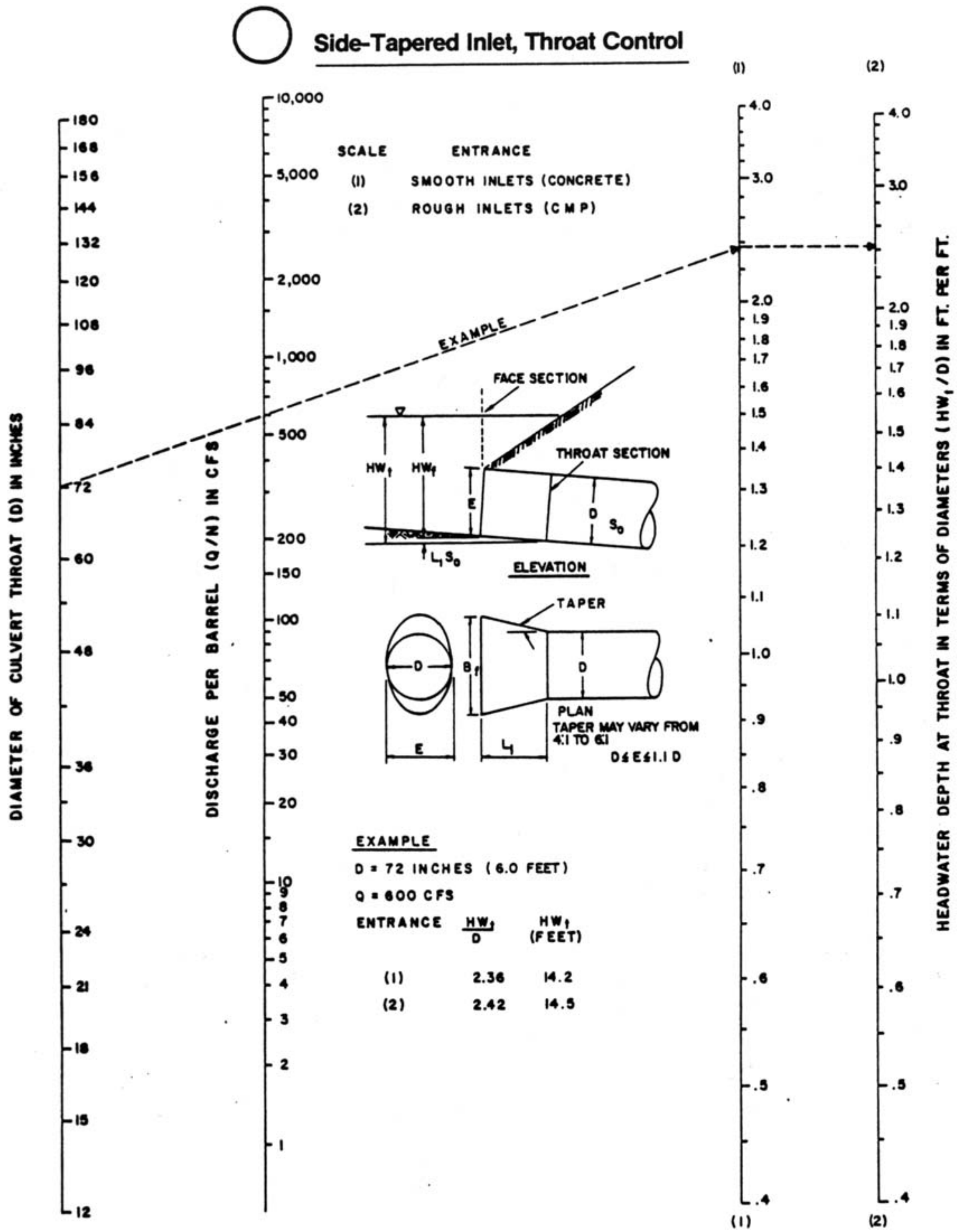
Back to Section 5.4.1



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

Figure 5-24
Critical Depth Chart for Structural Plate CMP Arch

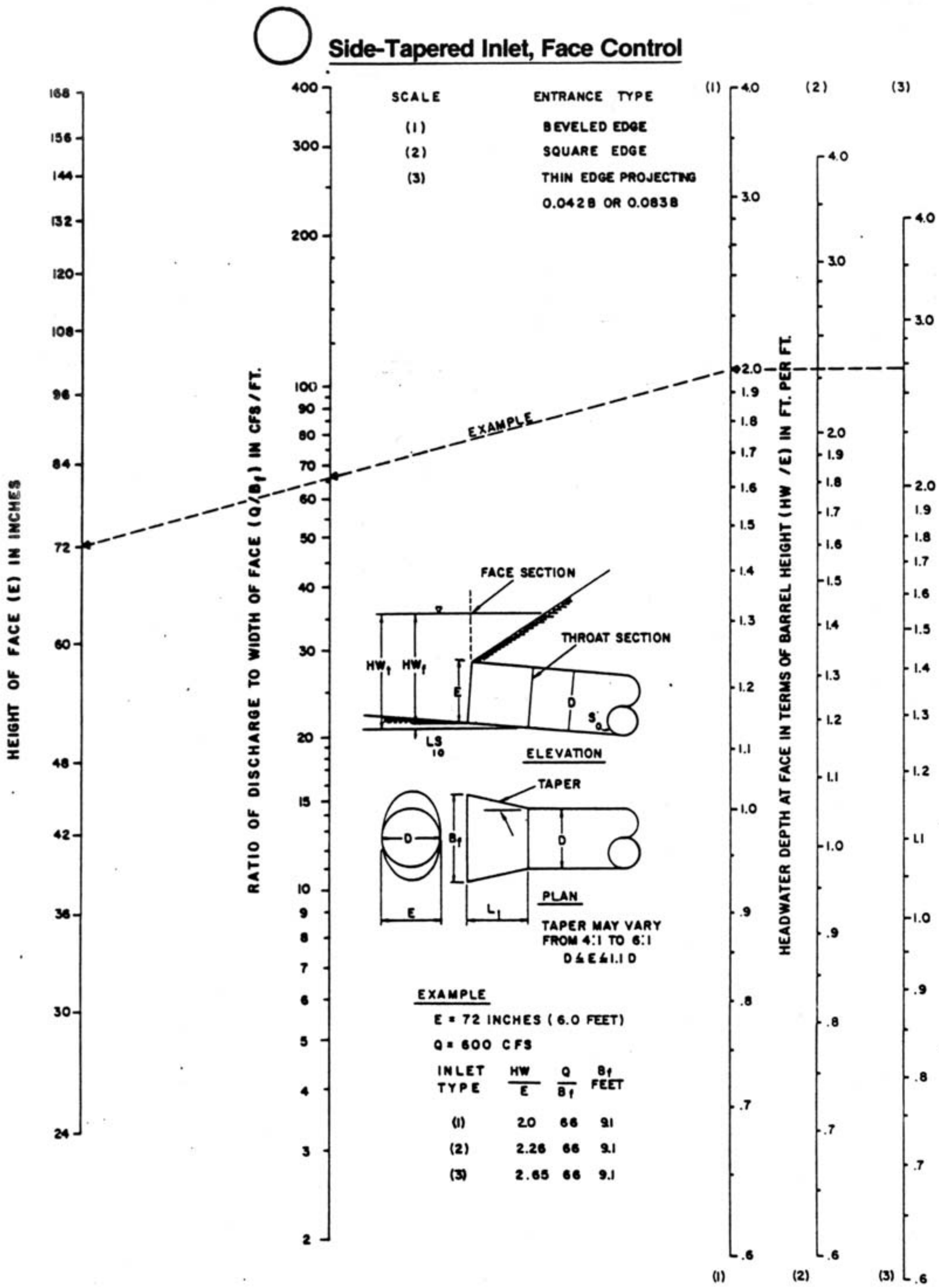
Back to Section 5.4.1



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

Figure 5-25
Throat Control Chart for Side-Tapered Inlets to Pipe Culvert
(Circular Section Only)

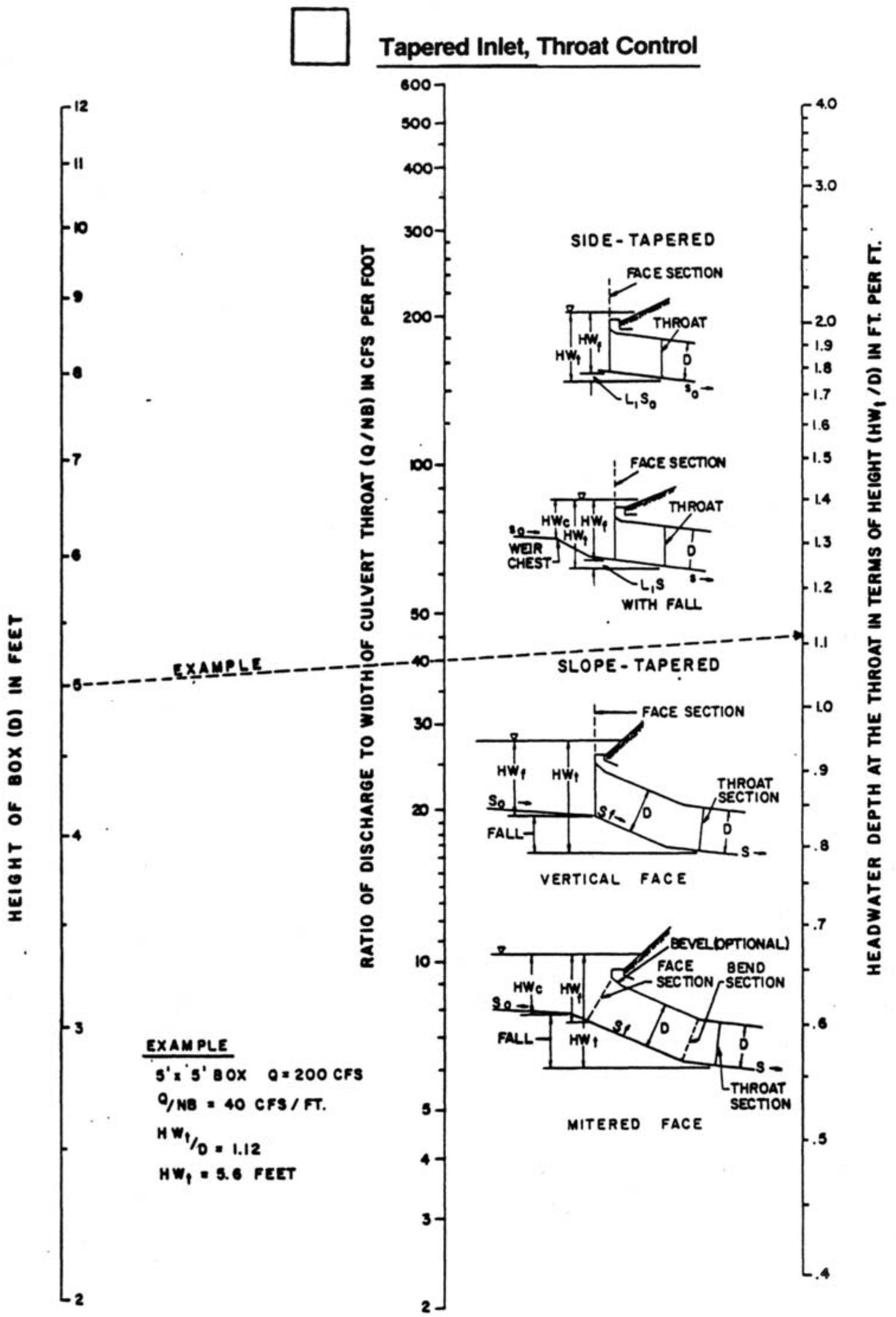
Back to Section 5.4.4



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

Figure 5-26
Face Control Chart for Side-Tapered Inlets to Pipe Culvert
(Non-Rectangular Sections Only)

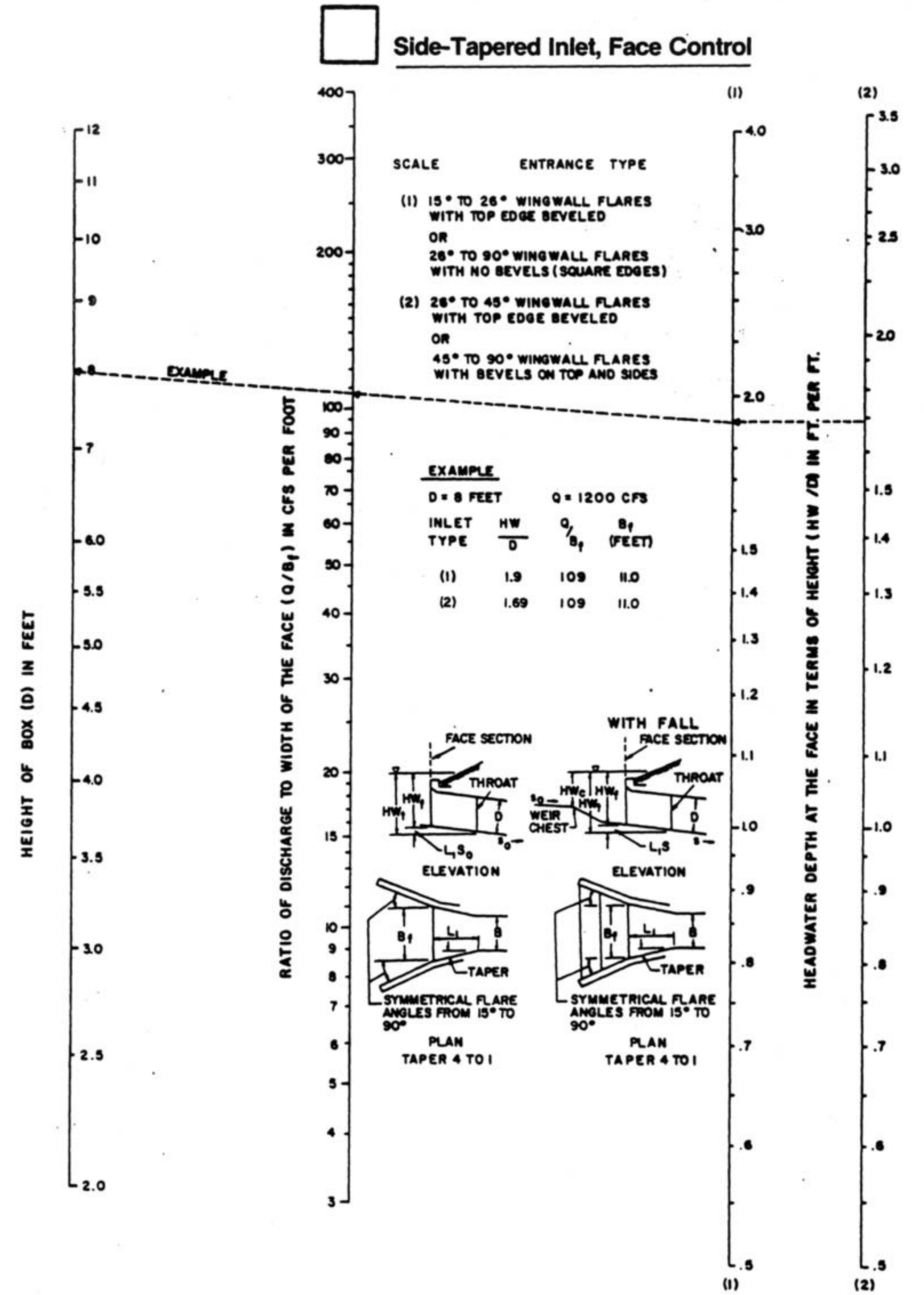
Back to Section 5.4.4



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

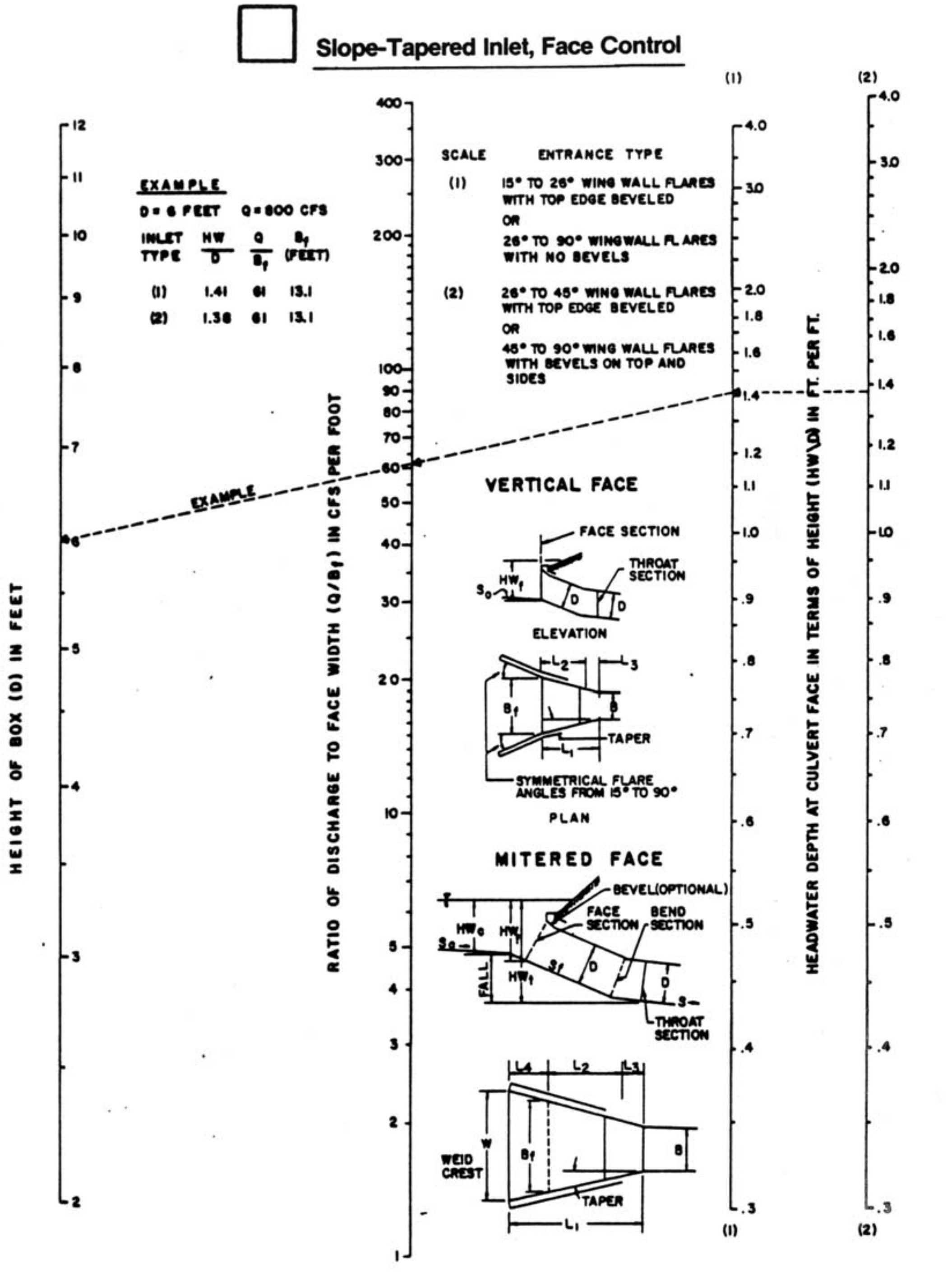
Figure 5-27
Throat Control Chart for Box Culverts with Tapered Inlets

Back to Section 5.4.4, 5.4.6



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

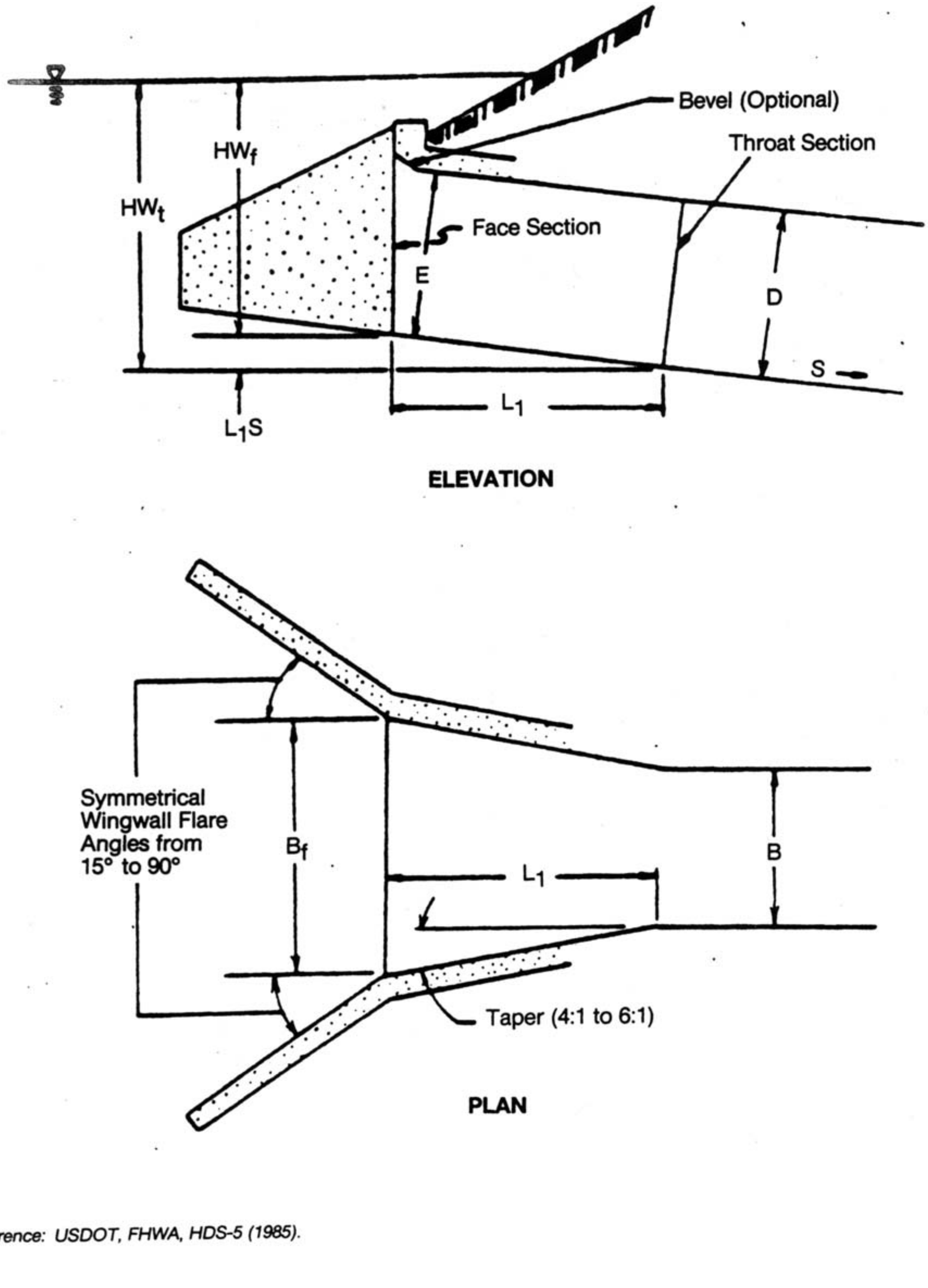
Figure 5-28
Face Control Chart for Box Culverts with Side-Tapered Inlets Back to Section 5.4.4, 5.4.6



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

Figure 5-29
Face Control Chart for Box Culverts with Slope-Tapered Inlets

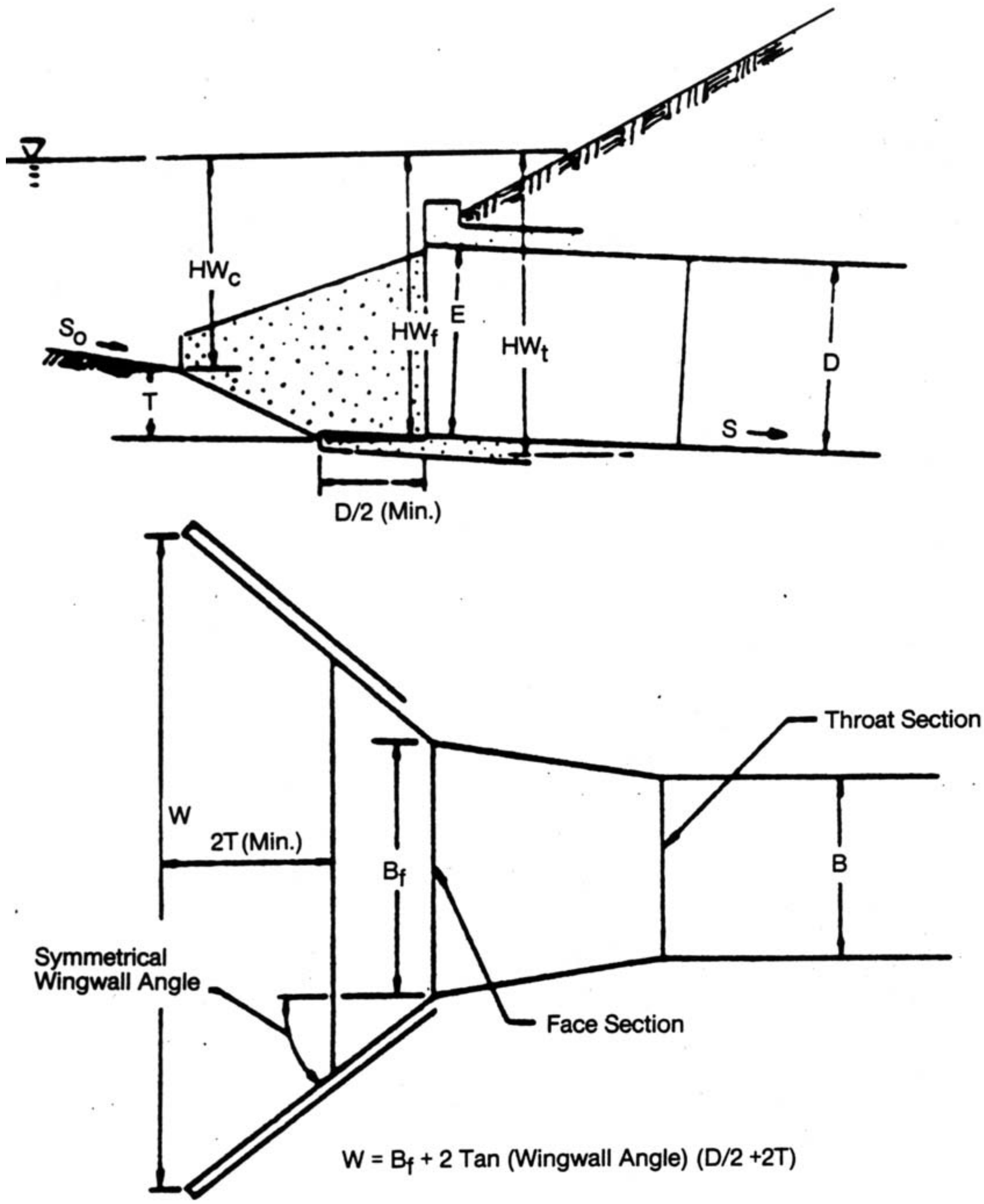
Back to Section 5.4.4



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

Figure 5-30
Typical Side-Tapered Inlet Details

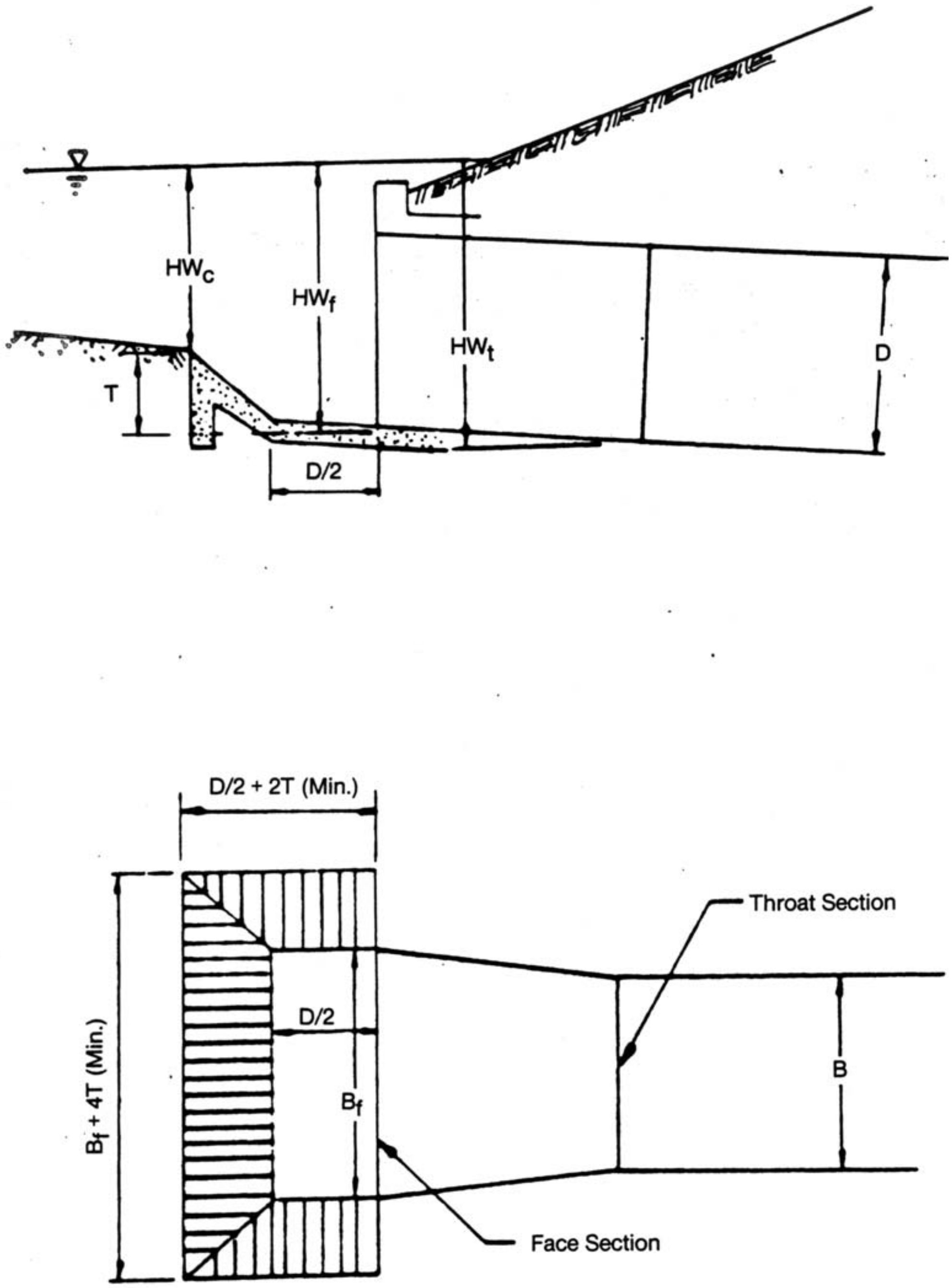
Back to Section 5.4.4



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

Figure 5-31
Side-Tapered Inlet with Upstream Depression Contained between Wingwalls

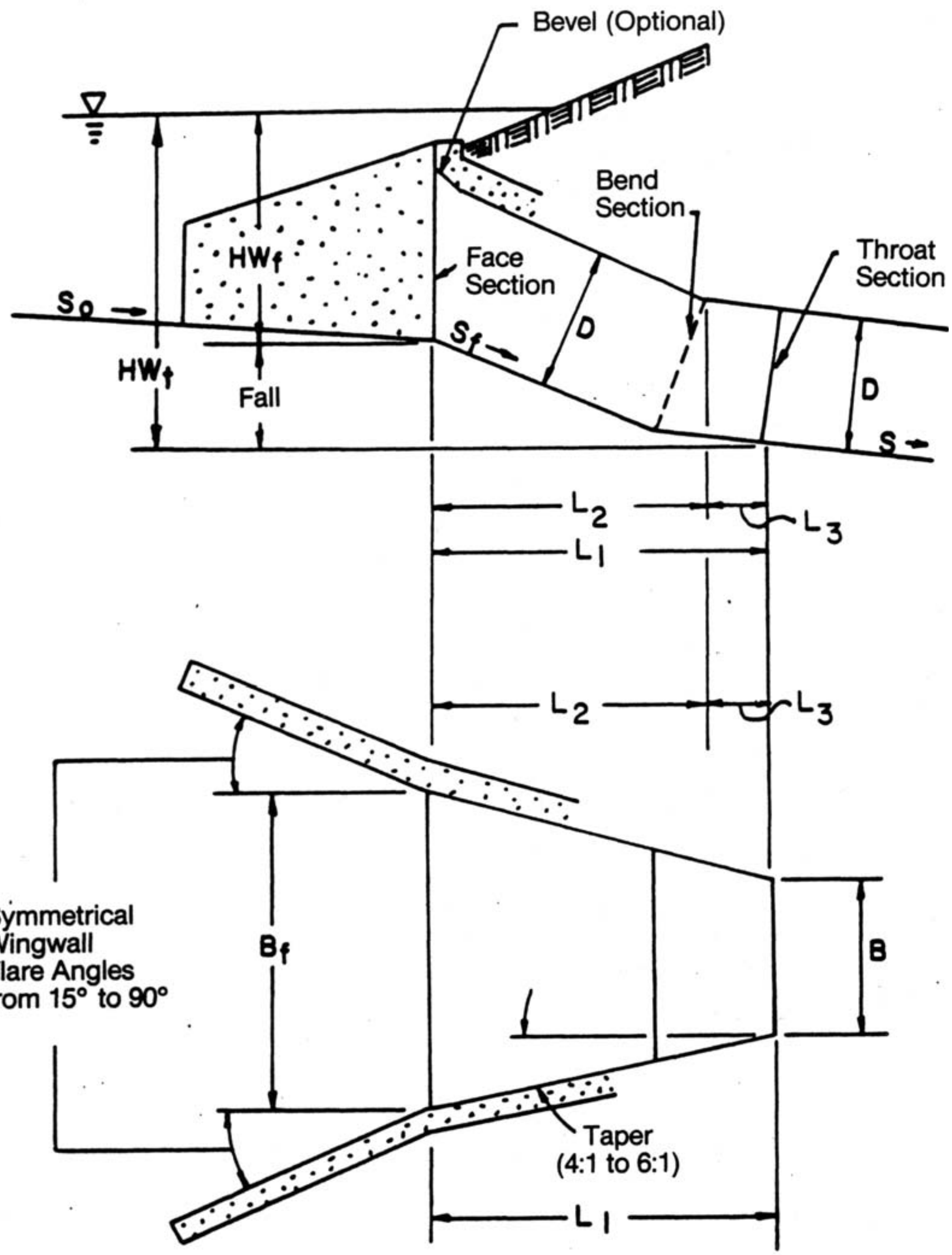
Back to Section 5.4.4



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

Figure 5-32
Side-Tapered Inlet with Upstream Sump

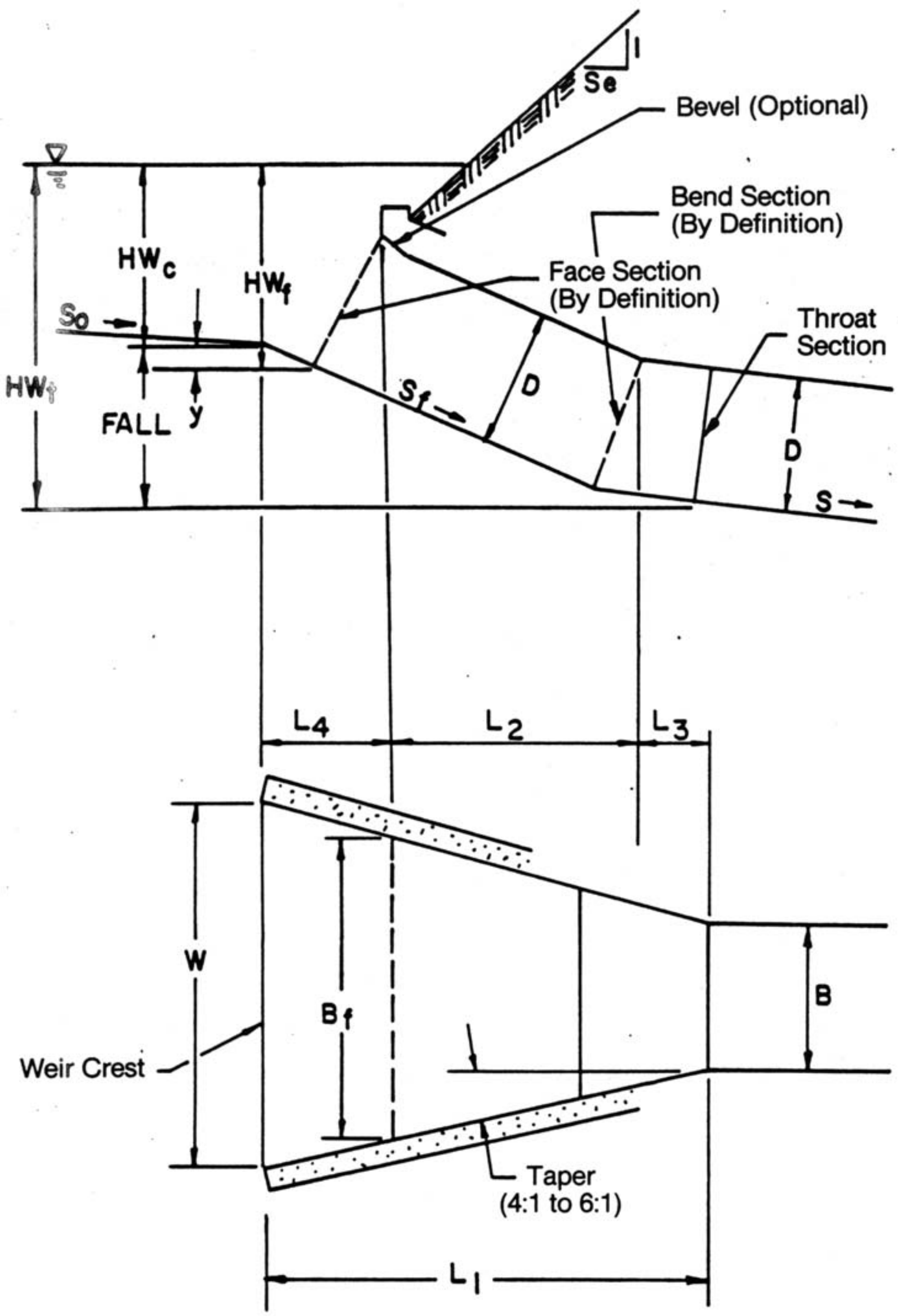
Back to Section 5.4.4



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

Figure 5-33
Slope-Tapered Inlet with Vertical Face

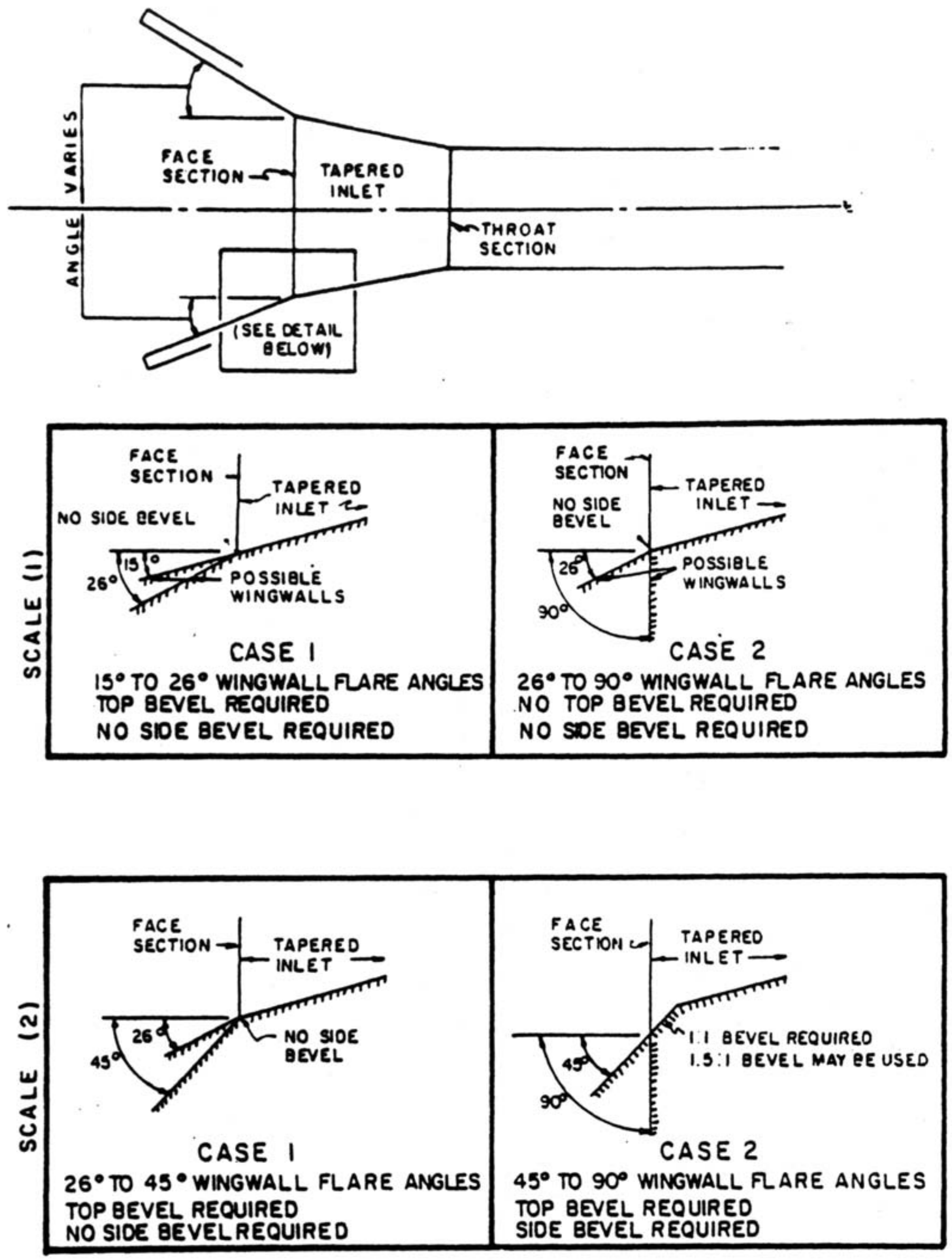
Back to Section 5.4.4



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

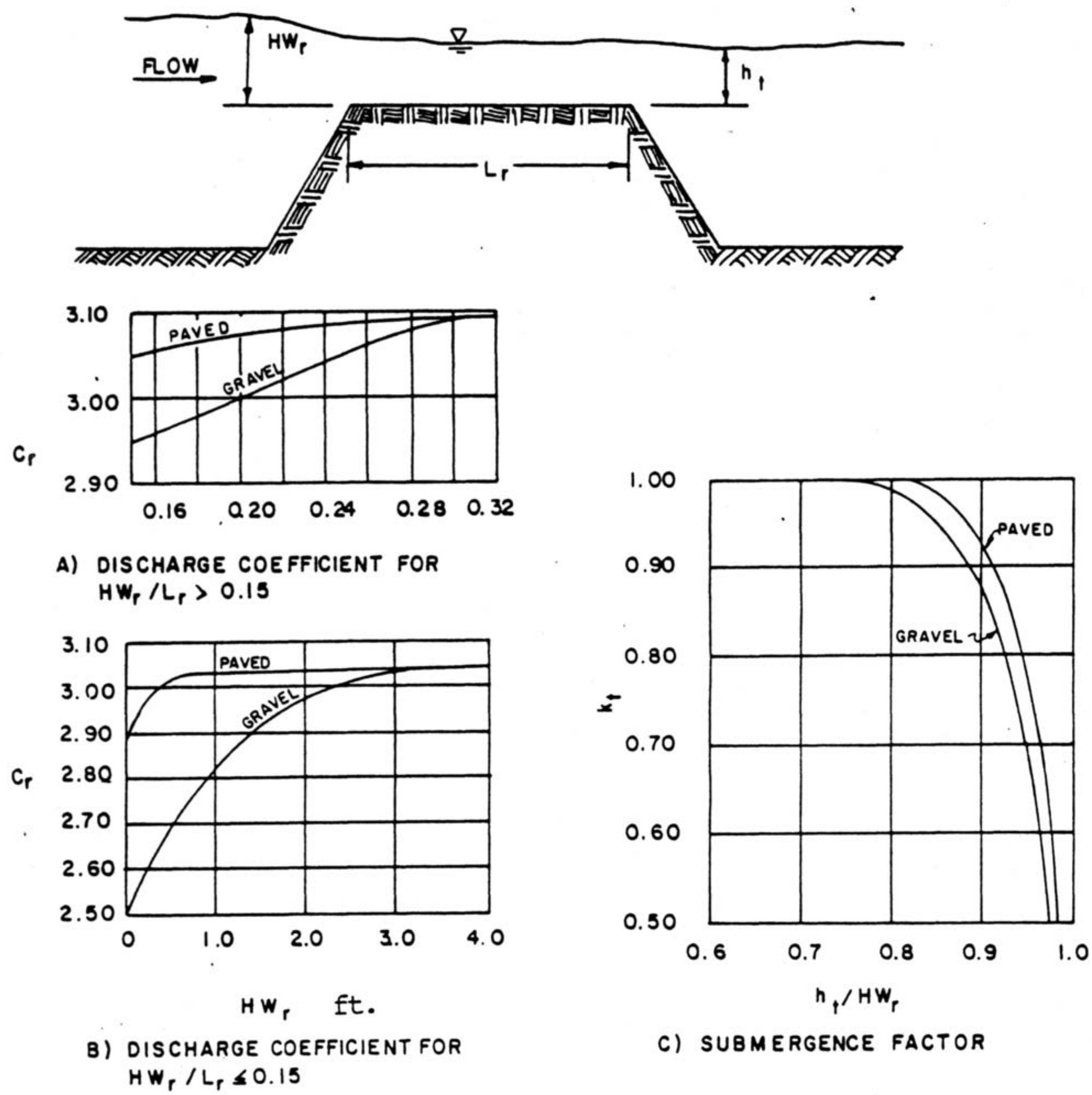
Figure 5-34
Slope-Tapered Inlet with Mitered Face

Back to Section 5.4.4



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

Figure 5-35
Inlet Edge Conditions, Face Section, Rectangular Tapered Inlets Back to Section 5.4.4



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

Figure 5-36
Discharge Coefficients for Roadway Overtopping

Back to Section 5.4.5, 5.4.6

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

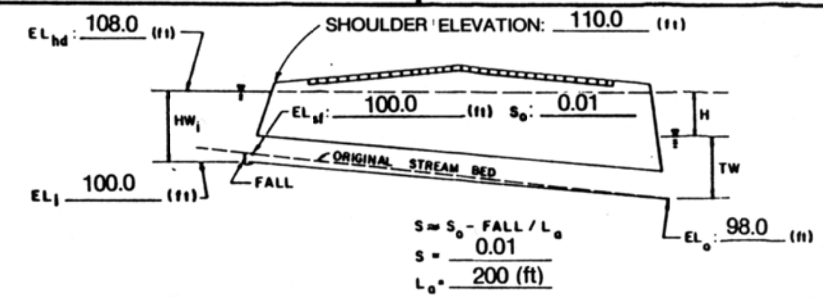
HYDROLOGICAL DATA

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

(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:

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

Back to Section 5.4.6

PROJECT: EXAMPLE 5-2
CHAPTER 5, SECTION 5.3.6

STATION: 2 + 50
SHEET 1 **OF** 1

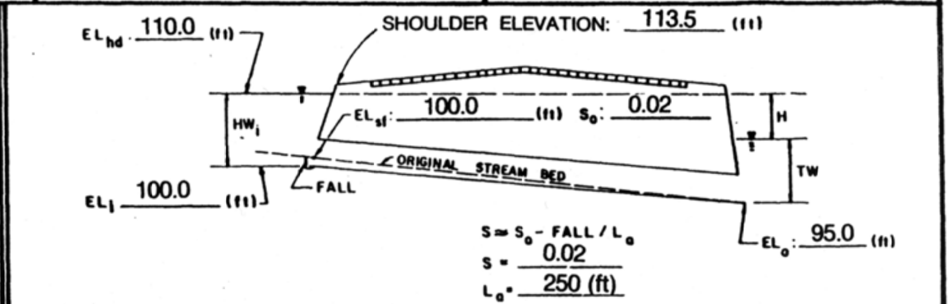
CULVERT DESIGN FORM
DESIGNER / DATE: VCA / 6/21
REVIEWER / DATE: JES / 6/21

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

(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:

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

Back to Section 5.4.6

PROJECT: EXAMPLE 5-3
CHAPTER 5, SECTION 5.3.6

STATION: 4 + 00
SHEET 2 **OF** 3

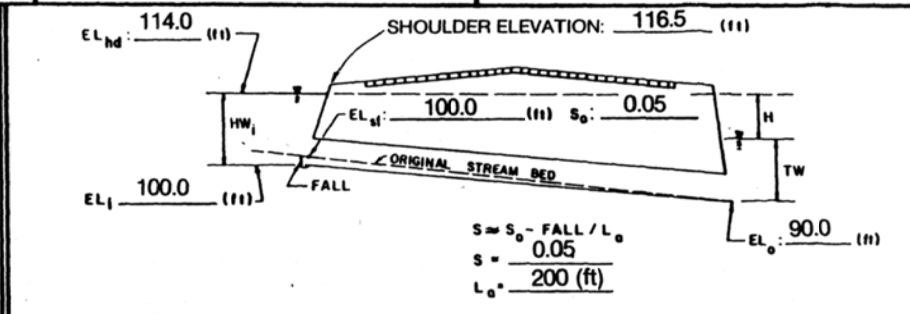
CULVERT DESIGN FORM
DESIGNER / DATE: VCA / 6/21
REVIEWER / DATE: JES / 6/21

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

(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:

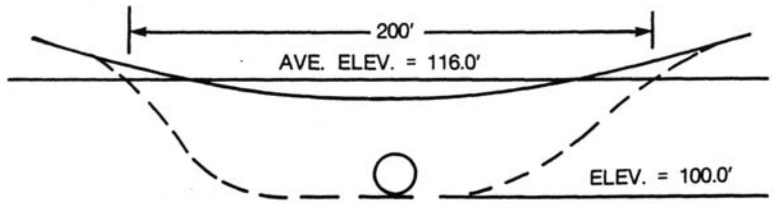
NEW Q_{100} RESULTS IN ROADWAY OVERTOPPING OF ABOUT 2.0' (SEE FIGURE 5-40).

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

Back to Section 5.4.6



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

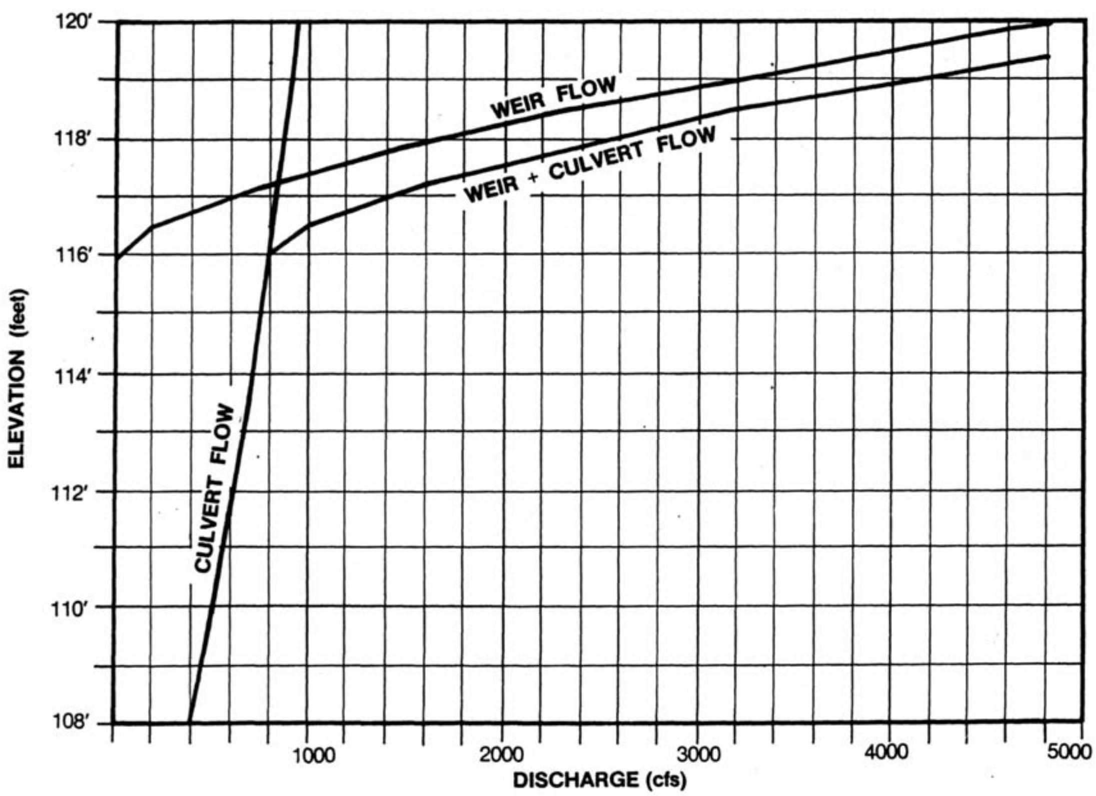


Figure 5-40
Roadway Overtopping Data for Example 5-3

Back to Section 5.4.6

PROJECT: EXAMPLE 5-4
CHAPTER 5, SECTION 5.3.6

STATION: 4 + 00
SHEET 1 **OF** 3

CULVERT DESIGN FORM
DESIGNER / DATE: VCA / 6/22
REVIEWER / DATE: JES / 6/22

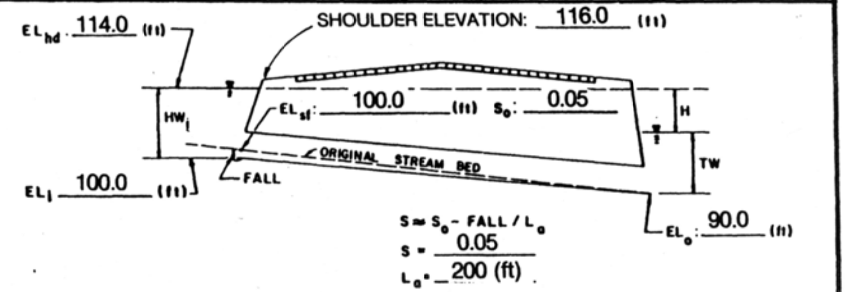
HYDROLOGICAL DATA

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)	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'-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

Back to Section 5.4.6

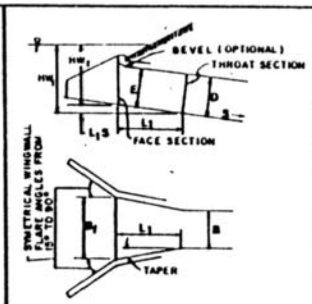
PROJECT: EXAMPLE 5-4
CHAPTER 5, SECTION 5.3.6

STATION: 4 + 00.
SHEET 2 OF 3

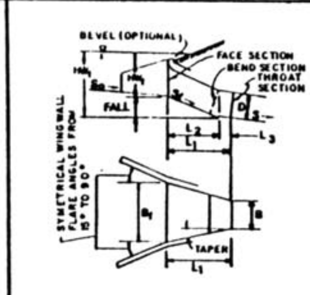
TAPERED INLET DESIGN FORM

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 ≥ E ≥ D : E = HEIGHT OF INLET
(4) FROM DESIGN CHARTS : FIG 5-25—5-29
(5) MIN. $B_f = Q/(Q/B_f)$
(6) MIN. $L_3 = 0.5\,NB$
(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 $= (L_2 + L_3) \Big/ \left[\frac{B_f - NB}{2}\right]$
(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. $W = 0.35\,Q/HW_c^{1.5}$

*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

Back to Section 5.4.6

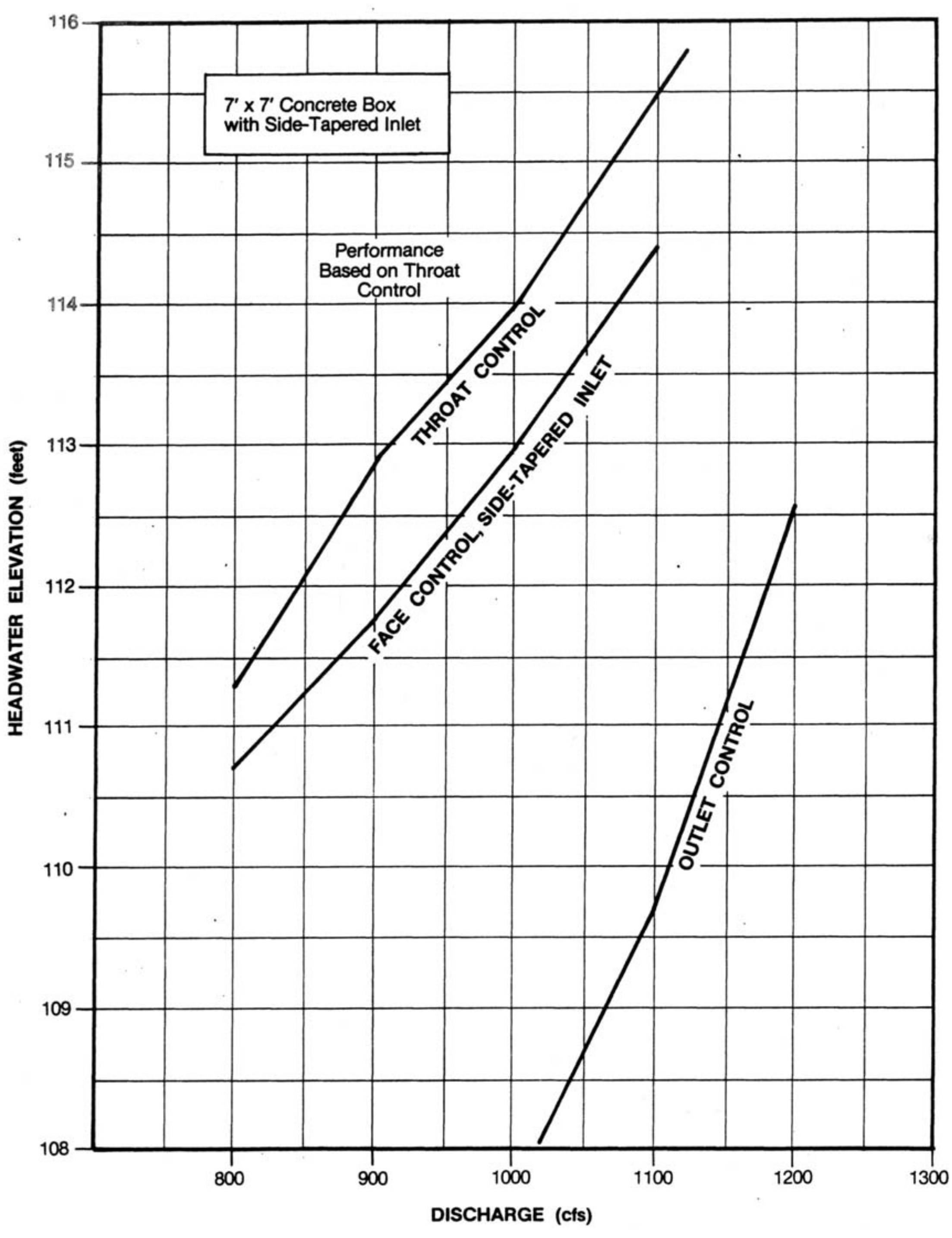


Figure 5-43
Results for Example 5-4

Back to Section 5.4.6

Table 5-1
Culvert Entrance Loss Coefficients

Type of Structure and Design of Entrance	Entrance Coefficient, ke
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 slopea	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 slopea	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

Note:
[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)

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City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 6: Storm Sewer Hydraulics

Volume 3: Best Management Practices Manual

Revision: 0
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Acronym List
(Chapter 6)

A	Cross-sectional area, (ft^2)
D	Pipe diameter, (ft)
g	Acceleration due to gravity, (32.2 ft/sec^2)
H_f	Energy loss due to friction, (ft)
H_L	Head loss due to pipe form conditions, (ft)
IDF	Intensity-Duration-Frequency
K	Loss coefficient for pipe form conditions
K_p	Circular pipe open channel conveyance factor
k_v	Pipe velocity factor
L	Conduit length, (ft)
n	Manning's roughness coefficient for sheet flow
PESC	Permanent Erosion Prevention and Sediment Control
Q	Discharge flow rate
R	Hydraulic radius of conduit, (ft)
S	Slope of the energy gradient, (ft/ft)
TCP	Temporary Construction Site Runoff Management Practices
v	Average velocity, (ft/sec)
v_1	Velocity in smaller pipe

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6.0 STORM SEWER HYDRAULICS

6.1 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 the City and/or County.

6.2 Design Criteria

6.2.1 Return Periods

Storm sewers are defined as closed conduits that receive storm water runoff collected at various types of intake structures, such as surface inlets, curb inlets, headwalls, etc., and transport it to a central discharge point such as an open channel, detention pond, lake, or so forth. These are differentiated from culverts, which typically concentrate open-channel flows to transport them under a specific local improvement, such as a street, before releasing the flow back into an open channel. Storm sewers shall be designed for the total intercepted flow based on the design event (see Volume 1, Section 6.3.1). The design event return period is 10 years. Where a portion of the storm sewer also serves as a culvert (cross drain) for a major intersection, major conveyance system or state route — that portion must be designed to accommodate the design (50-year) flow from that drainage area. For instance, a pipe carrying storm water from an inlet in the upstream curb line of a street to a junction box at a cross drain would be designed for the 10-year flow, but the cross drain itself would be designed for the 50-year flow from the off-street area plus any additional area added from the street drains.

6.2.2 Materials and Manning's n Values

Reinforced concrete pipe or box culverts are the only acceptable materials for storm sewer construction. Circular pipe is the preferred choice, but in cases where low clearance values dictate its use, horizontal elliptical or arch pipe may be approved by the City and/or County. Values for Manning's roughness coefficient are given below:

Concrete pipes and box culverts (precast or cast-in-place)	n = 0.013

Materials for use in storm sewer systems shall meet the requirements of the following specifications as applicable:

ASTM C-76	Circular Pipe
ASTM C-506	Arch Pipe
ASTM C-507	Elliptical Pipe
ASTM C-789	Concrete Box Sections

Where standard laying conditions assumed in the specifications listed above (bedding conditions, cover requirements, etc.) cannot be met, the use of materials meeting more rigorous requirements or other remedial actions may be required at the discretion of the City and/or County.

6.2.3 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 6.5.2), 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).

6.2.4 Pipe Size and Length

A minimum pipe size of 15 inches is required for storm sewers. Access spacing shall not exceed 400 feet for conduits less than 42 inches in diameter and shall not exceed 500 feet for pipes 42 inches and larger, without approval from the City and/or County.

6.2.5 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 1 foot. This requirement may be waived if the utility line has been accurately located at the point of conflict and approval is obtained from Memphis Light Gas and Water Division ahead of time. If the minimum clearance requirement is waived, a watertight encasement will be required and must extend at least 10 feet on each side of the crossing. Crossings with a skew angle of less than 45 degrees will not be allowed. 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.

6.2.6 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.

6.2.7 Easements

Easement requirements are given in Volume 1, Section 6.3.3.

6.3 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 6.2.

2. Hydrologic Calculations: Determination of design flow rates and volumes (see Section 6.4).

3. Hydraulic Calculations: Determination of pipe sizes required to carry design flow rates and volumes, as discussed in Section 6.5.

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.

6.4 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, the Rational Method will not be accepted on drainage areas greater than 10 acres. 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.

6.4.1 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 flow at an individual structure must be accounted for by adding it in directly to the next structure downstream. 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 Intensity-Duration-Frequency (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 2-4 (also see Section 2.4.1) 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.

6.4.2 Runoff Hydrograph Method

The runoff hydrograph method is used for areas larger or more complex than can be effectively represented using the Rational Method. The calculation of runoff hydrographs is discussed in detail in Chapter 2 of this manual.

6.5 Hydraulic Calculations

Hydraulic calculations are used to size conduits to handle the design flows determined from hydrologic calculations (see Section 6.4). 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.

6.5.1 Pressure Versus Gravity Flow

As a rule, storm sewer systems will be designed as gravity systems for the design (Q_{10}) flow (see Volume 1, Section 6.3.2). However, where a project drainage system incorporates elements of the existing drainage infrastructure, the City and/or County may on rare occasions allow the occurrence of pressure flow for design flow values to minimize the disturbance of existing pavement, traffic flow, etc., that would be required to replace those elements.

Should portions of storm sewers be allowed 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.

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 6-2. 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.

6.5.2 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

Friction Loss

The energy loss required to overcome friction caused by conduit roughness is generally calculated as:

$$H_f = \left(\frac{29 n^2 L}{R^{1.33}} \right) \frac{v^2}{2g} \tag{6-1}$$

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/second^2

Entrance, Exit, Expansion, Contraction, and Bend Losses

These head losses due to pipe form conditions are generally calculated as:

$$H_L = K \frac{v^2}{2g} \tag{6-2}$$

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/second^2

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 6-3 and 6-4.

The bend loss coefficient for storm sewer systems can be evaluated using Figure 6-3, 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 6-5.

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 6-4) 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 center lines 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 6-4 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 6-5. Several of these inefficient devices would appear to be improvements, indicating that special shapings deviating from those in Figure 6-4 should be used with caution.

6.5.3 Gravity Flow

The capacity of storm sewers flowing full under gravity flow conditions may be calculated using the following form of Manning's Equation:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (6\text{-}3)$$

Restated, the Manning's Equation can be written as follows:

$$v = \frac{0.592}{n} D^{2/3} S^{1/2} \qquad (6\text{-}4)$$

$$Q = vA \qquad (6\text{-}5)$$

$$Q = \frac{0.465}{n} D^{8/3} S^{1/2} \qquad (6\text{-}6)$$

Where:

Q = 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

Use of the Manning Equation in this form allows the calculation of full-flow capacity using only trial pipe size and slope without having to make separate calculations of area and wetted perimeter. Storm sewer capacity calculations based on Manning's Equation can also be accomplished using Figures 6-6, 6-7, and 6-8 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). There are also a myriad of stand-alone computer software packages and design modules within CAD packages that accomplish the calculation of pipe capacities given known or trial parameters for the pipe under design. These electronic aides may be used for the design of City and/or County storm sewer pipes, assuming they produce comparable results to methods currently in use by the City and/or County for the same set of input parameters. An example of the hand calculation of a pipe capacity using the included nomograph is given below.

Nomograph

The following steps are used for solving Manning's Equation using the circular pipe nomograph in Figure 6-6:

1. Determine input data, including slope in feet per foot, Manning's n value, and pipe diameter in inches or feet.

2. Connect a line from the slope scale, Point 1 at 0.003, to the Manning's n scale, Point 2 at 0.013 for concrete pipe, and note the point of intersection on the turning line, Point 3.

3. Connect a line from the discharge at 100 cfs, Point 4, to the point of intersection obtained in Step 2, Point 3, noting its intersection with the pipe diameter scale, Point 5 at 51 inches and extending the line back to the velocity scale intersecting it at point 6 indicating a velocity of 7.1 fps.

4. Since concrete pipe is not available in a 51-inch diameter, the next larger size of 54 inches would be selected. Going back to Step 3 above and connecting the turning point, Point 3, with the selected available size of 54 inches gives a revised capacity of 122 cfs.

Partial Flow Charts

For partial flow in a circular pipe. Figures 6-7 and 6-8 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:

$$K_p = \frac{Qn}{D^{8/3} S^{1/2}} \tag{6-7}$$

Where:

K_p	=	Circular pipe open channel conveyance factor
Q	=	Discharge rate for design conditions, in cfs
n	=	Manning's roughness coefficient (see Section 6.2.2)
D	=	Pipe diameter, in ft
S	=	Slope of the energy grade line, in feet/foot

3. Enter the x-axis of Figure 6-7 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 6-8 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 6-8 (Step 7)
D	=	Pipe diameter, in feet
S	=	Slope of the energy grade line, in feet/foot
n	=	Manning's roughness coefficient (see Section 6.2.2)

6.6 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.

6.7 Chapter Equations

$$H_f = \left(\frac{29n^2L}{R^{1.33}}\right)\frac{v^2}{2g} \tag{6-1}$$

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/second^2

$$H_L = K\frac{v^2}{2g} \tag{6-2}$$

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/second^2

$$v = \frac{1.49}{n}R^{2/3}S^{1/2} \tag{6-3}$$

$$v = \frac{0.592}{n}D^{2/3}S^{1/2} \tag{6-4}$$

$$Q = vA \tag{6-5}$$

$$Q = \frac{0.465}{n}D^{8/3}S^{1/2} \tag{6-6}$$

Where:

Q	=	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

$$K_p = \frac{Qn}{D^{8/3}S^{1/2}} \qquad (6\text{-}7)$$

Where:

K_p	=	Circular pipe open channel conveyance factor
Q	=	Discharge rate for design conditions, in cfs
n	=	Manning's roughness coefficient (see Section 6.2.2)
D	=	Pipe diameter, in ft
S	=	Slope of the energy grade line, in feet/foot

$$v = \frac{K_v D^{2/3} S^{1/2}}{n} \qquad (6\text{-}8)$$

Where:

v	=	Average velocity, in feet/second
k_v	=	Pipe velocity factor from Figure 6-8 (Step 7)
D	=	Pipe diameter, in feet
S	=	Slope of the energy grade line, in feet/foot
n	=	Manning's roughness coefficient (see Section 6.2.2)

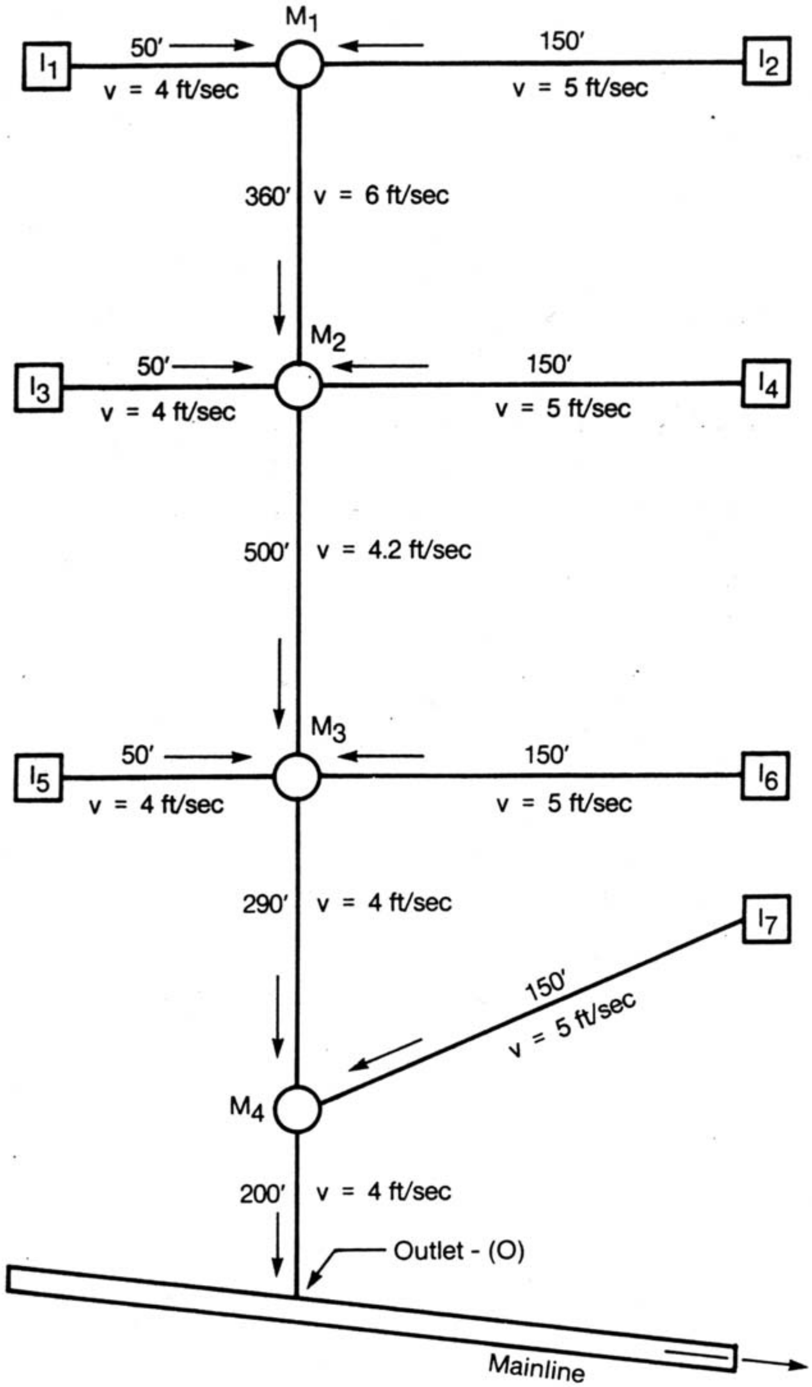

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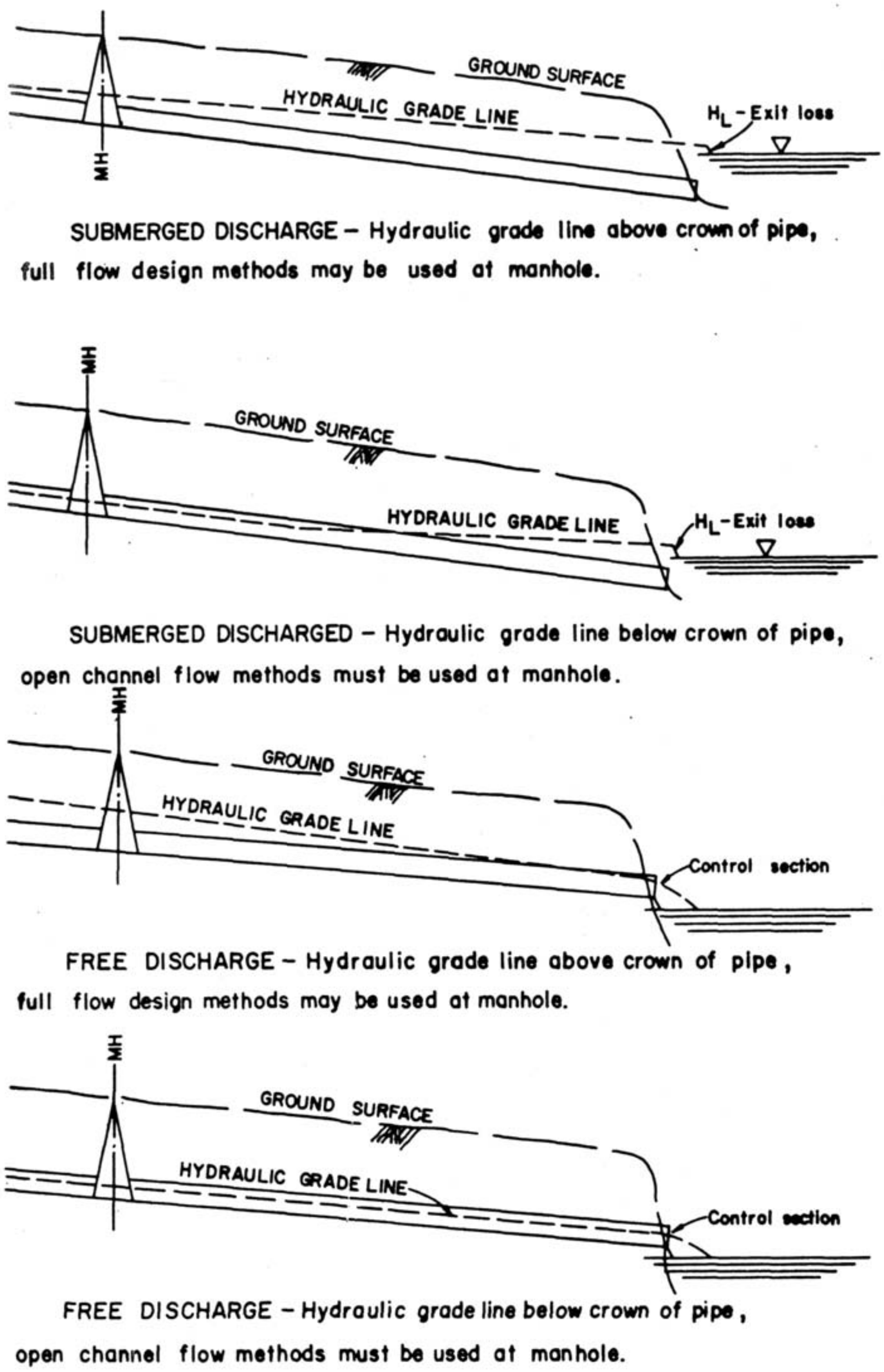
Figure 6-1
Hypothetical Storm Sewer System Layout
For Demonstrating Hydrologic Calculations



Reference: Wright-McLaughlin Engineers (1969).

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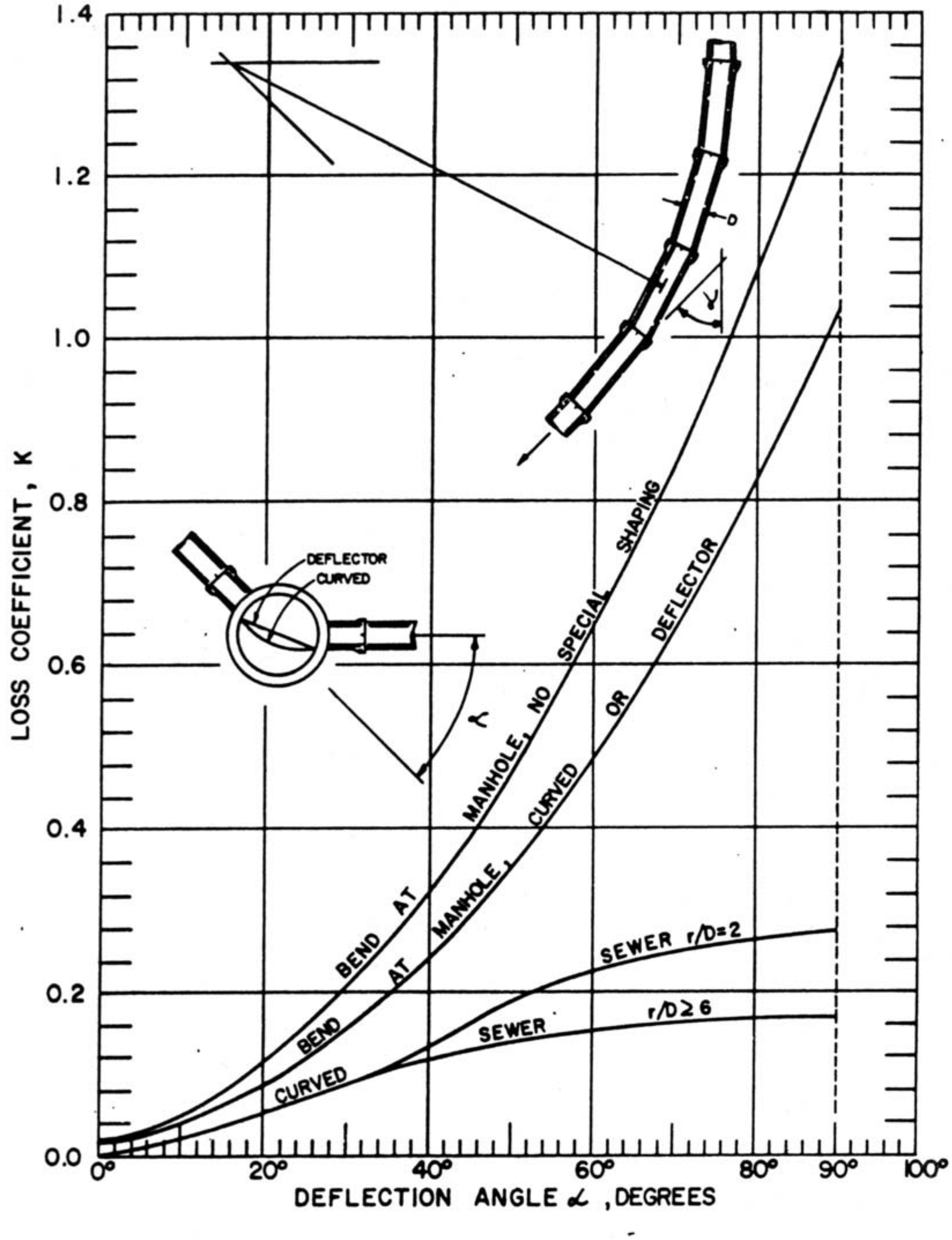
Figure 6-2
Determination of Pressure vs. Open-Channel Flow Conditions in Storm Sewer Systems



Reference: Wright-McLaughlin Engineers (1969).

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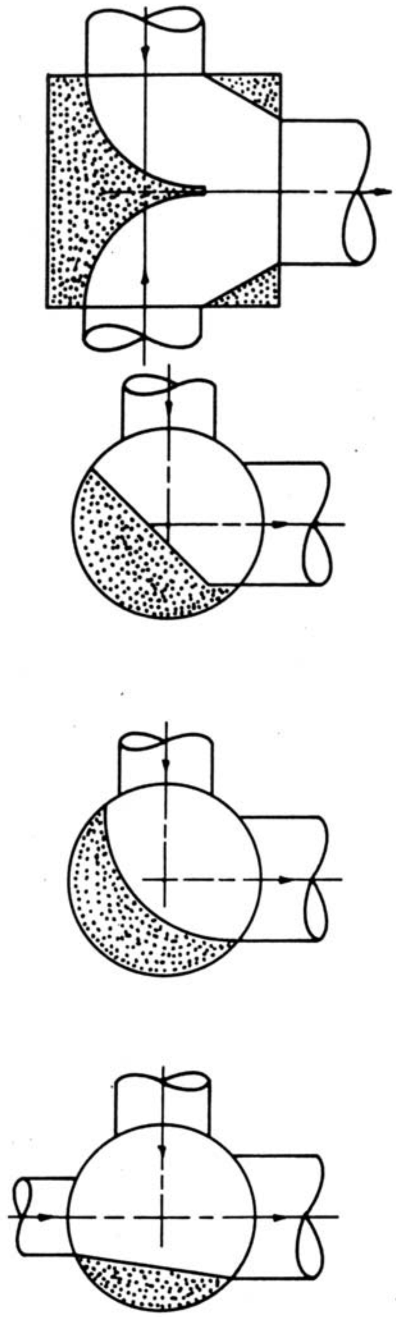
Figure 6-3
Storm Sewer Bend Loss Coefficient



Directly opposed lateral with deflector
(head losses are still excessive with this method, but are significantly less than when no deflector exists.)

Bend with straight deflector

Bend with curved deflector

Inline upstream main & 90° lateral with deflector

These methods of shaping the interior of a manhole were found efficient in University of Missouri (1958) tests.

Reference: Wright-McLaughlin Engineers (1969).

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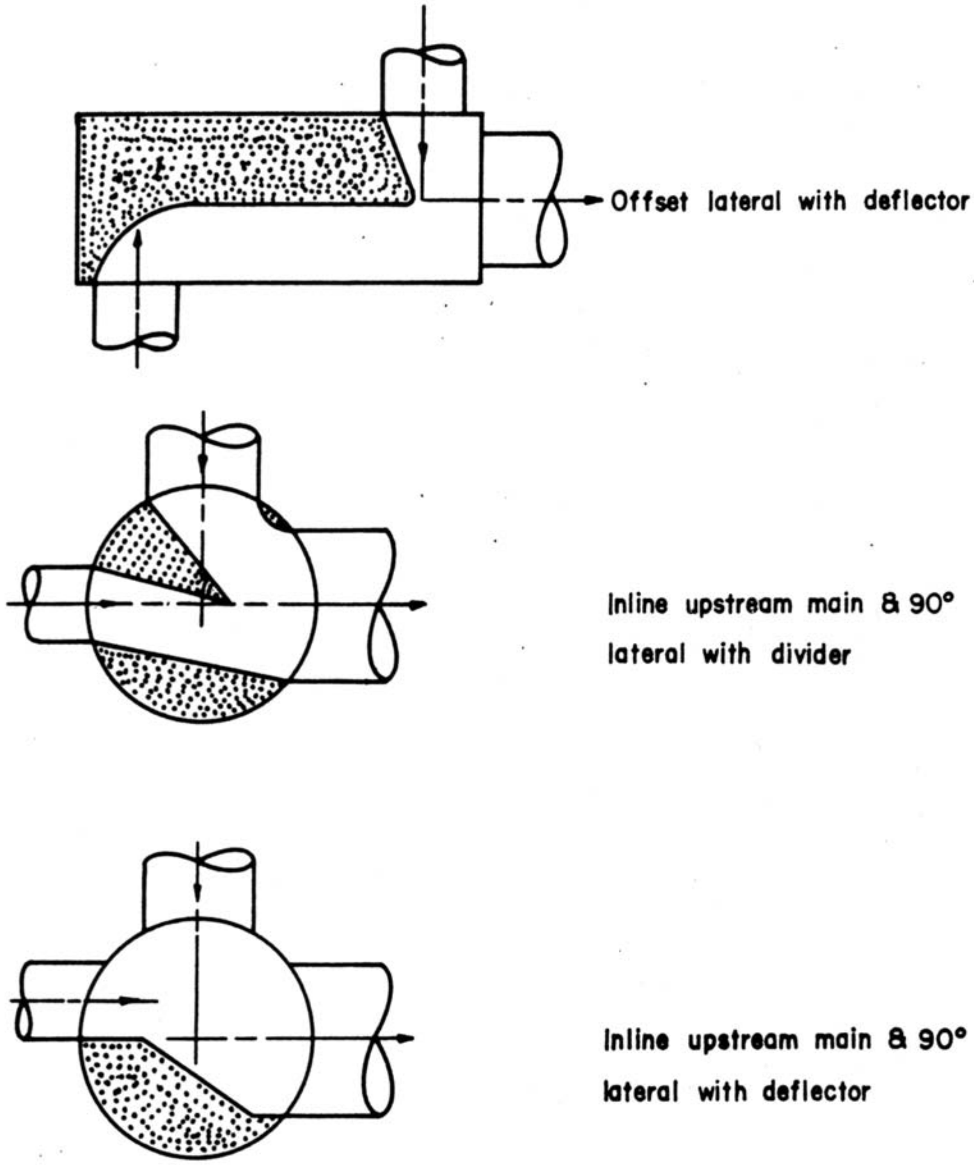
Figure 6-4
Efficient Manhole Shaping



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).

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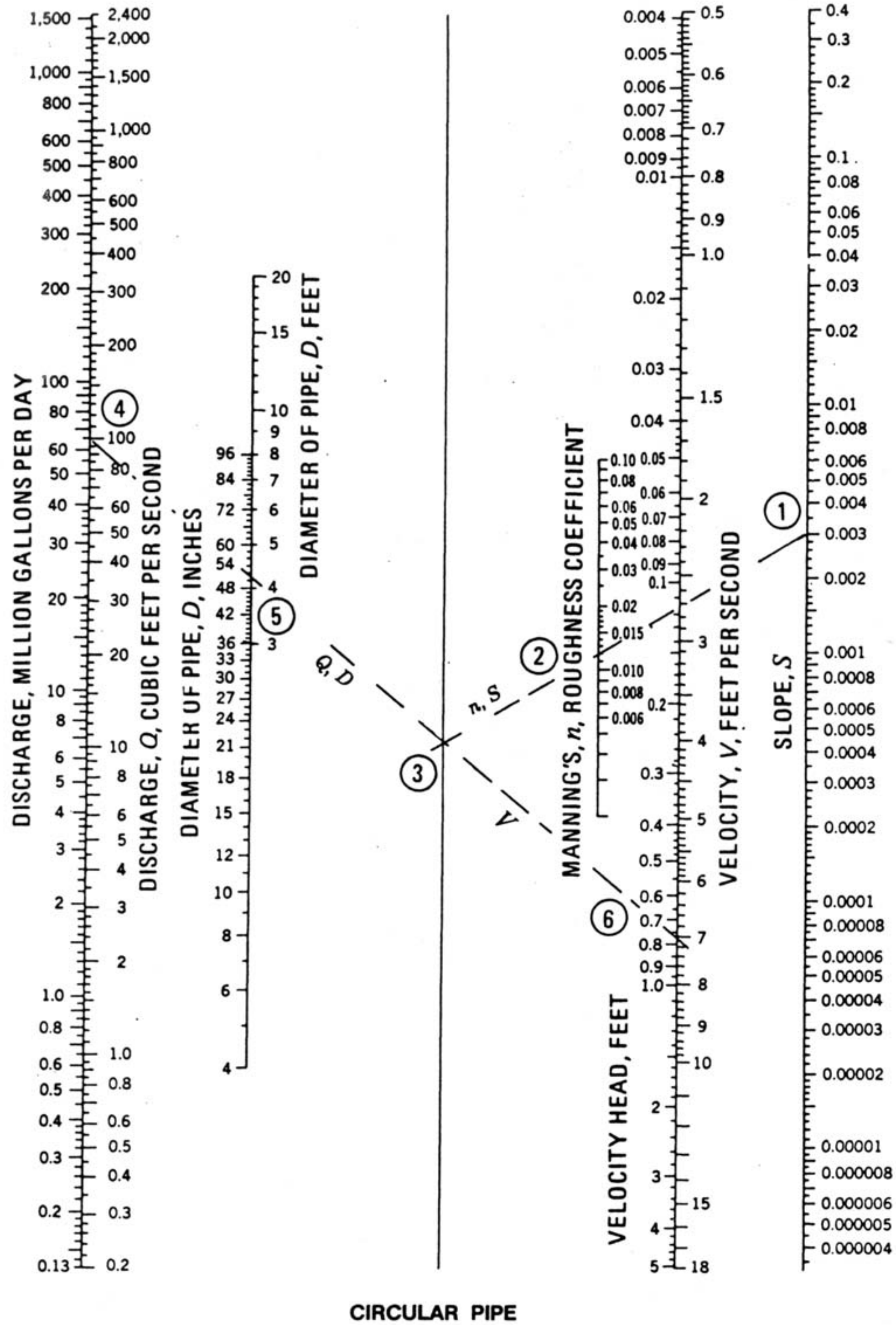
Figure 6-5
Inefficient Manhole Shaping



Reference: American Concrete Pipe Association (1980).

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Figure 6-6
Circular Pipe Nomograph for Solving Manning's Equation

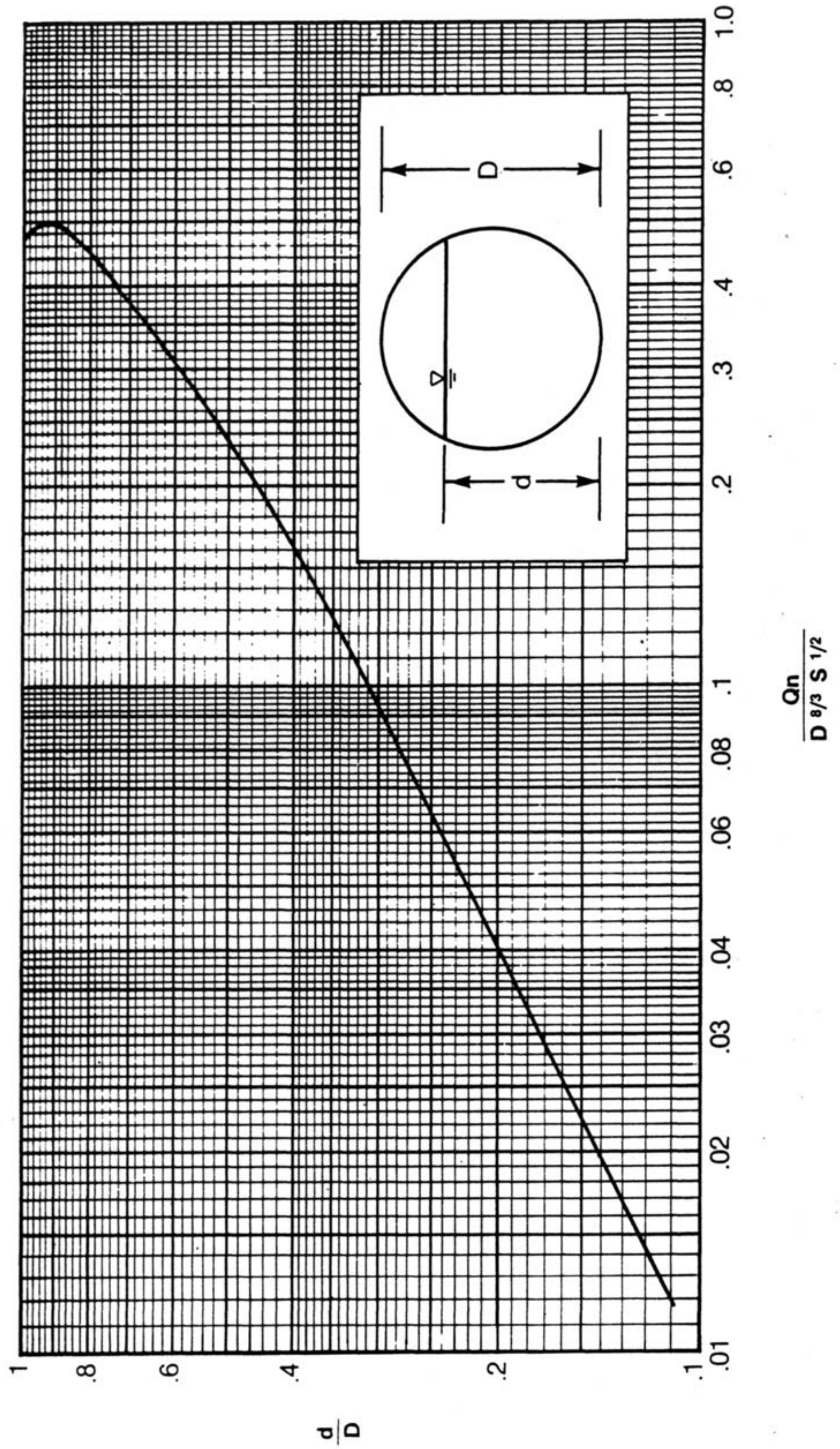

Figure 6-7
Circular Pipe Partial Flow Capacity Chart

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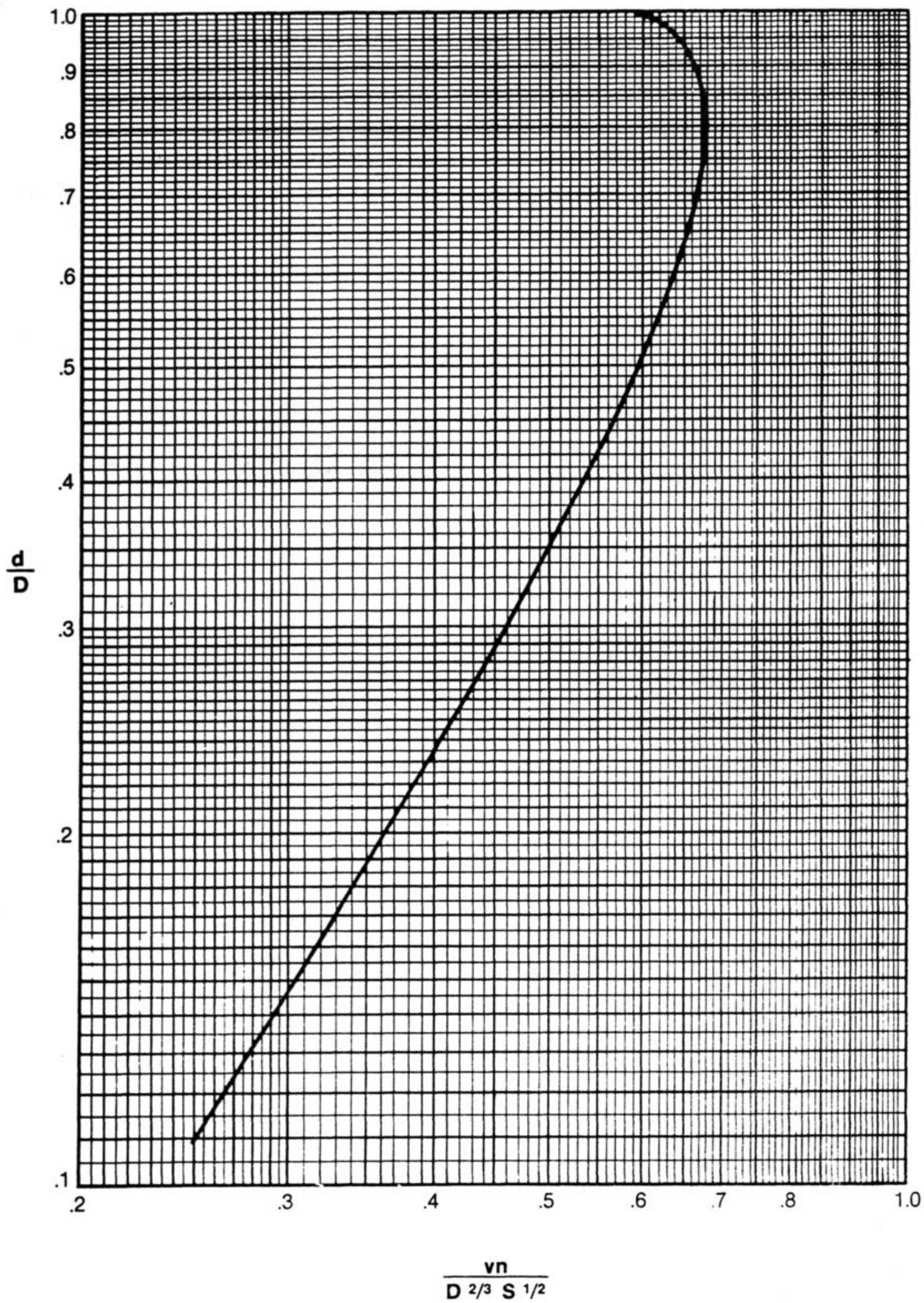

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Figure 6-8
Circular Pipe Partial Flow Velocity Chart

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 (C x Ca)	Inlet Flow Rate [c] (cfs)
1	2.0	8.0	6.4	.9	11.5
2	3.0	10.0	5.8	.9	15.7
3	2.5	9.0	6.1	.9	13.7
4	2.5	9.0	6.1	.9	13.7
5	2.0	8.0	6.4	.9	11.5
6	2.5	14.0	5.1	.9	11.5
7	2.0	8.0	6.4	.9	11.5

Notes:
[a] = Inlet and storm sewer system configuration are shown in Figure 6-1
[b] = Data for example calculations only. See Chapter 2 for IDF data
[c] = Calculated using the Rational Equation (see Chapter 2)

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Table 6-2
Results of Rational Method Calculations for the Hypothetical Storm Sewer System Shown in Figure 6-1

Storm[a] Sewer Segment	Tributary[b] Area (acres)	Design[b] Flow Rate (cfs)	Individual[b] Time of Concentration (min.)	Travel[c] Time (min.)	Cumulative[d] Time of Concentration (min.)	Runoff[a] Coefficient (CxCa)	Rainfall[e] Intensity (in./hr.)	Design Flow Rate (cfs)
$I_1 - M_1$	2.0	11.5	8.0	0.2				
$I_2 - M_1$	3.0	15.7	10.0	0.5				
$M_1 - M_2$	5.0			1.0	10.5	0.9	5.7	25.7
$I_3 - M_2$	2.5	13.7	9.0	0.2				
$I_4 - M_2$	2.5	13.7	9.0	0.5				
$M_2 - M_3$	10.0			2.0	11.5	0.9	5.5	49.5
$I_5 - M_3$	2.0	11.5	8.0	0.2				
$I_6 - M_3$	2.5	11.5	14.0	0.5				
$M_3 - M_4$	14.5			1.2	14.5	0.9	5.0	65.3
$I_7 - M_4$	2.0	11.5	8.0	0.5				
$M_4 - O$	16.5				15.7	0.9	4.6	68.3

Notes:

[a] = See Figure 6-1 for details
[b] = Tributary area data are presented in Table 6-1
[c] = Travel Time = pipe length/velocity/60 sec. per min
[d] = Includes travel time in pipe system
[e] = Data for example calculations only. See Chapter 2 for IDF data.

Back To : Section 6.4.1

Table 6-3
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)

Back to: Section 6.5.2

Table 6-4
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)

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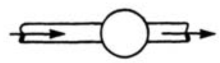
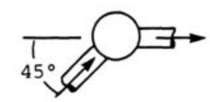
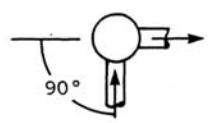
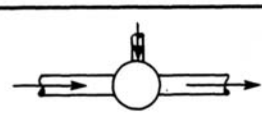
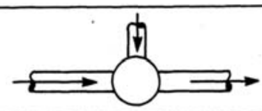
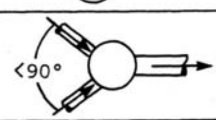
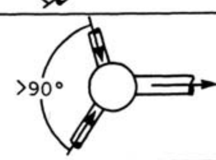
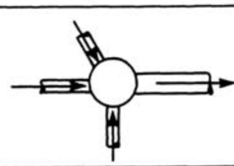
Table 6-5
Head Loss Coefficients for Manholes / Junctions

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.

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City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 7: Bridge Hydraulics

Volume 3: Best Management Practices Manual

Revision: 0
June 2006

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Acronym List
(Chapter 7)

AASHTO	American Association of State Highway and Transportation Officials
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
HEC-2	Hydraulic Engineering Circular No. 2
HEC-9	Hydraulic Engineering Circular No. 9
HEC-18	Hydraulic Engineering Circular No. 18
HEC-RAS	Hydrologic Engineering Center — River Analysis System
TDOT	Tennessee Department of Transportation
USDOT	United States Department of Transportation
USGS	United States Geological Survey
WSPRO	Water Surface Profile

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7.0 BRIDGE HYDRAULICS

7.1 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).

7.2 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. The user of this manual is specifically cautioned that the requirements of other governmental agencies such as the U.S. Army Corps of Engineers, Tennessee Department of Environment and Conservation, Tennessee Department of Transportation and the Federal Highway Administration are not addressed in this document.

7.2.1 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 to provide the clearances specified below. In addition, the structure should be designed so that the occurrence of the 500-year storm would not be expected to cause the structural failure of the bridge. 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 final selection of an appropriate design return period for a given bridge.

For consistency design peak flows for the specified return period should be obtained from federally funded flood studies (FEMA, Corps of Engineers, etc.) when available or readily deriveable for the project site. When such studies are not available the designer should generate design flows for the required return intervals using the procedures of Chapter 4 of this manual.

7.2.2 Approach Embankment Elevation

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. Proposed approach roadway elevations shall be set so that the entire pavement structure (base and surface courses) will be above the design high water elevation taking into account roadway cross slope and superelevation, if applicable. This requirement might be relaxed at the discretion of the City and/or County Engineer, if existing intersecting or connecting roadways which would have to be traversed to reach the proposed structure would be inundated for the design flood, making the proposed bridge inaccessible.

7.2.3 Clearances and Bridge Backwater

Low member clearance above design high water elevation shall be provided to prevent the occurrence of undue hydraulic forces against the bridge superstructure. In addition, vertical and horizontal bridge clearance requirements should consider the site-specific potential for blockage by debris and the need for passage of boat traffic. Guidance on estimating the potential for debris accumulation, analyzing and modeling debris accumulations to estimate effects on water surface profile and hydraulic loading on the bridge and design of debris countermeasures can be found in Hydraulic Engineering Circular No. 9 (HEC-9) Debris Control Structures Evaluation and Countermeasures by the USDOT, Federal Highway Administration. The method for establishing design values should be clearly documented.

Vertical Clearance

Unless a regulatory agency having appropriate jurisdiction (such as the U.S. Coast Guard or Corps of Engineers) has established higher values, the following minimum vertical clearances are required:

1. To allow debris to pass without causing damage, minimum clearance between design flood stage and the low member of the bridge shall be:

    a. High use or essential roadways — 2 feet
    b. Other roadways — 1 foot

2. For crossings subject to frequent small boat traffic, minimum clearance shall be:

    a. Rivers and streams — 6-foot clearance above mean annual flood stage

    b. Across lakes or other waterbodies which are subject to minimal fluctuations in elevation — 6-foot clearance above prevailing water elevation

Horizontal Clearance

In addition to allowing for passage of debris, horizontal clearance should be adequate to minimize encroachment and adverse backwater conditions caused by an excessive flow constriction (see Section 7.2). If costs are not prohibitively higher, bridges are preferred to multi-barreled culverts because of the reduced potential for debris blockage due to longer span length. The designer is cautioned, however, that except for perpendicular crossings (90 degree skew), horizontal clearance does not equal span distance. The span is measured center to center of the piers along the roadway centerline, while horizontal clearance is the projected clear area between adjacent substructure units taking into account the direction of travel of flow approaching the bridge.

Bridge backwater is also directly affected by the horizontal clearance of the structure. Backwater is the difference in the water surface elevation at a given location for a set flow with no road fill or structure present and the water surface elevation for the same flow with the proposed bridge and road fill in place. Allowable backwater for a proposed bridge will be determined on a case-by-case basis by the City and/or County Engineer taking into account factors such as the presence or absence of FEMA-designated floodways, existing upstream improvements and other appropriate site details.

7.2.4 Velocity and Scour Considerations

The average velocity is generally considered when the hydraulic capacity of a bridge opening is being evaluated, but localized velocities, particularly at proposed substructure units, must be determined for the theoretical scour calculations to be performed. 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.

Scour is defined in Hydraulic Engineering Circular No. 18 (HEC-18) Evaluating Scour At Bridges, USDOT, Federal Highway Administration 2001, as “the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams and from around the piers and abutments of bridges.” There have been several catastrophic bridge failures in West Tennessee, including some resulting in loss of life, where bridge scour has been at least a contributing factor.

Total scour consists of 1. Long-term streambed movement (aggradation or degradation) 2. General scour (contraction or other general scour) and 3. Local scour at piers or abutments. The cumulative effect of all the different types of scour must be taken into account in determining the proposed foundation elevations for new bridge. A good working knowledge of the principles and practices presented in HEC-18 are essential for the design of a stable bridge structure over a watercourse.

7.2.5 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 secondary channel(s) away from the main channel. The 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, and must also have foundations designed to withstand the predicted scour just like the main channel structure. These openings should be located and designed so they will not "invite" the main river to flow through them and potentially lead to channel realignment. Procedures for evaluating relief opening requirements are described in Chapter 4 of the WSPRO (HY-7) computer program documentation (USDOT, FHWA, 1986) and in Chapter 6 of the HEC-RAS User's Manual.

7.2.6 Spur Dikes

Given the depth to bedrock in Shelby County, bridge foundations will inevitably be set in erodible material. 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.

7.2.7 Structural Design

Bridges are to be designed in accordance with the latest edition of AASHTO Specifications for Highway Bridges, unless directed or approved otherwise by the City and/or County Engineer. The bridge shall be designed to resist the hydraulic force produced by the peak discharge from the 100-year storm in addition to all other required loads. In the structural design of the bridge it shall be assumed that the full extent of all the calculated scour has occurred at the time the design loading occurs.

In general, bridge substructure foundation elevations shall be set as follows:

Piers with footings constructed on top of piles — top of footing shall be set a minimum of 6 feet below the stable stream bed elevation, including channel degradation, if applicable, and at least as low as the contraction scour line.

Piles under footings or in pile bent substructure units shall have a penetration at least 10 feet below the computed elevation for the combination of all applicable scour components for the storm which produces the greatest amount of scour, up to the design storm.

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.

7.2.8 Bridge Deck Drainage

Proper drainage of the bridge deck is important because:

1. Deck structural and reinforcing steel is susceptible to corrosion from deicing salts

2. Moisture on bridge decks freezes before surface roadways

3. Hydroplaning often occurs at shallower depths on bridges due to the reduced surface texture of concrete bridge decks

Bridge deck drainage is often less efficient than roadway sections because cross slopes are flatter, parapets collect large amounts of debris, and drainage inlets or typical bridge scuppers are less hydraulically efficient and more easily clogged by debris. Because of the difficulties in providing for and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. For similar reasons, zero gradients and sag vertical curves should be avoided on bridges. Additionally, runoff from bridge decks should be collected immediately after it flows onto the subsequent roadway curb and gutter section where larger grates and inlet structures can be used. Where bridge deck flow exits onto a non-curb and gutter roadway section bridge end drains should be used unless it can be demonstrated that the exiting flow is minimal and means are provided to prevent erosion of the roadway embankment.

Bridge deck drainage systems shall be designed using a 10-year design storm and the principles discussed in earlier chapters on street drainage. If the use of a sag vertical curve on a bridge deck is unavoidable, then use of the 50-yr. design storm is recommended for the sag inlet and flanking inlets should be provided in case of clogging. Discharge from the bridge deck drains or drain downspouts may not be directed against bridge piers, girders, abutment berms, or other improvements in the vicinity of the bridge or in such a manner as to create an erosion problem.

7.3 Water Surface Profile And Scour Calculations

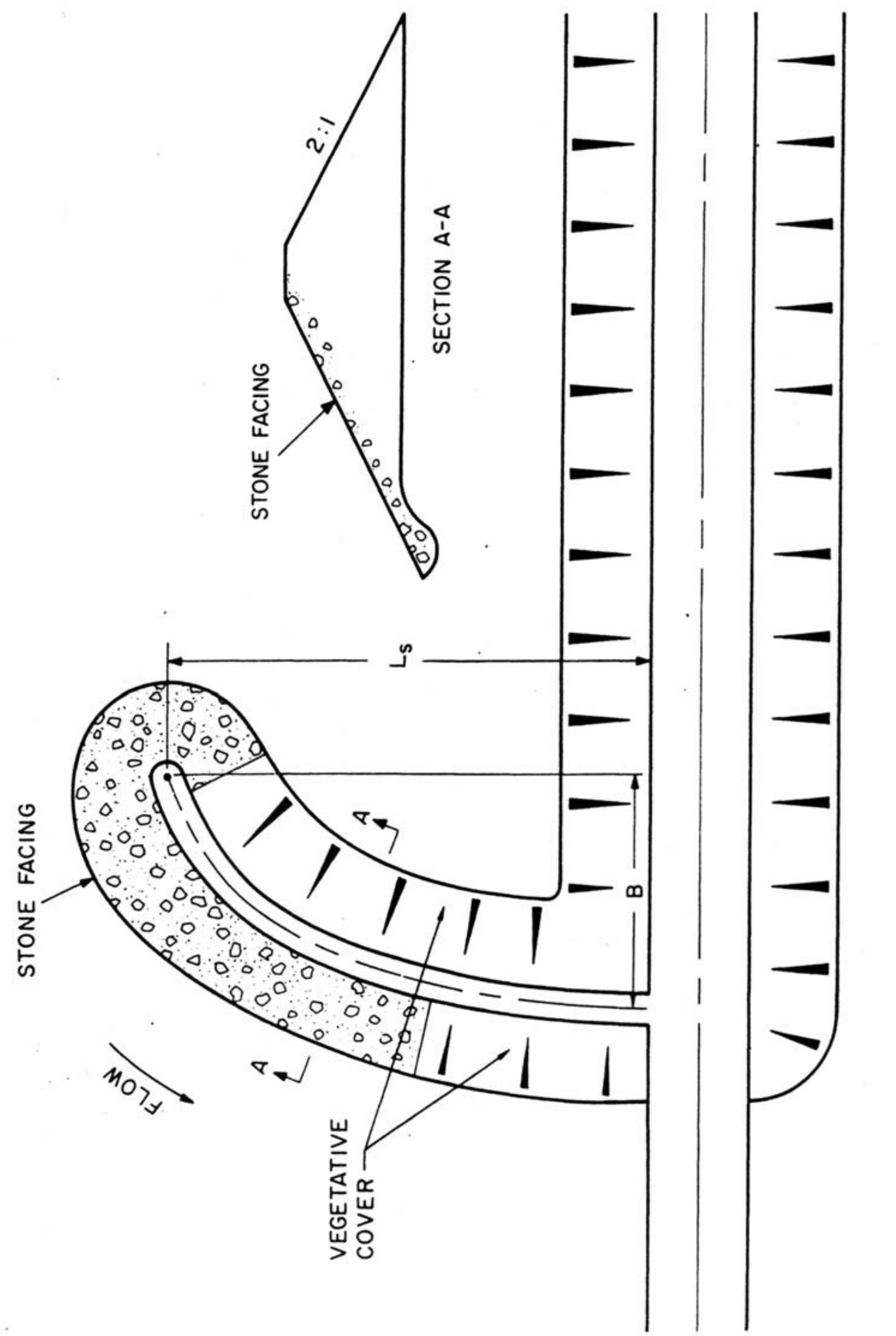
The procedure for performing water surface profile calculations at bridges should be consistent with the needs of the project. When changes to elevations and regulatory floodways presented on a Flood Insurance Rate Map or Floodway Maps are evaluated, consistent procedures should be used. This generally involves using the program and values of the original Flood Study to evaluate changes for approval.

Historically, the two most commonly used water surface profile computer programs in connection with bridge waterway hydraulics were HEC-2 by the U.S. Army Corps of Engineers, Hydrologic Engineering Center and WSPRO (HY-7) by the USDOT Federal Highway Administration. HEC-2 had been used successfully for years to calculate the effects of channel geometry, including encroachments such as bridges and roadway embankments, on water surface profiles. WSPRO also calculated water surface profiles, but added additional output options to enable the user to obtain more extensive information about the distribution of flow velocities across the channel and floodplain. The velocity information could then be used with the methods of Hydraulic Engineering Circular No. 18 (HEC-18), also by the Federal Highway Administration, to predict the amount of scour likely to occur at a given bridge location. HEC-2 eventually evolved into HEC-RAS (for River Analysis System) and current versions have been modified to incorporate elements of WSPRO and HEC-18 so that scour prediction calculations can now be made within the HEC-RAS software in addition to the calculation of project water surface elevations. In addition, on streams which have been studied previously and have existing HEC-2 models developed, this data can be imported into HEC-RAS models to reduce the amount of data input required and insure compatibility with current regulatory parameters. In order to facilitate the review of proposed bridge structures, the designer shall complete and submit a Hydraulic Report Summary Form with the bridge plans. A sample blank form modeled after the one used by the Tennessee Department of Transportation is included as Table 7-1.

7.4 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.

Figure 7-1
Plan and Cross Section of Spur Dike



Reference: USDOT, FHWA, HDS-1 (1978).

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Table 7-1
Bridge Design Data Summary Form
City Of Memphis, Tennessee
Shelby County, Tennessee

Date:______________ **Designer:**_______________________________________

A. SITE DATA

1. LOCATION

a. Name of Stream:
Channel Mile:___________________

b. Roadway Name: ____

c. Distance and Direction From Nearest Town:____

d. USGS 7½ Min. Quadrangle Name: ____

e. City or County: _______________________________

2. VICINITY

a. See attached location map or bridge survey.

b. Nature of Stream Bed Material:____

c. Is There Active or Recent Stream Bank Erosion at Site?: ____

d. Is Drift Present or Evidence of Removal in the Past (such as debris pile at site)?:____________________________

3. EXISTING BRIDGE DATA (IF APPLICABLE)

a. Existing Bridge Location Relative to Proposed New Structure:____

b. Bridge Length:______________ ft.

c. Bridge Width:_______________ ft.

d. Bridge Type: ____

e. Bridge Skew:___________degrees, LT. or RT.______

f. Drainage Area:_____________ mi.^{2}

g. Design Discharge:________________ ft.^{3}/sec.

h. Source of Design Discharge (FEMA, Designer, etc.):____

i. Design Storm Frequency:_________ yr.

j. Design Waterway Cross-sectional Area:_______________ ft.^{2}

k. Design Storm Water Surface Elevation:_______________ ft. (all elevations should be NGVD)

l. Design Backwater:____________ ft.

m. Design Velocity:_____________ ft./sec.

n. Overtopping Elevation:_____________ ft.

4. EXISTING WATER STAGES AT PROPOSED BRIDGE SITE

a. Maximum Recorded High Water Elev.:______________ft. Date:________________
Is Return Frequency Known?:______-yr. Source:____

b. Design Storm (___-yr.) Water Surface Elev. Without Proposed Bridge in Place:________ ft.

c. Is Site In a Reservoir? (Y/N):__, If Yes, Reservoir Name: ____
Normal Pool Elevation:________ ft. Minimum Pool Elevation:__________ ft.

e. Is Site Subject To Backwater/Tailwater Conditions From Another Waterbody? (Y/N):__

f. If Answer To e. Is Yes, Elev.:_________ft. Backwater Source:____

B. HYDROLOGICAL ANALYSIS

1. FLOOD RECORDS

a. Floods in Tennessee — Magnitude and Frequency — 1992 [ ] U.S.G.S. [ ]Corps of Engineers [ ] TVA [ ] Other [ ]
If Other, Specify:____

b. Stream Gage No.:_____________________________________ At Site [ ] In Vicinity [ ]

c. None Available [ ]

2. DRAINAGE AREA

a. _______ sq. mi. Calculated: _____ Published: ____

3. DISCHARGE

a. Magnitude: ________ _________ __________ __________ __________ __________ _________ , ft.^{3}/sec.
Frequency: 2 yr. 5 yr. 10 yr. 25 yr. 50 yr. 100 yr. 500 yr.

b. Proposed Overtopping: Frequency,____-year & Discharge,___________ ft.^{3}/sec.

c. Source: ____ Floods in Tennessee — Magnitude and Frequency — 1992
____ Corps of Engineers
____ Federal Insurance Study, Date: _____________________
____ Other (Specify)____

4. STREAM SLOPE

a. From USGS Quad Map:_____________ ft./ft.

b. From Site Survey Data:________________ ft./ft.

c. From Flood Flow Profiles: _____________ ft./ft.

C. HYDRAULIC ANALYSIS OF PROPOSED BRIDGE

1. PROPOSED STRUCTURE

a. Drainage Area:_________ mi.^{2}
Design Storm Frequency:____ yr. Design Discharge:_______________________ ft.^{3}/sec.
Design Velocity:___________ ft./sec. Design Bridge Backwater:________________ ft.
Design Bridge Backwater Elev.:________ ft. Roadway Overtopping Elevation:__________ ft.
Design Waterway Area:____________ ft.^{2} below elev.____________ ft.

b. Is Backwater From a Downstream Source a Consideration? (Y/N) :_____
If Yes,: Backwater Elevation = _________ft. For _____-Year Storm And
_________ft. For _____-Year Storm
Describe Backwater Source: ____

c. Are Spur Dikes Needed (Y/N): ___
Describe Reason: ____

d. Is Channel Transitioning Involved (Y/N): __ (If Yes, Attach Detail)

e. Is Channel Change Involved (Y/N): __ (If Yes, Attach Detail)

f. Is Bank Protection Needed (Y/N): __ (If Yes, Attach Detail)

g. Final Horizontal Layout: See Attached Drawing No. ________________

h. Final Bridge Elevation View: See Attached Drawing No. ________________

D. SCOUR ANALYSIS OF PROPOSED BRIDGE

1. CHANNEL CHARACTERISTICS

a. USGS/TDOT "observed" scour ranking at existing bridge is, or at nearest bridge upstream [ ] /downstream [ ] is ______________. Identify Bridge:____

b. USGS/TDOT "potential" scour ranking at existing bridge is, or at nearest bridge upstream [ ]/downstream [ ] is______________. Identify Bridge:____

c. Current stage of channel evolution: Stable [ ] Degrading [ ] Widening [ ] Aggrading [ ]

d. Streambed material type: silt/sand [ ]; coarse gravely sand [ ]; gravel/cobbles [ ]

2. COMPUTED SCOUR DEPTH

(Provide Values Below For Most Scour-Critical Substructure Unit And Attach Detail or Show on Other Attachments The Total Scour Line For The Full Bridge Section)

a. Design discharge (____ yr.) =___________ $ft.^3/sec.$

b. Design average velocity (____ yr.) =______ ft./sec.

c. Estimated long-term degradation [ ] /aggradations [ ] =_____ ft.

d. Estimated contraction scour =______ ft.

e. Estimated pier scour =______ ft., at (identify location) ____

f. Estimated total scour depth =______ ft., at____

g. Preliminary top of footing elev.: _____________ ft. (based on soils report? Y/N):______
and/or pile tip elev.: _____________ ft. (based on soils report? Y/N): ______

h. Comments: ____

____

____

____

E. GENERAL SITE COMMENTS OR REMARKS

____

____

____

____

____

Is the location governed by the National Flood Insurance Program Regulations? (Y/N): _____

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City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 8: Detention/Retention Hydraulics

Volume 3: Best Management Practices Manual

Revision: 0
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Acronym List
(Chapter 8)

Θ	Angle of v-notch, in degrees
A	Area of orifice, (ft^2)
BMPs	Best Management Practices
C	Orifice coefficient
C	Broad-crested weir coefficient
C_1	Discharge coefficient
CAD	Computer Aided Design
CDMM	Camp Dresser and McKee, Inc.
cfs	Cubic feet per second
C_i	Weir discharge coefficient
dt	Routing computational interval, (sec)
EL_i	Invert elevation of weir opening i, (ft)
EL_{WS}	Elevation of the water surface, (ft)
FEMA	Federal Emergency Management Agency
g	Acceleration due to gravity, (32.174 ft/sec^2)
H_1	Upstream head above crest, (ft)
H_2	Downstream head above crest, (ft)
H	Head on Vortex of notch, (ft)
H	Head above orifice centroid, (ft)
H	Head above weir crest, (ft)
H	Head above the weir crest excluding velocity head, (ft)
H^a	Measured head, at least 2.5H upstream of the weir
H_c	Height of weir crest above channel bottom, (ft)
HEC-HMS	Hydrologic Engineering Center — Hydrologic Modeling System
L	Horizontal weir length, (ft)
L	Broad-crested weir length, (ft)
L_i	Length of weir opening i, (ft)
O_1	Outflow rate at time 1, (cfs)
O_2	Outflow rate at time 2, (cfs)
O_3	Outflow rate at time 3, (cfs)
PMP	Probable maximum precipitation
Q	Discharge, (cfs)
Q_f	Free flow, (cfs)
Q_i	Peak inflow rate
Q_i	Cumulative discharge above weir opening i, (cfs)
Q_o	Peak outflow rate
Q_s	Submergence flow, (cfs)

SCS	Soil Conservation Services
SWMM	Storm Water Management Model
TDEC	Tennessee Department of Environment and Conservation
T_i	Duration of basin inflow
USEPA	United States Environmental Protection Agency
USDA	United States Department of Agriculture
V_s	Storage volume estimate
V_t	Total storm water volume discharged during designated period, (ft^3)

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8.0 DENTENTION/RETENTION HYDRAULICS

8.1 Synopsis

Land development activities often alter the hydrologic characteristics of a watershed, which may in turn affect the timing, velocity, magnitude, and quality of runoff. Storm water detention/retention to address quantity and quality is required by City of Memphis and Shelby County Drainage Design policy (see Volume 1 — Sections 2.4, Storm Water Quantity and Quality Detention) to control and mitigate adverse impacts on existing downstream drainage systems caused by development upstream. BMPs provide a series of controls discussed in Volume 3, which detention and retention systems are a component.

This chapter provides general design criteria for detention/retention basins as well as procedures for performing preliminary sizing and acceptable methods of making final reservoir routing calculations. HEC-HMS from the U.S. Army Corps of Engineers Hydrologic Engineering Center and USEPA's SWMM are acceptable public domain software for performing detention routing calculations. Other proprietary software already being used by a designer may be allowed on a case-by-case basis, however the City and/or County may require that it be run with a "standard" set of input parameters to demonstrate that it produces output values comparable to the accepted methods. Volume 2 — Section 1 of this manual contains a standardized data sheet for listing the site parameters that the designer of a detention/retention facility will be required to include with the design submittal in order for the City and/or County to effectively review the proposed structure(s). Flood-prone drainage areas may require retention/detention storage areas designed to address special conditions to protect public safety from flooding and/or facility failure. Wet detention facilities with a permanent pool also require special design considerations to protect the public health and safety. This chapter is supported and augmented by Volume 3 — Section 4.5, Storm Water Treatment Removal Goals.

8.2 Design Criteria

In general, detention facilities offer temporary storm water storage accompanied by controlled release of the stored water, while retention refers to storm water storage without access to a positive outlet below a certain elevation. Since retention facilities will impound a permanent pool of water below the lowest outlet elevation, water budget calculations may be required at the discretion of the City and/or County if the facility is intended to serve as an aesthetic element for the development of which it is a part.

The design considerations for detention/retention facilities should include:

1. Multiple systems and Best Management Practices (BMPs)
2. Maximum allowable outflow rate for design storm
3. Detention volumes ("live" pool and permanent pool, if applicable)
4. Grading, depth, and geometry requirements
5. Outlet structure(s) performance throughout its expected range of operation

8.2.1 Multiple Systems and Best Management Practices

Detention and retention can be used separately or together in series or parallel to other storm water BMPs to offer cumulative benefit to storm water quantity and/or quality. Selecting a series of practices is discussed in Volume 3 — Section 4.2, Identify Objectives, including how to select BMP treatment options.

8.2.2 Release Rates

Outlet structure peak release rate for the 10-year, 24-hour duration storm shall not exceed the pre-development peak runoff rate for the same storm, with emergency overflow capable of handling the 100-year post-development discharge, except where waived or altered by the City and/or County. The release rate for the two-year and five-year 24-hour duration storms shall be controlled so that the volume of water released from hour 11 to hour 18 for the post-development storm does not exceed the volume of runoff that would have occurred during the same time period for the same storm, respectively, for pre-development conditions. Studies done by the consulting firm of Camp Dresser and McKee, Inc. (CDM) on both flat and steep drainage systems having a travel time of less than 24 hours indicate that limiting post-development outflow volumes to pre-development values during the hour 11 to hour 18 "time window" can be effective in preventing downstream flooding from increased runoff volume due to basin development. If a proposed project lies within the limits of a watershed for which an approved Master Drainage Plan has been done and a basin-specific critical time window has been identified, then that time window should be used for volume detention calculations in lieu of the standard hour 11 to 18 window.

8.2.3 Detention Volume

Detention volume shall be adequate to attenuate the post-development peak discharge rate to the allowable rate determined for Section 8.5 and 8.6. Additionally, sufficient storage shall be provided so that the volume of runoff discharged from the structure from hour 11 to hour 18 of the 24-hour post-development two-year and five-year storms shall not exceed the volume passed downstream during the same time period for the corresponding pre-development storm. 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 silt accumulation during construction is in excess of sedimentation estimates, then the permanent detention volume design dimensions shall be restored before as-built certification is submitted. The detention volume should be oversized to allow for long-term (five to 10 years) sediment storage.

8.2.4 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 8.2.5. Storm water 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. Geotechnical slope stability analysis is recommended and may be required for embankments greater than 6 feet in height and for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soils engineering textbooks, including those by Spangler and Handy (1982) and Sowers and Sowers (1970).

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 designed to control that flow, such as landscape swales and biofilters.

The bottom area of dry detention facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2% 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.

The maximum depth of storm water 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 emergent aquatic vegetation (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 conditions that could result in anaerobic conditions and odor problems.

Other considerations when setting depths include regulatory 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. Embankments greater than 20 feet in height (above the stream bed elevation at the downstream toe), or impounding more than 30 acre-feet of water are generally subject to the requirements of the 1973 Safe Dams Act (see Section 8.2.5 and Section 8.2.6).

8.2.5 Outlet Structure

Outlet structures selected for detention/retention facilities should typically include a principal outlet structure (riser, weir, orifice, etc.) and an emergency overflow and must be able to accomplish the several requirements of these regulations. Outlet structures can take the form of drop inlets or any combination of pipes, weirs, orifices, and other geometric structures such as shown in Figure 8-1. The principal outlet is intended to convey the two-, five-, and 10-year post-development design storms while effecting the required attenuation specified above 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 8.2.2, 8.2.3, and 8.2.4.

8.2.6 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 Tennessee Department of Environment and Conservation (TDEC), Division of Water Supply. Rules and regulations relating to applicable dams are promulgated by that 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, see Table 8-1. Exceptions are that dams less than 6 feet in height (regardless of potential storage) or impounding less than 15 acre-feet (regardless of height) are excluded. Several additional exemptions are allowed from the Safe Dams Act and the applicable state office should be contacted to resolve questions.

Dams subject to regulation under the Safe Dams Act must meet certain design requirements as detailed in TDEC's "Chapter 1200-5-7 Rules and Regulations Applied To The Safe Dams Act of 1973." Compliance with the appropriate provisions of those regulations shall not be considered as a waiver of any of the requirements of this document, or vice versa. Should conflicts arise concerning design requirements of the two entities, it should be brought to their attention in order that they may reach a mutually agreeable compromise. Applicable regulations should be consulted for further details and engineering requirements.

8.3 General Water Quantity Procedures

The following three relationships should be considered when sizing a storm water detention facility:

1. Inflow hydrographs for a range of design storms (see Chapter 2). This should include the two-, five-, 10-, and 100-year events.

2. Stage-area-storage curve for the detention basin (see Figure 8-2 for an example).

3. Stage-discharge curve for basin outlet control structure (see Figure 8-3 for an example).

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 two-, five-, 10-, and 100-year design storms, as required in Volume 1, using procedures from Chapter 2. Both pre- and post-development hydrographs are required for the two-, five-, and 10-year design storms. Only the post-development hydrograph is required for the 100-year design storm.

2. Perform preliminary calculations to evaluate detention storage requirements (see Section 8.5) for the two- and five-year hydrographs from Step 1.

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 an appropriate computer model or manual method. If the routed post-development peak discharges from the 10-year design storm exceed the pre-development peak discharge, 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. Consider emergency overflow from the 100-year design storm and establish freeboard requirements.

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 may be required to show that downstream impacts can be controlled.

7. Evaluate the control structure outlet velocity and provide stabilization if velocity is greater than 4 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).

8.4 Outlet Hydraulics

Runoff volume and peak outflow rate detention requirements herein will normally dictate the use of a well-designed outlet structure for the release of excess storm water. For the convenience of the designers using this document, accepted equations for calculating the theoretical discharge of various configurations of outlet devices are given below. These include sharp-crested weir flow equations for no-end contractions, two-end contractions, and submerged discharge conditions, 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, and all outlet structure designs should incorporate measures to prevent debris accumulations from damaging the structure or affecting its hydraulic capacity.

8.4.1 Sharp-Crested Weirs—No End Contractions

A sharp-crested weir with no end contractions is illustrated in Part A of Figure 8-4. 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 8-4, Part C)
- H_c = Height of weir crest above channel bottom, in feet (see Figure 8-4, 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 8-4. 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 8-4, Part C)
- H_c = Height of weir crest above channel bottom, in feet (see Figure 8-4, 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):

$$Q = C_1 \tan\left(\frac{\theta}{2}\right) H^{2.5} \qquad (8\text{-}5)$$

Where:

C_1 = discharge coefficient (See Figure 8-5)

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:

$$Q = 4.97\, a^{1/2}\, b(H - a/3) \qquad (8\text{-}6)$$

$$x/b = 1 - \frac{1}{\pi}\left(\arctan\sqrt{y/a}\right) \qquad (8\text{-}7)$$

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, but if further refinement is desired a hydraulics handbook (such as Brater and King, 1976) can be consulted
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, but it should be noted the accuracy of the estimate is significantly affected by the time base used for the schematic triangular hydrograph used to approximate the curvilinear design hydrograph (see Figure 8-7 for graphical representation of terms).

8.6 Routing Calculations

The use of computerized hydraulic routing models such as HEC-HMS from the U.S. Army Corps of Engineers Hydrologic Engineering Center or SWMM from USEPA are required for final reservoir routing calculations for detention facility final design. Other proprietary software already being used by a designer may be allowed on a case-by-case basis, however the City and/or County may require that it be run with a "standard" set of input parameters to demonstrate that it produces output values comparable to the accepted methods. Whatever method is used must be capable of providing detailed time series data for hydrographs it generates in order that storm water mass outflow volumes can be computed from hour 11 until hour 18 for both pre- and post-construction conditions so that compliance with volume detention requirements can be verified.

8.6.1 Detention/Retention Basin Routing Calculations

The following procedure is used to perform the required detention calculations:

1. Develop an outflow hydrograph (or at least calculate the peak flow) for the area(s) proposed to be developed representing the resultant runoff from the 10-year, 24-hour theoretical storm for the area in its existing condition before any of the proposed improvements are installed. This value then becomes the maximum allowable outflow value for the same storm occurring on the area(s) after installation of the proposed improvements. For the typical site development this will obviously require the installation of

some type of detention structure. The two-year and five-year, 24-hour pre-construction hydrographs should also be developed at this time for use, as discussed in Step 4 below. This is usually fairly easily accomplished by adding additional rainfall totals for these storms to multi-ratio runs or running multiple meteorological models with the same basin data and control specifications.

2. Develop an outflow hydrograph for the area(s) proposed to be developed representing the resultant runoff from the 10-year, 24-hour theoretical storm for the area when all of the proposed improvements are installed. Input parameters should accurately reflect all factors affecting the amount and timing of the storm water runoff, such as: increased percentage of impervious surface and resulting decreases in rainfall abstractions, shortened basin response times due to increased flow velocities in both overland and concentrated flow regimes, and so forth.

3. Route the hydrograph from Step 2 through a structure designed in accordance with the requirements of this document such that it provides adequate temporary storage of storm water and appropriate outlet works so that the peak discharge from the structure does not exceed the value established in Step 1. All flow from the design (10-year, 24-hour) storm must be handled by the low-level (primary) outlet without any flow getting into the high-level (emergency) outlet discussed in Step 7 below.

4. In addition to meeting peak flow-rate reduction requirements, detention structures must be sized so that the cumulative volume that passes out of the structure from hour 11 to hour 18 of the 24-hour storm (or a different site-specific time window) for the two-year and five-year post-development hydrographs does not exceed the volume passed downstream for the same time period for the respective pre-development storms. For any given combination of available storage and discharge capacity, this requirement may be more restrictive than the peak discharge reduction criteria alone.

5. The volume of storm water discharged during the designated time period is computed as the accumulation of the incremental volumes for each computational interval during the period. Expressed algebraically this is:

$$V_t = [(O_1 + O_2)/2 \text{ x } dt] + [(O_2 + O_3)/2 \text{ x } dt] + [(O_x + O_y)/2 \text{ x } dt] \qquad (8\text{-}10)$$

Where:

V_t = Total storm water volume discharged during designated period, in cubic feet
O_1 = Outflow rate at time 1, in cfs
O_2 = Outflow rate at time 2, in cfs
O_3 = Outflow rate at time 3, in cfs
dt = Routing computational interval, in seconds

This equation assumes that the computational interval used is uniform throughout the designated period. If this were not the case, then the appropriate time interval would need to be used for the individual step being calculated. This calculation is most easily done by exporting the time and outflow data from the computer routing output for the specified time period and importing it into a relatively simple spreadsheet for performing the extensive calculations.

6. Alternately, if the routing software produces hydrographs that can be exported as files useable by the designer's CAD software, the area under the appropriate hydrograph between the hours designated would represent the outflow volume for that period, assuming the proper unit's conversions are taken into account.

7. Once a basin/outlet works combination has been developed that meets the three attenuation requirements (10-year peak outflow, five-year volume, and two-year volume) the designer should proceed to provide a means to convey the 100-year, 24-hour post-development storm through the structure without overtopping of any portion of the impoundment embankment or inundation of areas adjacent to the detention structure. This requirement is normally accomplished by the use of an open overflow section at an appropriate elevation below the top of the basin embankment. It is preferred that this overflow section (emergency spillway) be excavated in native earth outside the limits of the embankment and well vegetated to provide erosion protection in the event it is activated. Where available space or traffic circulation requirements make this impractical, consideration will be given to the use of a well-armored overflow section and outfall channel within the limits of the embankment, or the use of a closed-type (box culvert) emergency spillway. If a closed structure is used it must be capable of carrying the design discharge under gravity flow conditions (not pressure flow) and should be sized so that it is not readily susceptible to clogging from floating debris.

8. Finally, as a safety precaution considering the uncertainties involved with the routing of theoretical storm events, the minimum crown elevation of the detention structure embankment or finished grade of all areas adjacent to the structure should be set at least 1 foot above the peak elevation calculated for water surface in the routing of the 100-year, 24-hour storm. This requirement is not to be construed to supersede any minimum elevation requirements established by FEMA or other regulatory agencies, see Table 8-2.

9. The input parameters, equations, output results, and backup data assimilated in the design process are required to be submitted with a proposed detention/retention basin design to allow the City and/or County to efficiently review the design.

8.6.2 Example Problems

Example 8-1. Routing Using the HEC-HMS Computer Software

Because of its ready availability as a free download and its successful use over an extended period of time and a broad range of applications, the HEC-HMS software is highly recommended as a tool for performing the routing calculations necessary to meet the requirements of this document. An example application of a HEC-HMS model using data and hydrographs from Example 2-10 is presented below. The example is for a 35-acre (0.0547-square-mile) site that is to be developed as a single-family residential subdivision with RS-8 zoning. In Example 2-10, the following results were obtained:

1. Pre-development weighted Runoff Curve Number = 68
   Pre-development Initial Abstraction = 0.942 in.
   Pre-development SCS Lag Time = 12.4 min.

   Using the methods discussed in Chapter 2 and the changes that occur in the basin due to the proposed development, the following were calculated:

   Post-development weighted Runoff Curve Number = 77
   Post-development Initial Abstraction = 0.597 in.
   Post-development SCS Lag Time = 9.5 min.

   Pre-development peak flow for the 10-yr, 24-hr. storm = 87.4 cfs
   Post-development peak flow for the 10-yr, 24-hr. storm = 134.7 cfs
   Pre-development two-yr volume discharge, hr. 11 to 18 = 127,448 ft^3
   Post-development two-yr volume discharge, hr. 11 to 18 = 195,496 ft^3
   Pre-development five-yr volume discharge, hr. 11 to 18 = 192,448 ft^3
   Post-development five-yr volume discharge, hr. 11 to 18 = 270,788 ft^3

Peak flows shown above are given directly by software standard output tables. Volumes shown were calculated using the methodology discussed in Step 5 above. The time series data for the appropriate portion of the hydrograph under consideration were exported into a Microsoft Excel spreadsheet that had been set up to average the flow for the computation interval and multiply that average flow in cfs by the time interval in seconds yielding the outflow volume in cubic feet for each interval. These individual volumes were then accumulated over the required time period to give total volume of outflow for the period of interest.

2. Estimate the required storage volume using equation 8-9.

   $V = 0.5 * (2.4 \text{ hr.}) * (134.7\text{cfs} - 87.4\text{cfs}) * 3600 \text{ sec./hr.} = 204{,}336 \text{ ft}^3$

   The area where the example detention pond is to be built will allow for a practical depth of about 6 ft, so as a rough first estimate, the average pool size will be about:

   $204{,}336/6 = 34{,}056 \text{ ft}^2$, or approximately 185 ft x 185 ft

   This is only approximate and the final dimensions will be determined by the actual geometry of the pond and outlet structures.

3. Assuming that the pool size calculated above occurs at mid-depth of 3 ft and that the side slopes of the pond are on 3H:1V slopes, this would give a pond dimension of 170 ft x 170 ft for the bottom. Inspection of the data from Step 1 above shows that the runoff volume produced by the two-year, 24-hour post-development storm during the specified period is 195,496 ft^3, while the pre-development two-year volume discharge is 127,448 ft^3 (which is also the maximum allowable pond discharge). The difference of 195,496 — 127,448 = 68,048 ft^3 would need to be stored by the pond. Using the pond dimensions given above, this amount of storage would occur at a depth of about 2.4 ft, which appears reasonable given the desired maximum depth of 6 ft.

4. The other element required to perform the flood routing is the outlet structure from the pond. Since the pond will need to discharge approximately 127,500 ft^3 in a period of seven hours (from hour 11 to hour 18 as defined in section 8.2.3) the average discharge will need to be:

127,500 ft^3 / (7 hrs. x 3600 sec./hr.) or 5.1 cfs. Since this represents the average discharge, the peak will be assumed to be twice this, or 10.2 cfs at a hydraulic head of about 2.4 ft Use of culvert design nomographs or software show that this requirement could be met with a 18-inch concrete pipe and headwall arrangement, but this scenario could not accommodate the higher flows of the other design storms (such as the 87.4 cfs for the 10-yr storm) even with excessive headwater elevations. If the pipe size were increased to accommodate the 10-yr discharge, then the flows for the two-year and five-year storms would be excessive and the volume discharge would exceed the allowable amounts.

5. For the reasons discussed in Step 4. it appears a compound type outlet structure will be required. The first trial will be for an 18-inch diameter orifice. With a water depth of 2.4 feet, the head H would be:

   2.4 — (9/12) = 1.65 ft, and the discharge from the orifice equation is:

   $Q = 0.6\ A\ (\ 2gH\ )^{0.5} = 0.6 \times 1.767 \times (64.34 \times 1.65)^{0.5} = 10.9$ cfs (which is approximately equal to the 10.2 cfs estimated in Step 4 above)

6. The assumed pond and orifice dimensions from Step 5 were next entered as a "reservoir" into the HEC-HMS model developed previously and the model was run for the 2-year precipitation event. The outflow during the period of interest was calculated using a computer spreadsheet to be 151,073 $ft.^3$ This significantly exceeded the allowable value of 127,448 ft^3 indicating that the diameter of the outlet needs to be reduced and the size of the pond needs to be expanded. For the next trial the pond bottom will be increased to 200 ft x 200 ft. In addition, the diameter of the orifice will be decreased to 14 inches.

7. Data was input to the HEC-HMS model for a pond having a 200 ft x 200 ft bottom and a 14-inch diameter orifice outlet. This run produced a peak stage of 102.63, which was acceptable, but the accumulated outflow amounted to 138,141 ft^3, which was still in excess of the allowable value of 127,448 by about 8%.

8. It was felt that the 14-inch diameter was as small as was practical, so the size of the basin was increased again, this time to 225 ft x 225 ft. This trial produced satisfactory results with a peak stage of 102.19 and outflow volume of 125,395 ft^3, which is below the allowable of 127,448 ft^3. Next the model was rerun with the five-year and 10-year storms

to see how the pond performed. This configuration would handle the other design storms, but the outflow volume from the five-year storm only amounted to 164,403 ft^3 compared to an allowable maximum of 192,448 ft^3 and the peak discharge from the 10-year storm (9.4 cfs) only reaching about 11% of its allowable value (87.4 cfs). In order to add additional discharge and more efficiently use the available storage, a 16-foot-long weir was added to the outlet structure at elevation 102.25, just above the peak stage of the two-year routing.

9. Re-doing the routing with the revised outlet produced a peak stage of 102.73 for the five-year storm routing and a discharge volume of 191,099 ft^3, which is slightly below the allowable value of 192,448 ft^3 and is acceptable. This configuration was also checked for the 10-year storm and produced a peak discharge of 46.1 cfs compared to an allowable of 87.4 cfs and a peak stage of 103.09. This outflow structure could be built as an inlet-type box with a low-level 14-inch orifice in one wall with the bottom of the orifice at elevation 100.0 and the top of the box at elevation 102.25 to act as the weir portion of the structure. An outlet pipe would have to be provided with adequate capacity to prevent interference with the operation of the orifice and weir. Alternately, it could be constructed as an exit channel from the pond with a wall serving as the weir and the orifice placed at the lower elevation through the wall. As with the box structure, the outlet channel would have to have sufficient capacity to allow the elements to function as designed.

10. The final required hydrologic criteria is that the pond be able to pass the 100-year, 24-hour storm without overtopping of the embankment. The same configuration as modeled in Step 9. above was run for the 100-year storm and was able to carry the required flow peaking at elevation 104.14 with a peak flow of 139.0 cfs. The top of the embankment would then need to be at or above 105.15 elevation. This scenario would be acceptable, but the caveat about the capacity of the outfall system would remain in effect, meaning that the outfall would have to accommodate a flow of nearly 140 cfs instead of approximately 50 for the 10-year storm. Probably a more cost-effective manner to accommodate the 100-year storm is with a lowered overflow section serving as an emergency spillway.

11. The model was rerun once more adding an additional emergency spillway (broad-crested weir) at elevation 103.1, which is just above the peak stage from the 10-year storm routing (103.09). A trial length of 30 feet was used for the emergency spillway, which produced a peak stage of 103.86, setting the minimum top of embankment at 104.86. This scenario would allow the top of the embankment to be set about 0.3 feet lower than using the

results of Step 9. This would save on embankment material costs, but the biggest savings would probably be in providing an outfall pipe with a capacity of only 50 cfs compared to one with a capacity of nearly 140 cfs. The designer is cautioned that the calculations shown above only account for the required storm water storage volumes. Appropriate accommodations for sediment storage during both the construction phase and long-term operation of the site must also be made as discussed in other sections of this manual.

8.6.3 NRCS TR-55 Method

The USDA, NRCS now has a Windows-based computerized version of its graphical method for estimating the peak flow reduction capability of detention ponds (WinTR-55). This method is not discussed in detail in this manual, since it is only recommended for preliminary calculations, however it does incorporate automated modules for calculating basin time of concentration and weighted runoff curve numbers, which the designer may find convenient to use.

8.7 Permanent Pool Facilities

The City and/or County recognizes that at times permanent pool facilities may be designed for use as detention/retention structures. It also recognizes that wet detention ponds may be 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. The City and/or County may reject facility designs with the potential for becoming nuisances or health hazards.

8.8 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, the City and/or County discourages the design of storm water 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.

8.9 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).

8.10 Chapter Equations

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right)LH^{1.5} \tag{8-1}$$

Where:

Q = Discharge, in cfs
H = Head above the weir crest excluding velocity head, in feet (see Figure 8-4, Part C)
H_c = Height of weir crest above channel bottom, in feet (see Figure 8-4, Part C)
L = Horizontal weir length, in feet\

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right)(L - 0.2H)H^{1.5} \tag{8-2}$$

Where:

Q = Discharge, in cfs
H = Head above the weir crest excluding velocity head, in feet (see Figure 8-4, Part C)
H_c = Height of weir crest above channel bottom, in feet (see Figure 8-4, Part C)
L = Horizontal weir length, in feet

$$Q_s = Q_f\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \tag{8-3}$$

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

$$Q = C\,L\,H^{1.5} \tag{8-4}$$

Where:

Q = Discharge, in cfs
C = Broad-crested weir coefficient
L = Broad-crested weir length, in feet
H = Head above weir crest, in feet

$$Q = C_1 \tan\left(\frac{\theta}{2}\right) H^{2.5} \quad (8\text{-}5)$$

Where:

C_1	=	discharge coefficient (See Figure 8-5)
Q	=	Discharge, in cfs
θ	=	Angle of v-notch, in degrees
H	=	Head on vortex of notch, in feet

$$Q = 4.97\, a^{1/2}\, b(H - a/3) \quad (8\text{-}6)$$

$$x/b = 1 - \frac{1}{\pi}\left(\arctan\sqrt{y/a}\right) \quad (8\text{-}7)$$

where Q is the weir discharge, in cfs, and the dimensions a, b, H, x, and y are shown in Figure 8-6.

$$Q = CA\,(2gH)^{0.5} \quad (8\text{-}8)$$

Where:

Q	=	Discharge, in cfs
C	=	Orifice coefficient, a value of 0.6 is usually appropriate, but if further refinement is desired a hydraulics handbook (such as Brater and King, 1976) can be consulted
A	=	Area of orifice, in square feet
g	=	Acceleration due to gravity, 32.174 feet/second^2
H	=	Head above orifice centroid, in feet

$$V_s = 0.5\, T_i\, (Q_i - Q_o) \quad (8\text{-}9)$$

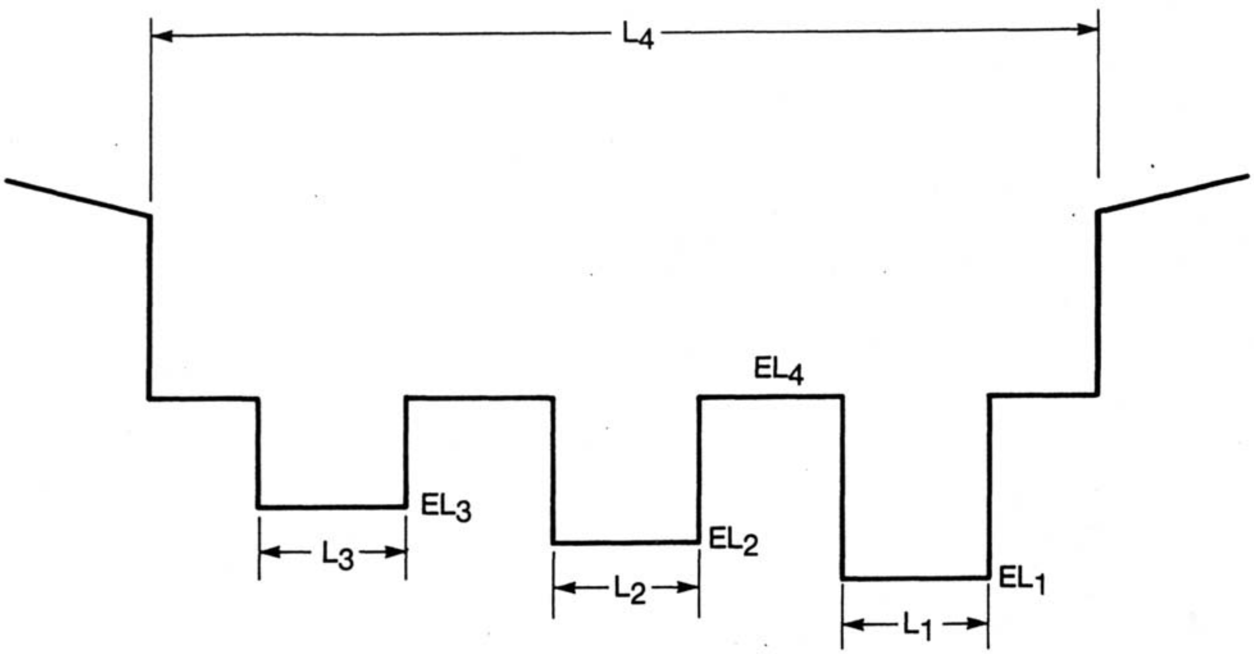
Where:

V_s	=	storage volume estimate
Q_i	=	Peak inflow rate
Q_o	=	Peak outflow rate
T_i	=	Duration of basin inflow

$$V_t = [(O_1 + O_2)/2 \text{ x } dt] + [(O_2 + O_3)/2 \text{ x } dt] + \ldots\ldots[(O_x + O_y)/2 \text{ x } dt] \qquad (8\text{-}10)$$

Where:

V_t = Total storm water volume discharged during designated period, in cubic feet
O_1 = Outflow rate at time 1, in cfs
O_2 = Outflow rate at time 2, in cfs
O_3 = Outflow rate at time 3, in cfs
dt = Routing computational interval, in seconds



TYPICAL ELEVATION

For Water Surface Elevation

From	To	Discharge Is Computed By
EL_1	EL_2	$Q_1 = C_1L_1 (EL_{WS} - EL_1)^{3/2}$
EL_2	EL_3	$Q_2 = C_2L_2 (EL_{WS} - EL_2)^{3/2} + Q_1$
EL_3	EL_4	$Q_3 = C_3L_3 (EL_{WS} - EL_3)^{3/2} + Q_2$
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

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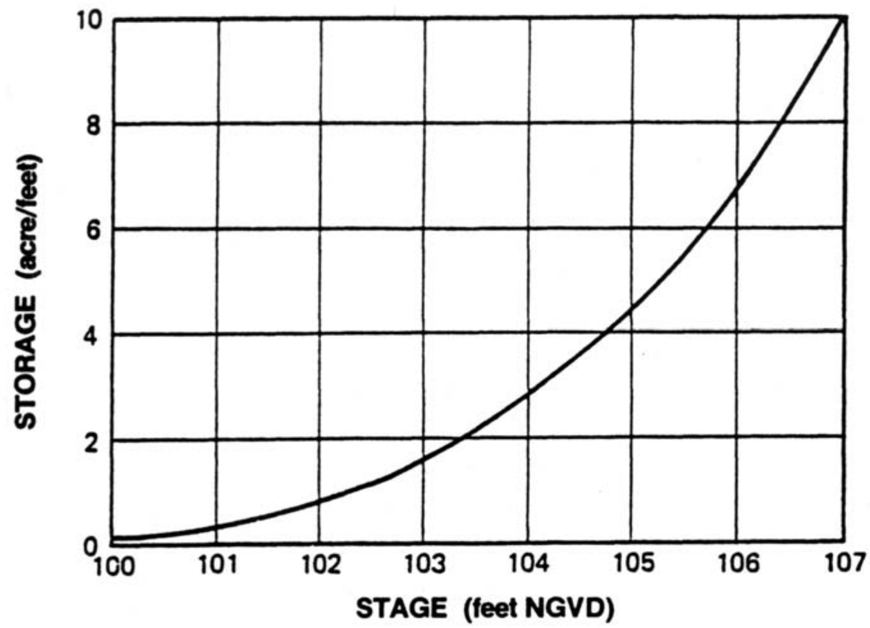

Figure 8-2
Example Stage-Storage Curve

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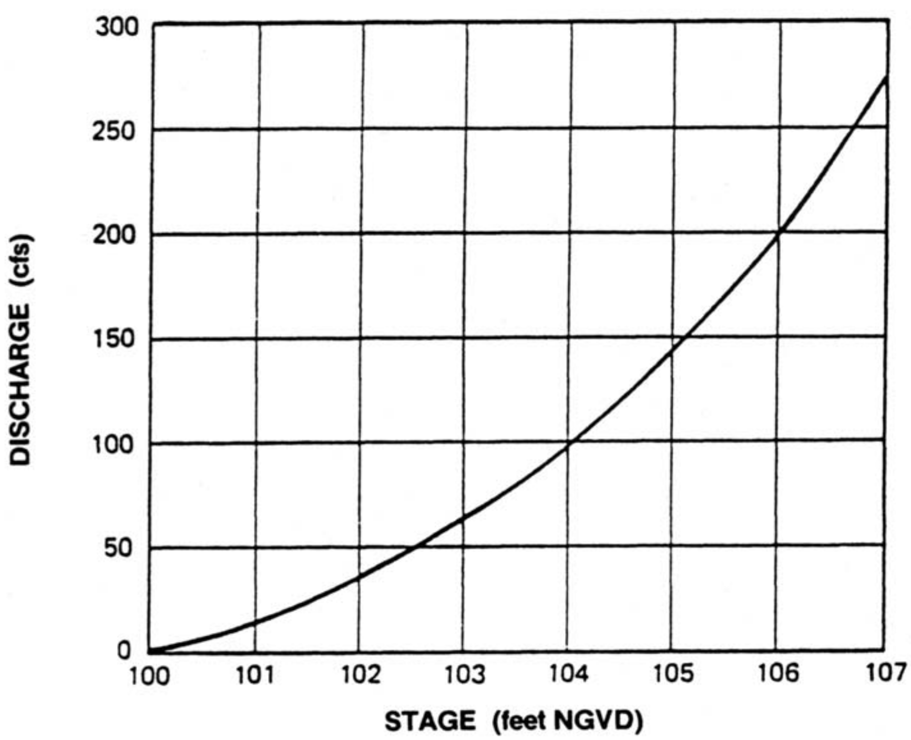

Figure 8-3
Example Stage-Discharge Curve

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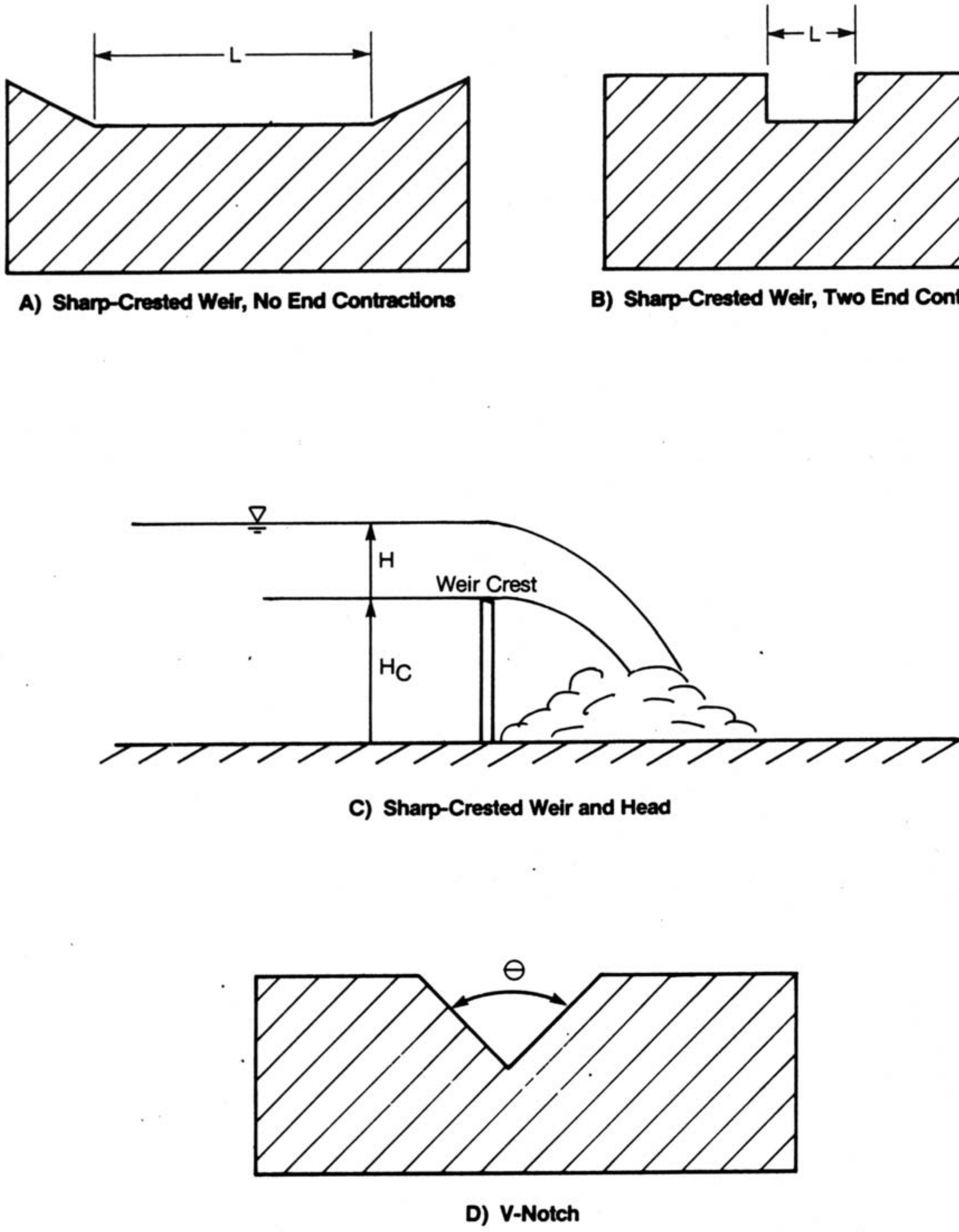

Figure 8-4
Illustrations of Weir Flow Control Structures

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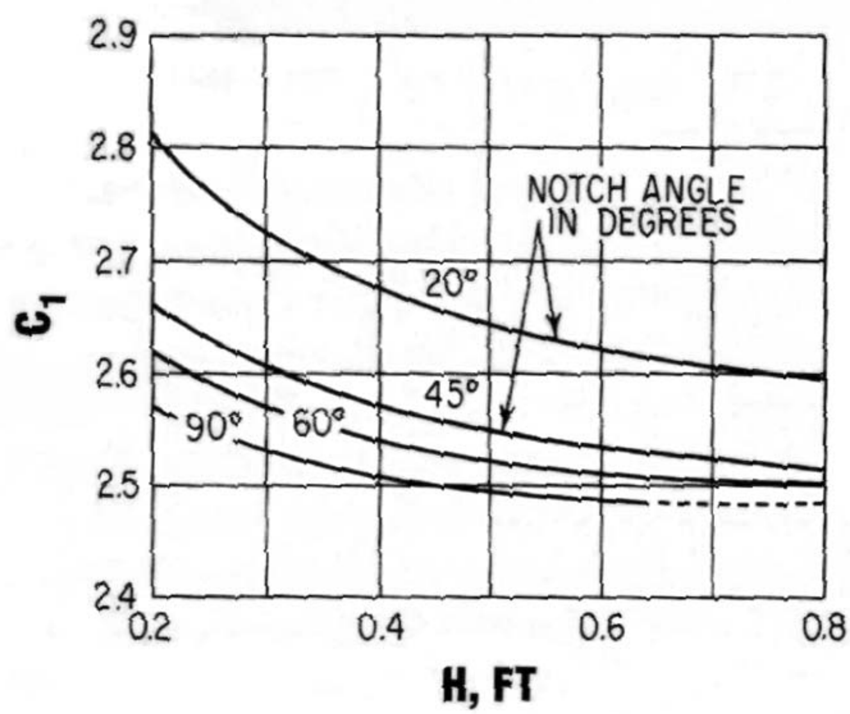

Figure 8-5
Sharp-Crested "V" Notch Weir Discharge Coefficients

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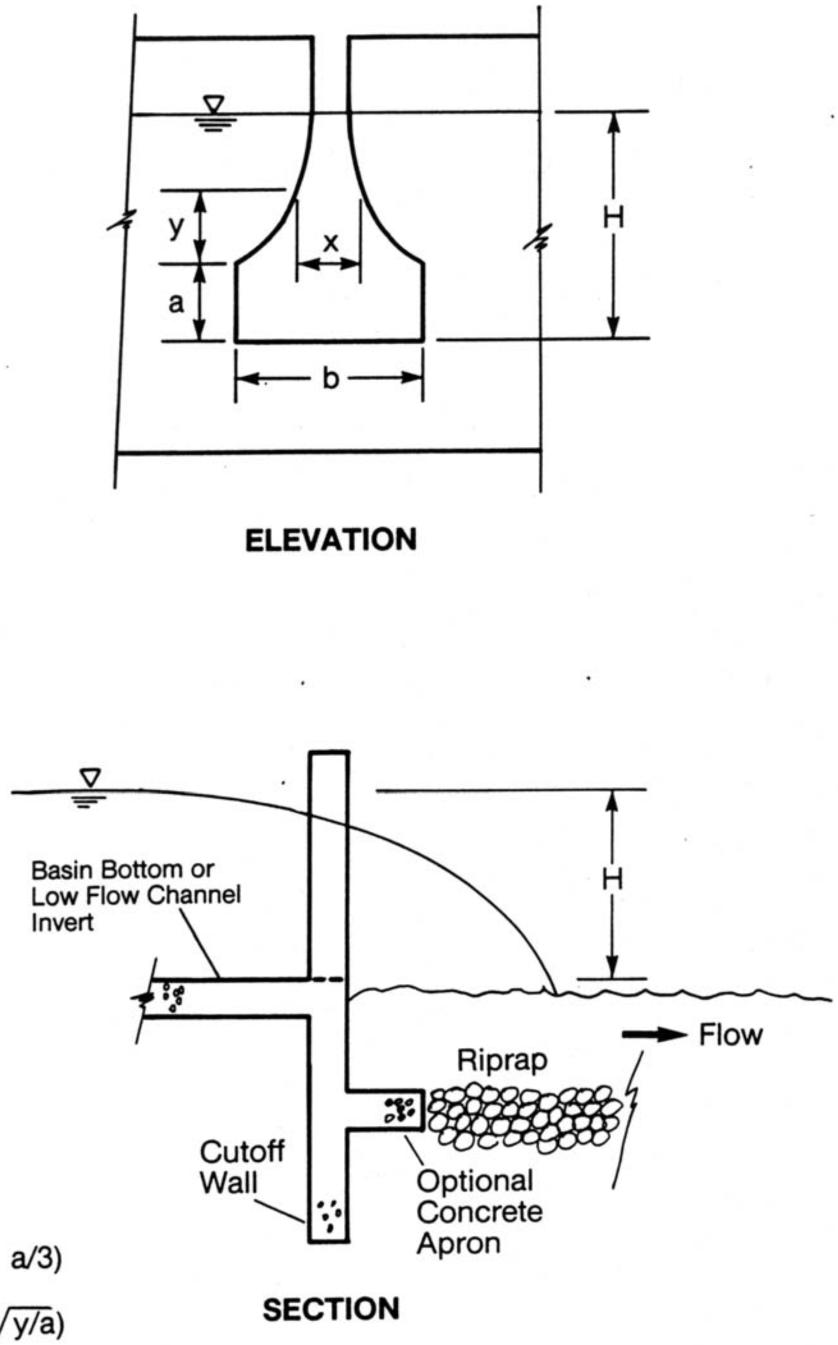

$$Q = 4.97\, a^{1/2} b\, (H - a/3)$$

$$x/b = 1 - 1/\pi\, (\arctan \sqrt{y/a})$$

Reference: Sandvik (1985).

Figure 8-6
Dimensions Used for Design of a Proportional Weir

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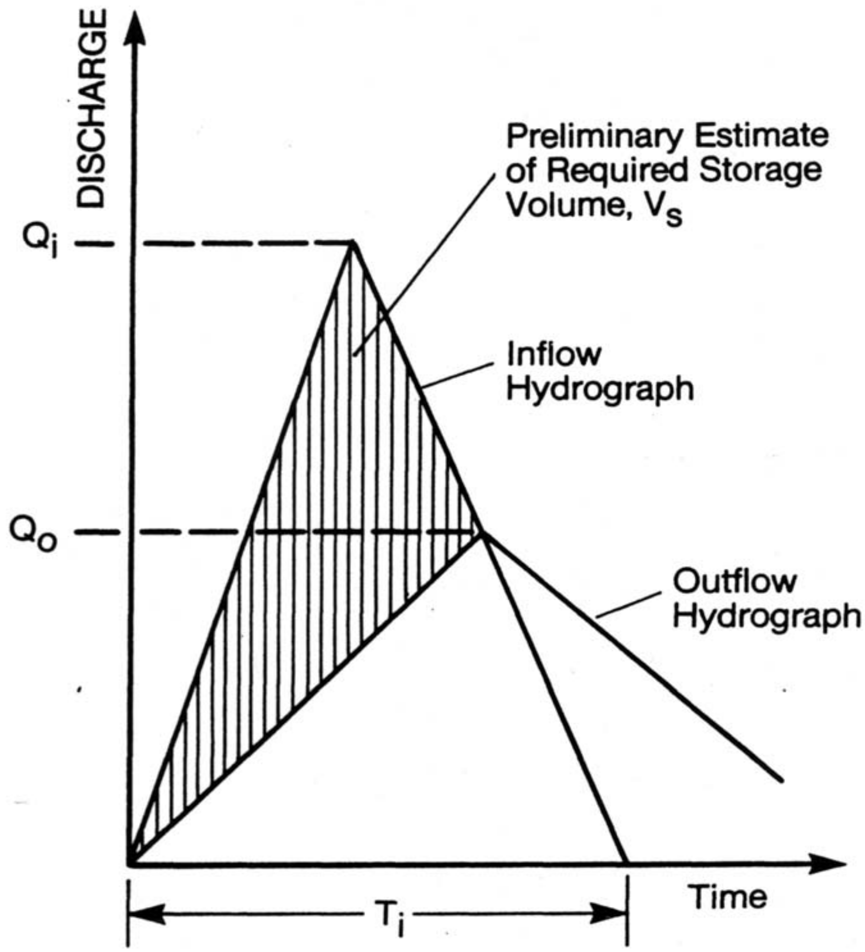

$$V_s = 0.5\, T_i\, (Q_i - Q_o)$$

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

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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

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Table 8-2
Minimum Freeboard Design Storms for Dams in Tennessee

Hazard Potential Category	Size	Freeboard Design Storm (6-hour)
Category 3	Small	100 yr
(Low)	Intermediate	PMP[a]
	Large	½ PMP
Category 2	Small	½ PMP
(Significant)	Intermediate	½ PMP
	Large	PMP
Category 1	Small	½ PMP
(High)	Intermediate	PMP
	Large	PMP

Note:
[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)

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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

Note:
[a] = Measured at least 2.5H upstream of the weir

Reference: Brater and King (1976)

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City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 9: Erosion Prevention and Sediment Control

Volume 3: Best Management Practices Manual

Revision: 0
June 2006

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Acronym List
(Chapter 9)

A	Sediment yield from a project site with erosion control, (tons/ac/yr)
A	Soil loss, (tons/ac), for the time period selected for R
BMPs	Best Management Practices
BMPM	Best Management Practices Manual
C	Cropping management factor, dimensionless
CP	Control practice factor
Co	Control due to any erosion control practice other than C_s and C_r
C_s	Control due to surface stabilization
C_r	Control due to runoff reduction practices
C_r	Runoff control factor, dimensionless
D_c	Sediment delivery ratio with controls
EI	Erosion index
EP & SC	Erosion control and sediment control procedures
i	Sequential number of segments
K	Soil erodibility factor, (tons/ac/R unit)
L	Horizontal slope length, (ft)
LS	Length-slope factor, dimensionless
LoD	Loring Silt Loam
m	Slope-length exponent
MeB	Memphis Silt Loam
n	Number of segments
N	Number of diversions placed across a uniform slope
NC	No live vegetation
NRCS	Natural Resources Conservation Service
P	Erosion control factor, dimensionless
R	Rainfall factor
RUSLE	Revised Universal Soil Loss Equation
S	Slope, percent
SWMM	Storm Water Management Manual
T	Soil loss tolerance, (tons/ac/yr)
USDA	United States Department of Agriculture
WC	75 percent cover of grass and weeds, having an average drop fall height of 20 inches

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9.0 EROSION PREVENTION AND SEDIMENT CONTROL

9.1 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. Erosion control measures and practices are actions that are taken to inhibit the dislodging and transporting of soil particles by water or wind, including actions that limit the area of exposed soil and minimize the time the soil is exposed. Specific control measures to mitigate adverse impacts are required by the regulations contained in Volume 3: Best Management Practices Manual (BMPM) of the Storm Water Management Manual (SWMM). To address regulatory requirements, an erosion and sediment control plan should be prepared according to Volume 3, Chapter 5. 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. Erosion and sediment control measures that will be implemented.

4. Removal of temporary measures, when appropriate, and establishment of permanent stabilization.

5. Post-construction runoff control measures that will be implemented.

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.

9.2 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 sediment barriers, 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 3 — Chapter 5.

9.3 Applying Best Management Practices

Guidance for applying control measures when developing an erosion and sediment control plan is presented in Volume 3: BMPM. Volume 3 — Chapter 2 provides additional detail about the erosion and sedimentation processes while Volume 3 — Chapter 4 discusses types of BMPs and how to select them. Key references for Volume 3 can be found in Volume 3 — Chapter 10.

The regulations presented in Volume 1 of the SWMM should be thoroughly reviewed and considered in selecting BMPs to present on the EP&SC plan.

9.4 Revised Universal Soil Loss Equation (RUSLE)

The RUSLE 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 RUSLE 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 RUSLE must be determined for each problem considered. Guiding principles and data for determining these parameters in Memphis and Shelby County are discussed in this section.

Computer programs designed to aid in estimating soil loss for a variety of site conditions are available for download through the United States Department of Agriculture's web site: http://www.ars.usda.gov/Research/docs.htm?docid=5971.

9.4.1 Rainfall Factor (R)

The average annual R value for the Memphis/Shelby County area is approximately 300 (USDA, AH 703). 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 300 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 272 for a 2-year return period to 536 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 RUSLE (Equation 9-1). For example, if the expected average soil loss for a 5-year design storm is desired, an R value of 70 is used in the RUSLE. Calculation of R values for individual gauged storm events is outlined in the USDA's AH 703.

Computer programs and associated databases designed to evaluate soil loss are available at United States Department of Agriculture's web site:

http://www.ars.usda.gov/Research/docs.htm?docid=5971

The databases provided include rainfall factor values (R) for Memphis and Shelby County.

9.4.2 Soil Erodibility Factor (K)

The soil erodibility factor (K) used in RUSLE is a quantification of the average long-term soil and soil-profile response to the erosive powers of rainstorms. Soil erodibility (K) factors are published in the NRCS soil survey report (1970) for Shelby County. Soil survey information can be found online at: http://websoilsurvey.nrcs.usda.gov/app/. Soil erodibility data published in the soil survey are presented in Table 9-3.

Background information on estimation of soil erodibility can be found in USDA's AH 703. Soil erodibility (K) can be estimated for soils with no direct K-value using the procedures outlined in AH 703. Soil erodibility factors (K) can also be found in USDA's computer program databases which can be downloaded at USDA's website as mentioned in Section 9.4.1.

9.4.3 Length-Slope Factor (LS)

The effect of topography on soil loss estimates using RUSLE is quantified in the equation through the use of the length-slope factor (LS), sometimes referred to as the topographic factor. The LS factor for slopes with fairly uniform surfaces can be estimated by the following procedure:

1. Identify uniform slope segments and estimate the slope length, in feet, and the slope steepness, in percent. Slope lengths are defined as the horizontal distance from the origin of overland flow to the point where either (1) the slope gradient decreases enough that deposition begins or (2) runoff becomes concentrated in a defined channel (Wischmeier and Smith, 1978). Surface runoff will usually concentrate in less than 300 ft, which is appropriate limit in most situations. Some typical RUSLE slopes are illustrated on Figure 9-2.

2. Estimate ratio of rill to interrill erosion and select the table with appropriate LS values. The following descriptions may aid in table selection:

   - Table 9-4: Low rill to interrill ratio — consolidated soil conditions, such as rangeland, with cover (applicable to thawing soil where both interrill and rill erosion are significant).

   - Table 9-5: Moderate rill to interrill ratio — moderately consolidated soil conditions, such as row-cropped agriculture, with little-to-moderate cover (not applicable to thawing soil).

   - Table 9-6: High rill to interrill ratio — freshly prepared construction and other highly disturbed soil conditions with little or no cover (not applicable to thawing soil).

   - Table 9-7: Thawing soils — most of the erosion is caused by surface flow.

3. Using slope length and slope steepness selected in step 1, find the appropriate LS value on the table selected in step 2.

If the actual slope is irregular, special considerations may be required. Typical concave and convex slopes are illustrated in Figure 9-3. These irregular slopes can be analyzed using Tables 9-4 through 9-7 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. Select the appropriate LS table based on slope conditions as described above.

3. Using the total slope length in the selected table, read the LS factor for each of the gradients listed in Step 1.

4. Select an appropriate slope-length exponent, m, from Table 9-8.

5. Multiply the LS factors from Step 3 by the appropriate factors from Table 9-9.

6. Add the products obtained from Step 5 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 five-step procedure described above can be used to estimate the LS factor for slope segments above and below the point of deposition.

Further explanation of the science and methodology behind length-slope factors can be found in AH 703. LS factors for various regions, including Memphis and Shelby County are included in USDA's RUSLE computer programs. These computer programs and associated databases can be found on USDA's web site: http://www.ars.usda.gov/Research/docs.htm?docid=5971

9.4.4 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

If no control practice is planned, a C value of 1.0 is used. 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 9.5. Tables 9-9 through 9-14 present C_s factors for various site conditions. C_r can be quantified using the expression (Chen, 1974):

$$C_r = \frac{1}{\sqrt{N+1}} \quad (9\text{-}3)$$

Where:

C_r = Runoff control factor, dimensionless

N = Number of diversions placed across a uniform slope

9.4.5 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 300 tons/acre/year and 95 percent control, the target soil loss for an erosion and sediment control plan is approximately 15 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 RUSLE, Equation 9-1.

2. Estimate the soil loss tolerance, T, using Table 9-3. The T factor is an estimate of the maximum average annual rate of soil erosion by wind and/or water that can occur without adversely affecting potential for vegetative growth. The rate is in tons per acre per year. It should be noted that it is an acceptable practice to set T = 0. In cases involving sites sensitive to sediment loss problems (impacted receiving waterways, discharge onto high-traffic roadways, and discharge to wetland areas are some examples), it is recommended that T = 0.

3. Estimate the sediment delivery ratio with controls, D_c, using the equation:

$$D_c = \frac{A - T}{20} \tag{9-4}$$

Where:

D_c = Sediment delivery ratio with controls

A = Sediment yield from a project site with erosion control, in tons/acre/year (calculated using RUSLE, Equation 9-1)

T = Soil loss tolerance, in tons/acre/year (Table 9-3)

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.

It is important to note that if sediment yield, A, is smaller than soil loss tolerance, T, the sediment delivery ratio, D_c, will be calculated to be negative. In this case, Equation 9-4 should be re-evaluated with T = 0.

9.4.6 Example Problem

Example 9-1. Plan Evaluation

A 35-acre site on both MeB (Memphis Silt Loam) and LoD (Loring Silt Loam) soils with a 200-foot long, 6 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 RUSLE to estimate soil loss for average annual conditions.

1. The average annual rainfall factor is given in Section 9.4.1 as R = 300.

2. The soil erodibility factor from Table 9-3 for both MeB (Memphis Silt Loam) and LoD (Loring Silt Loam) soils soils is K = 0.49.

3. The length-slope factor from Table 9-6 for a 200-foot, 6 percent slope is LS = 1.25.

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 9-10 for rough, irregular, tracked conditions is C_s = 0.90 and, since no no control practice is planned, C_r = 1.0 and C_o = 1.0. From Equation 9-2:

   $CP = C_s\, C_r\, C_o = (0.90)\,(1.0)\,(1.0) = 0.90$

5. The soil loss is estimated as:

   $A = (300)\,(0.49)\,(1.25)\,(0.90)$

   $A = 165.38$ tons/acre/year

6. The soil loss tolerance, T factor, for LoD soil is 4 tons/acre/year, and 5 tons/acre/year for MeB soil. The lower T factor will control, so for this example, T = 4 tons/acre/year.

7. The sediment delivery ratio is estimated using Equation 9-4:

   $$D_c = \frac{165.38 - 4}{20}$$

   $D_c = 8.07$

8. Based on estimated soil loss, with $D_c > 1$, the proposed activity is unacceptable.

9. Improve erosion control by constructing a diversion (N=1) 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.

10. The improved length-slope factor from Table 9-6 for a 100-foot, 6 percent slope is LS = 0.79.

11. The improved surface stabilization factor from Table 9-13 for straw or hay mulch applied at a rate of 1.5 tons/acre on a 6 percent slope is C_s = 0.12.

12. The runoff control factor from Equation 9-3 with one diversion across the slope (N=1) is $C_r = 0.707$.

$$C_r = \frac{1}{\sqrt{1+1}} = 0.707$$

13. The improved control practice factor from Equation 9-2 is computed as:

CP = (0.12) (0.707)

CP = 0.085

14. The improved soil loss is estimated as:

A = (300) (0.49) (0.79) (0.085)

A = 9.87 tons/acre/year

15. From Step 6, T = 4 tons/acre/year. However, because the soil loss estimate from Step 7 (165.38 tons/acre/year with no control practices) far exceeded the recommended 15 tons/acre/year (Section 9.4.5) and A > T, a conservative approach will be taken: T = 0 will be used for this iteration.

16. The improved sediment delivery ratio using Equation 9-4 is:

$$D_c = \frac{9.87 - 0}{20}$$

$D_c = 0.49$

17. Based on estimated soil loss with improvements, with $D_c \leq 1$, the proposed activity is acceptable.

9.5 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 3, Chapter 2 discusses the erosion process including water, stream and channel, wind erosion and factors that influence it. In addition, Volume 3 — Chapter 2 discusses selecting erosion prevention activities.

Following a brief description of temporary and permanent considerations, factors for use with the RUSLE 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 3: BMPM.

9.5.1 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 3 — Chapter 4.

9.5.2 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 9-10. For permanent pasture, rangeland, idle land, and grazed woodland, C_s factors can be estimated from values reported in Table 9-11. For undisturbed woodland, C_s factors can be estimated from values reported in Table 9-12.

Soil stabilization factors for mulches, seeding and vegetation, and chemical binders and tacks are discussed below.

Mulches

Table 9-13 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 3, ES-07.

Seeding and Vegetation

C_s factors for temporary and permanent seedings are presented in Table 9-14. 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 9-15. Suitable vegetative cover plants and plant mixtures are listed in Table 9-16 along with appropriate planting dates and application rates. Additional detail is provided in Volume 3, ES-08, ES-09, and ES-10.

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 9-14. Additional detail is provided in Volume 3 — Appendix ES and Chapter 4.

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 3, ES-20.

9.5.3 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 9-9 through 9-14.

3. Determine the fraction of the annual R value for each time period, using the EI factors from Figure 9-1 (see Section 9.4.1).

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 9-17. Since a construction schedule is subject to unplanned changes, a worst-case scenario should be considered.

9.5.4 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 RUSLE 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).

9.5.5 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, diversions (drains, swales, and berms), and bank stabilization. Appropriate construction specifications should be developed by the design engineer as guided by Volume 3 — Chapter 5.

9.5.6 Outlet Protection

Design procedures for outlet protection should be consistent with the erosion prevention information for open channels, energy dissipation methods, and additional information provided in Volume 3: BMPs. 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.

9.6 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 3 — Chapter 2.

9.6.1 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 3 — Chapter 4.

9.6.2 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 3, Chapters 2 and 5.

9.6.3 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 designers are strongly encouraged to use 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 3 — ES-18 and ES-19. Information on permanent detention facilities is provided in Chapter 8 of this manual.

9.6.4 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 3 — ES-01 and ES-03.

9.7 Chapter Equations

$$A = R\ K\ LS\ C\ P \tag{9-1}$$

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

$$CP = C_s\ C_r\ C_o \tag{9-2}$$

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

$$C_r = \frac{1}{\sqrt{N+1}} \tag{9-3}$$

Where:

C_r = Runoff control factor, dimensionless
N = Number of diversions placed across a uniform slope

$$D_c = \frac{A-T}{20} \tag{9-4}$$

Where:

D_c = Sediment delivery ratio with controls
A = Sediment yield from a project site with erosion control, in tons/acre/year (calculated using RUSLE, Equation 9-1)
T = Soil loss tolerance, in tons/acre/year (Table 9-3)

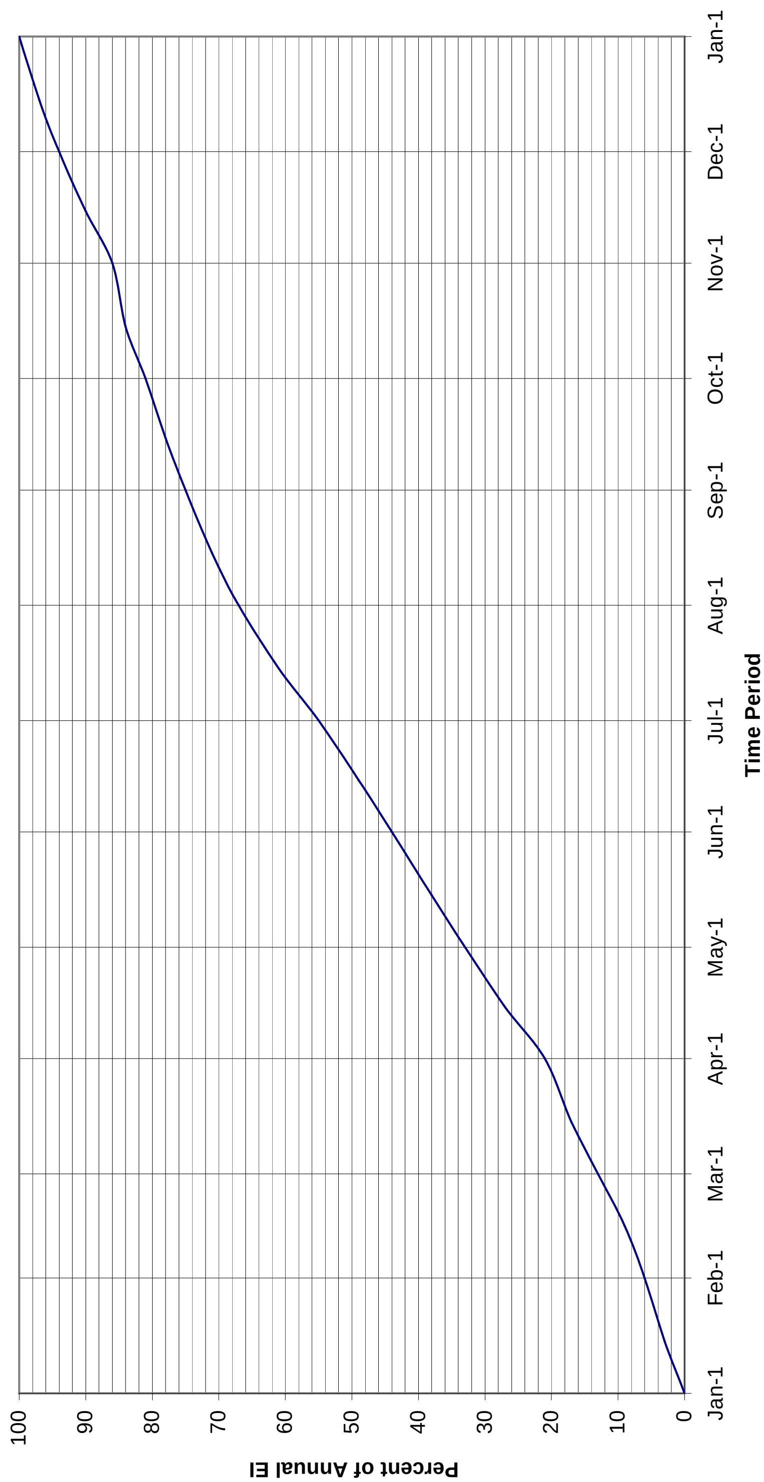

Figure 9-1
Erosion Index (EI) Distribution Curve
Applicable to Memphis and Shelby County

Back to Section 9.4.1, 9.5.3

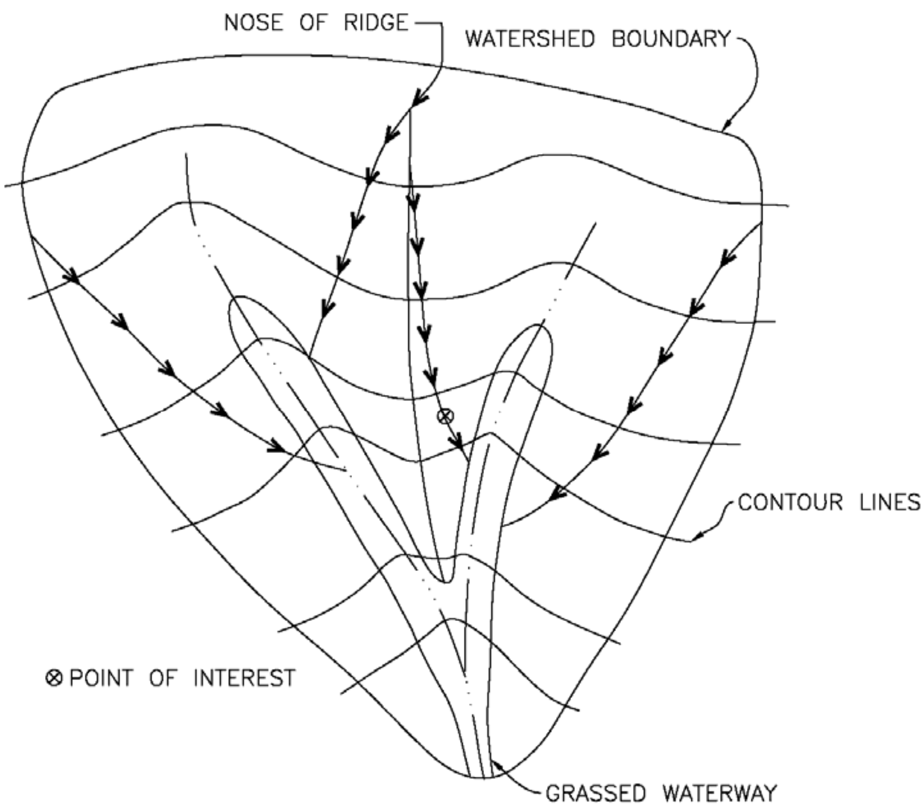

Figure 9-2
Illustration of Typical RUSLE Slope Lengths

Back to Section 9.4.3

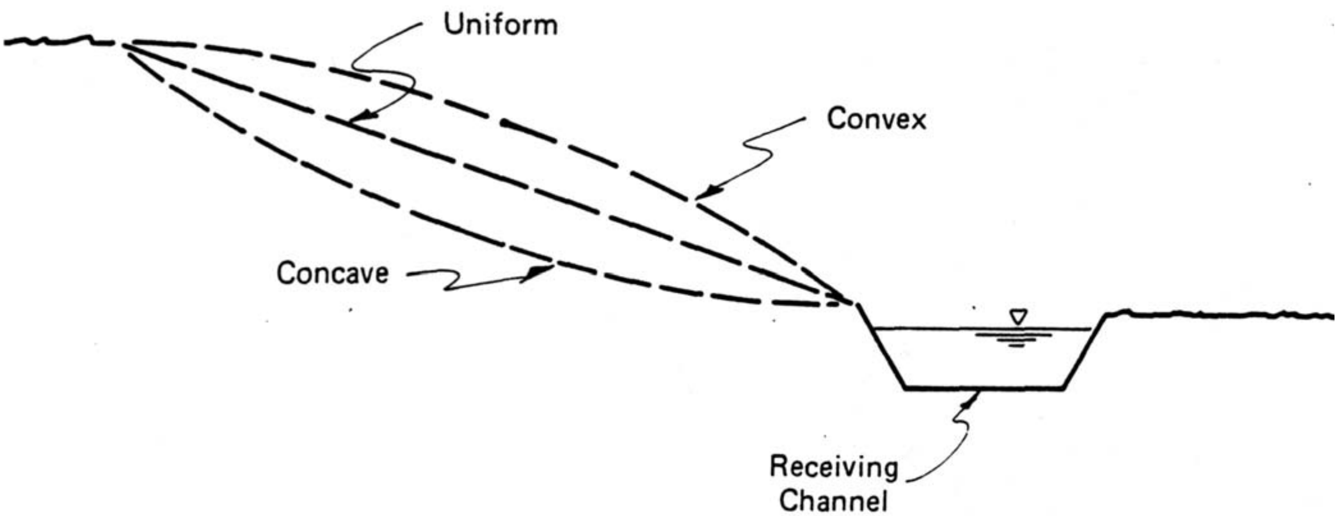

Figure 9-3
Conceptual Sketch of Typical Concave and Convex Slopes

Back to Section 9.4.3

Table 9-1
Annual Rainfall Factor (R) Values for 2-, 5-, and 20-Year Return Periods for Memphis and Shelby County

Observed R Value Annual Range	Annual Rainfall Factor (R) Value for Various Return Period Frequencies		
(22 Years)	**2-Year**	**5-Year**	**20-Year**
139-595	272	384	536

Reference: Wischmeier and Smith (1978)

Back to Section 9.4.1

Table 9-2
Expected Single Storm Rainfall Factor (R) Values for Memphis and Shelby County

Return Period Frequencies				
1-Year	**2-Year**	**5-Year**	**10-Year**	**20-Year**
43	55	70	82	91

Reference: Wischmeier and Smith (1978)

Back to Section 9.4.1

Table 9-3
RUSLE Related Attributes
Shelby County, Tennessee

Map symbol and soil name	% of Map Unit	Hydrologic group	K	T factor	Representative Value % Sand	% Silt	% Clay
Ad:							
Adler	100	C	0.43	5	13.6	68.9	17.5
Bo:							
Bonn	100	D	0.49	3	20.8	67.7	11.5
Bw:							
Bowdre	100	C	0.37	5	5.5	47	47.5
Ca:							
Calloway	100	C	0.49	4	1.8	78.2	20
Co:							
Collins	100	C	0.43	5	5	82.5	12.5
Cr:							
Commerce	100	C	0.43	5	11.4	68.1	20.5
Cs:							
Convent	100	C	0.43	5	21.4	69.6	9
Cu:							
Crevasse	100	A	0.1	5	94.4	0.6	5
Cv:							
Bruno, overwash	100	A	0.28	5	15.7	59.3	25
Fm:							
Falaya	91	D	0.49	5	20.7	67.3	12
Waverly	9	D	0.43	5	6.2	81.8	12
Fs:							
Udorthents, silty	100	B	0.49	5	14.3	72.7	13
Fy:							
Udorthents, loamy	100	B	0.2	5	43.5	40	16.5
GaA:							
Grenada	100	C	0.49	4	3	83	14
GaB:							
Grenada	100	C	0.49	4	3	83	14
GaB2:							
Grenada	100	C	0.49	4	3	83	14
GaC:							
Grenada	100	C	0.49	4	3	83	14
GaC3:							
Grenada	100	C	0.49	3	3	83	14
GaD:							
Grenada	100	C	0.49	4	3	83	14
GaD2:							

Table 9-3
RUSLE Related Attributes
Shelby County, Tennessee

Map symbol and soil name	% of Map Unit	Hydrologic group	K	T factor	Representative Value % Sand	% Silt	% Clay
Grenada	100	C	0.49	4	3	83	14
GgD3:							
Grenada	100	C	0.49	3	3	83	14
Gr:							
Udorthents, silty	100	B	0.49	5	14.3	72.7	13
Gs:							
Gullied land	100	B	—	5	—	—	—
He:							
Henry	100	D	0.49	4	5	81.5	13.5
Ib:							
Iberia	100	D	0.37	5	17.6	49.4	33
Lb:							
Udorthents, silty	100	—	—	—	—	—	—
LoB:							
Loring	100	C	0.49	4	3	84	13
LoB2:							
Loring	100	C	0.49	4	3	84	13
LoC2:							
Loring	100	C	0.49	4	3	84	13
LoD:							
Loring	100	C	0.49	4	3	84	13
LoD2:							
Loring	100	C	0.49	4	3	84	13
LoD3:							
Loring	100	C	0.49	3	3	84	13
MeB:							
Memphis	100	B	0.49	5	3.5	81.5	15
MeB2:							
Memphis	100	B	0.49	5	3.5	81.5	15
MeC2:							
Memphis	100	B	0.49	5	3.5	81.5	15
MeD2:							
Memphis	100	B	0.49	5	3.5	81.5	15
MeD3:							
Memphis	100	B	0.49	5	3.5	81.5	15
MeE:							
Memphis	100	B	0.49	5	3.5	81.5	15
MeF3:							

Table 9-3
RUSLE Related Attributes
Shelby County, Tennessee

Map symbol and soil name	% of Map Unit	Hydrologic group	K	T factor	Representative Value		
					% Sand	% Silt	% Clay
Memphis	100	B	0.49	5	3.5	81.5	15
MeG:							
Memphis	100	B	0.49	5	3.5	81.5	15
MP:							
Gravel pits	50	—	—	—	—	—	—
Mines	50	—	—	—	—	—	—
Rb:							
Robinsonville	100	B	0.28	5	63.5	30.5	6
Rn:							
Robinsonville	100	B	0.28	5	63.5	30.5	6
Sh:							
Sharkey	100	D	0.32	5	11.8	38.2	50
Sw:							
Sharkey, ponded	100	D	0.32	5	11.8	38.2	50
Tu:							
Tunica	100	D	0.32	5	3	42	55
W:							
WATER	100	—	—	—	—	—	—
Wv:							
Waverly	100	D	0.43	5	20.7	67.3	12

Back to Section 9.4.2, 9.4.5, 9.4.6

Table 9-4
Values for Topographic Factor (LS) for Low Ratios of Rill to Interrill Erosion[1]

Horizontal Slope Length, L (ft)

Slope, S (%)	<3	6	9	12	15	25	50	75	100	150	200	250	300	400	600	800	1,000
0.2	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
0.5	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
1.0	0.12	0.12	0.12	0.12	0.12	0.13	0.13	0.14	0.14	0.15	0.15	0.15	0.15	0.16	0.16	0.17	0.17
2.0	0.20	0.20	0.20	0.20	0.20	0.21	0.23	0.25	0.26	0.27	0.28	0.29	0.30	0.31	0.33	0.34	0.35
3.0	0.26	0.26	0.26	0.26	0.26	0.29	0.33	0.36	0.38	0.40	0.43	0.44	0.46	0.48	0.52	0.55	0.57
4.0	0.33	0.33	0.33	0.33	0.33	0.36	0.43	0.46	0.50	0.54	0.58	0.61	0.63	0.67	0.74	0.78	0.82
5.0	0.38	0.38	0.38	0.38	0.38	0.44	0.52	0.57	0.62	0.68	0.73	0.78	0.81	0.87	0.97	1.04	1.10
6.0	0.44	0.44	0.44	0.44	0.44	0.50	0.61	0.68	0.74	0.83	0.90	0.95	1.00	1.08	1.21	1.31	1.40
8.0	0.54	0.54	0.54	0.54	0.54	0.64	0.79	0.90	0.99	1.12	1.23	1.32	1.40	1.53	1.74	1.91	2.05
10.0	0.60	0.63	0.65	0.66	0.68	0.81	1.03	1.19	1.31	1.51	1.67	1.80	1.92	2.13	2.45	2.71	2.93
12.0	0.61	0.70	0.75	0.80	0.83	1.01	1.31	1.52	1.69	1.97	2.20	2.39	2.56	2.85	3.32	3.70	4.02
14.0	0.63	0.76	0.85	0.92	0.98	1.20	1.58	1.85	2.08	2.44	2.73	2.99	3.21	3.60	4.23	4.74	5.18
16.0	0.65	0.82	0.94	1.04	1.12	1.38	1.85	2.18	2.46	2.91	3.28	3.60	3.88	4.37	5.17	5.82	6.39
20.0	0.68	0.93	1.11	1.26	1.39	1.74	2.37	2.84	3.22	3.85	4.38	4.83	5.24	5.95	7.13	8.10	8.94
25.0	0.73	1.05	1.30	1.51	1.70	2.17	3.00	3.63	4.16	5.03	5.76	6.39	6.96	7.97	9.65	11.04	12.26
30.0	0.77	1.16	1.48	1.75	2.00	2.57	3.60	4.40	5.06	6.18	7.11	7.94	8.68	9.99	12.19	14.04	15.66
40.0	0.85	1.36	1.79	2.17	2.53	3.30	4.73	5.84	6.78	8.37	9.71	10.91	11.99	13.92	17.19	19.96	22.41
50.0	0.91	1.52	2.06	2.54	3.00	3.95	5.74	7.14	8.33	10.37	12.11	13.65	15.06	17.59	21.88	25.55	28.82
60.0	0.97	1.67	2.29	2.86	3.41	4.52	6.63	8.29	9.72	12.16	14.26	16.13	17.84	20.92	26.17	30.68	34.71

Note:
[1] = Such as for rangeland and other consolidated soil conditions with cover (applicable to thawing soil where both interrill and rill erosion are significant)

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Table 9-5
Values for Topographic Factor (LS) for Moderate Ratios of Rill to Interrill Erosion[1]

Slope, S (%)	Horizontal Slope Length, L (ft)															
	<3	**6**	**9**	**12**	**15**	**25**	**50**	**75**	**100**	**150**	**200**	**250**	**300**	**400**	**600**	**800**	**1,000**
0.2	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.60	0.60	0.60
0.5	0.70	0.70	0.70	0.70	0.70	0.80	0.80	0.80	0.90	0.90	0.90	0.90	0.90	0.10	0.10	0.10	0.10
1.0	0.11	0.11	0.11	0.11	0.11	0.12	0.13	0.14	0.14	0.15	0.16	0.17	0.17	0.18	0.19	0.20	0.20
2.0	0.17	0.17	0.17	0.17	0.17	0.19	0.22	0.25	0.27	0.29	0.31	0.33	0.35	0.37	0.41	0.44	0.47
3.0	0.22	0.22	0.22	0.22	0.22	0.25	0.32	0.36	0.39	0.44	0.48	0.52	0.55	0.60	0.68	0.75	0.80
4.0	0.26	0.26	0.26	0.26	0.26	0.31	0.40	0.47	0.52	0.60	0.67	0.72	0.77	0.86	0.99	1.10	1.19
5.0	0.30	0.30	0.30	0.30	0.30	0.37	0.49	0.58	0.65	0.76	0.85	0.93	1.01	1.13	1.33	1.49	1.63
6.0	0.34	0.34	0.34	0.34	0.34	0.43	0.58	0.69	0.78	0.93	1.05	1.16	1.25	1.42	1.69	1.91	2.11
8.0	0.42	0.42	0.42	0.42	0.42	0.53	0.74	0.91	1.04	1.26	1.45	1.62	1.77	2.03	2.47	2.83	3.15
10.0	0.46	0.48	0.50	0.51	0.52	0.67	0.97	1.19	1.38	1.71	1.98	2.22	2.44	2.84	3.50	4.06	4.56
12.0	0.47	0.53	0.58	0.61	0.64	0.84	1.23	1.53	1.79	2.23	2.61	2.95	3.26	3.81	4.75	5.56	6.28
14.0	0.48	0.58	0.65	0.70	0.75	1.00	1.48	1.86	2.19	2.76	3.25	3.69	4.09	4.82	6.07	7.15	8.11
16.0	0.49	0.63	0.72	0.79	0.85	1.15	1.73	2.20	2.60	3.30	3.90	4.45	4.95	5.86	7.43	8.79	10.02
20.0	0.52	0.71	0.85	0.96	1.06	1.45	2.22	2.85	3.40	4.36	5.21	5.97	6.68	7.97	10.23	12.20	13.99
25.0	0.56	0.80	1.00	1.16	1.30	1.81	2.82	3.65	4.39	5.69	6.83	7.88	8.86	10.65	13.80	16.58	19.13
30.0	0.59	0.89	1.13	1.34	1.53	2.15	3.39	4.42	5.34	6.98	8.43	9.76	11.01	13.30	17.37	20.99	24.31
40.0	0.65	1.05	1.38	1.68	1.95	2.77	4.45	5.87	7.14	9.43	11.47	13.37	15.14	18.43	24.32	29.60	34.48
50.0	0.71	1.18	1.59	1.97	2.32	3.32	5.40	7.17	8.78	11.66	14.26	16.67	18.94	23.17	30.78	37.65	44.02
60.0	0.76	1.30	1.78	2.23	2.65	3.81	6.24	8.33	10.23	13.65	16.76	19.64	22.36	27.45	36.63	44.96	52.70

Note:
[1] = Such as for row-cropped agriculture and other moderately consolidated soil conditions with little-to-moderate cover (not applicable to thawing soil)

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Table 9-6
Values for Topographic Factor (LS) for High Ratios of Rill to Interrill Erosion[1]

Slope, S (%)	Horizontal Slope Length, L (ft)																
	<3	**6**	**9**	**12**	**15**	**25**	**50**	**75**	**100**	**150**	**200**	**250**	**300**	**400**	**600**	**800**	**1,000**
0.2	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.06
0.5	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.09	0.09	0.10	0.10	0.10	0.11	0.12	0.12	0.13
1.0	0.09	0.09	0.09	0.09	0.09	0.10	0.13	0.14	0.15	0.17	0.18	0.19	0.20	0.22	0.24	0.26	0.27
2.0	0.13	0.13	0.13	0.13	0.13	0.16	0.21	0.25	0.28	0.33	0.37	0.40	0.43	0.48	0.56	0.63	0.69
3.0	0.17	0.17	0.17	0.17	0.17	0.21	0.30	0.36	0.41	0.50	0.57	0.64	0.69	0.80	0.96	1.10	1.23
4.0	0.20	0.20	0.20	0.20	0.20	0.26	0.38	0.47	0.55	0.68	0.79	0.89	0.98	1.14	1.42	1.65	1.86
5.0	0.23	0.23	0.23	0.23	0.23	0.31	0.46	0.58	0.68	0.86	1.02	1.16	1.28	1.51	1.91	2.25	2.55
6.0	0.26	0.26	0.26	0.26	0.26	0.36	0.54	0.69	0.82	1.05	1.25	1.43	1.60	1.90	2.43	2.89	3.30
8.0	0.32	0.32	0.32	0.32	0.32	0.45	0.70	0.91	1.10	1.43	1.72	1.99	2.24	2.70	3.52	4.24	4.91
10.0	0.35	0.37	0.38	0.39	0.40	0.57	0.91	1.20	1.46	1.92	2.34	2.72	3.09	3.75	4.95	6.03	7.02
12.0	0.36	0.41	0.45	0.47	0.49	0.71	1.15	1.54	1.88	2.51	3.07	3.60	4.09	5.01	6.67	8.17	9.57
14.0	0.38	0.45	0.51	0.55	0.58	0.85	1.40	1.87	2.31	3.09	3.81	4.48	5.11	6.30	8.45	10.40	12.23
16.0	0.39	0.49	0.56	0.62	0.67	0.98	1.64	2.21	2.73	3.68	4.56	5.37	6.15	7.60	10.26	12.69	14.96
20.0	0.41	0.56	0.67	0.76	0.84	1.24	2.10	2.86	3.57	4.85	6.04	7.16	8.23	10.24	13.94	17.35	20.57
25.0	0.45	0.64	0.80	0.93	1.04	1.58	2.67	3.67	4.59	6.30	7.88	9.38	10.81	13.53	18.57	23.24	27.66
30.0	0.48	0.72	0.91	1.08	1.24	1.86	3.22	4.44	5.58	7.70	9.67	11.55	13.35	16.77	23.14	29.07	34.71
40.0	0.53	0.85	1.13	1.37	1.59	2.41	4.24	5.89	7.44	10.35	13.07	15.67	18.17	22.95	31.89	40.29	48.29
50.0	0.58	0.97	1.31	1.62	1.91	2.91	5.16	7.20	9.13	12.75	16.16	19.42	22.57	28.60	39.95	50.63	60.84
60.0	0.63	1.07	1.47	1.84	2.19	3.36	5.97	8.37	10.63	14.89	18.92	22.78	26.51	33.67	47.18	59.93	72.15

Note:
[1] = Such as for freshly prepared construction and other highly disturbed soil conditions with little or no cover (not applicable to thawing soil)

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Table 9-7
Values for Topographic Factor (LS) for Thawing Soils Where Most of the Erosion is Caused by Surface Flow

Slope, S (%)	Horizontal Slope Length, L (ft)												
	15	**25**	**50**	**75**	**100**	**150**	**200**	**250**	**300**	**400**	**600**	**800**	**1,000**
0.2	0.02	0.03	0.04	0.05	0.06	0.07	0.09	0.10	0.10	0.12	0.15	0.17	0.19
0.5	0.04	0.05	0.07	0.09	0.10	0.12	0.14	0.16	0.17	0.20	0.24	0.28	0.31
1.0	0.60	0.08	0.11	0.14	0.16	0.20	0.23	0.26	0.28	0.32	0.40	0.46	0.51
2.0	0.11	0.14	0.20	0.25	0.29	0.35	0.41	0.46	0.50	0.58	0.71	0.82	0.91
3.0	0.16	0.21	0.29	0.36	0.42	0.51	0.59	0.66	0.72	0.83	1.02	1.17	1.31
4.0	0.21	0.27	0.38	0.47	0.54	0.66	0.77	0.86	0.94	1.08	1.33	1.53	1.71
5.0	0.26	0.33	0.47	0.58	0.67	0.82	0.94	1.06	1.16	1.34	1.64	1.89	2.11
6.0	0.31	0.40	0.56	0.69	0.79	0.97	1.12	1.26	1.38	1.59	1.95	2.25	2.51
8.0	0.41	0.52	0.74	0.91	1.05	1.28	1.48	1.65	1.81	2.09	2.56	2.96	3.31
10.0	0.48	0.62	0.88	1.08	1.25	1.53	1.77	1.98	2.16	2.50	3.06	3.54	3.95
12.0	0.54	0.70	0.98	1.21	1.39	1.71	1.97	2.20	2.41	2.78	3.41	3.94	4.40
14.0	0.59	0.76	1.08	1.32	1.53	1.87	2.16	2.41	2.64	3.05	3.74	4.31	4.82
16.0	0.64	0.82	1.17	1.43	1.65	2.02	2.33	2.61	2.86	3.30	4.04	4.67	5.22
20.0	0.73	0.94	1.33	1.63	1.88	2.30	2.66	2.97	3.25	3.76	4.60	5.31	5.94
25.0	0.83	1.07	1.51	1.85	2.13	2.61	3.02	3.37	3.69	4.27	5.23	6.03	6.75
30.0	0.91	1.18	1.67	2.05	2.36	2.89	3.34	3.73	4.09	4.72	5.78	6.68	7.47
40.0	1.07	1.38	1.95	2.39	2.75	3.37	3.90	4.36	4.77	5.51	6.75	7.79	8.71
50.0	1.19	1.54	2.18	2.67	3.08	3.77	4.35	4.87	5.33	6.16	7.54	8.71	9.74
60.0	1.30	1.67	2.37	2.90	3.35	4.10	4.74	5.30	5.80	6.70	8.20	9.47	10.59

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Table 9-8
Slope-Length Exponents (m) For a Range of Slopes and Rill / Interrill Erosion Classes[1]

	Rill / Interrill Ratio		
Slope (%)	**Low**	**Moderate**	**High**
0.2	0.02	0.04	0.07
0.5	0.04	0.08	0.16
1.0	0.08	0.15	0.26
2.0	0.14	0.24	0.39
3.0	0.18	0.31	0.47
4.0	0.22	0.36	0.53
5.0	0.25	0.40	0.57
6.0	0.28	0.43	0.60
8.0	0.32	0.48	0.65
10.0	0.35	0.52	0.68
12.0	0.37	0.55	0.71
14.0	0.40	0.57	0.72
16.0	0.41	0.59	0.74
20.0	0.44	0.61	0.76
25.0	0.47	0.64	0.78
30.0	0.49	0.66	0.79
40.0	0.52	0.68	0.81
50.0	0.54	0.70	0.82
60.0	0.55	0.71	0.83

Note:
[1] = Not applicable to thawing soils

Source: McCool et al. (1989)

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Table 9-9
Soil Loss Factor to Estimate Soil Loss on a Segment of a Uniform Slope

Number of Segments	Sequential Number of Segments	Slope-Length Exponent (m) 0.05	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
2	1	0.97	0.93	0.87	0.81	0.76	0.71	0.66	0.62	0.57
	2	1.03	1.07	1.13	1.19	1.24	1.29	1.34	1.38	1.43
3	1	0.95	0.90	0.80	0.72	0.64	0.58	0.52	0.46	0.42
	2	1.01	1.02	1.04	1.05	1.06	1.05	1.05	1.04	1.03
	3	1.04	1.08	1.16	1.23	1.30	1.37	1.43	1.50	1.55
4	1	0.93	0.87	0.76	0.66	0.57	0.50	0.44	0.38	0.33
	2	1.00	1.00	0.98	0.96	0.94	0.92	0.88	0.85	0.82
	3	1.03	1.05	1.09	1.13	1.16	1.18	1.20	1.22	1.23
	4	1.04	1.08	1.17	1.25	1.33	1.40	1.48	1.55	1.62
5	1	0.92	0.85	0.73	0.62	0.53	0.45	0.38	0.32	0.28
	2	0.99	0.97	0.94	0.90	0.86	0.82	0.77	0.73	0.69
	3	1.01	1.03	1.04	1.05	1.06	1.06	1.06	1.05	1.03
	4	1.03	1.06	1.12	1.17	1.21	1.29	1.29	1.32	1.35
	5	1.05	1.09	1.17	1.26	1.34	1.50	1.50	1.58	1.65

Notes:
Soil loss factors $= [i^{m+1} - (i - 1)^{m+1}]/n^m$

Where:

i	=	Sequential number of segments
m	=	Slope length exponent
n	=	Number of segments

Values are forced to give a factor total equal to the number of segments
Values from RUSLE computer program may differ slightly due to rounding

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Table 9-10
Surface Stabilization (C) 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)

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Table 9-11
Surface Stabilization (C_s) Factors for Permanent Pasture, Rangeland, Idle Land, and Grazed Woodland[a]

Vegetable Canopy			***Cover That Contacts the Surface***					
Type and Height of Raised Canopy[b]	**Canopy Cover[c] %**	**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	25	G	.36	.17	.09	.038	.013	.033
or short brush		W	.36	.20	.13	.083	.041	.011
(20-inch fall height)	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	25	G	.40	.18	.09	.040	.013	.003
(6.5-ft fall height)		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	25	G	.42	.19	.10	.041	.013	.003
Appreciable low brush		W	.42	.23	.14	.089	.042	.011
(13-ft fall height)	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

Notes:

[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)

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Table 9-12
Surface Stabilization (C_s) Factors for Undisturbed Woodlands

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

Notes:

[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 9-11. Where woodlands are being harvested, grazed, or burned, also use Table 9-11.

[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)

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Table 9-13
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

Notes:

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)

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Table 9-14
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

Notes:
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

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Table 9-15
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:

[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 = olive 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	.95

[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 9-11. For sites more than 8 years old, use Table 9-12.

[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 9-11. For sites treated more than 8 years ago, use Table 9-12.

Reference: Wischmeier and Smith (1978)

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Table 9-16
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

Notes:

[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)

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Table 9-17
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	0.22	0.22
6/1 — 8/1	Temporary Seeding	0.40	0.31	0.124
8/1 — 12/31	Permanent Seeding	0.05	0.25	0.013

Notes:
Composite C_s for exposure scheduling is the sum of each weighted C_s factor or 0.358
[a] = Obtained from Figure 9-1
[b] = Product of the C_s factor and the fraction of annual R during the specified time period

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City of Memphis/Shelby County

STORM WATER MANAGEMENT MANUAL

City of Memphis Division of Public Works and Division of Engineering
Shelby County Public Works Department

Volume 1: Regulations

Volume 2: Drainage Manual

Chapter 10: Outlet Protection

Volume 3: Best Management Practices Manual

Revision: 0
June 2006

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Acronym List
(Chapter 10)

a	1/2 (W), (ft) (for baffled outlet)
A	Flow area, (ft^2)
b	3/8 (W), (ft) (for baffled outlet)
B	Width of barrel, (ft) (for box culvert)
c	1/2 (W), (ft) (for baffled outlet)
C_1	TW/y_2, dimensionless
d_{50}	Median riprap diameter
d	Depth of flow in a pipe, (ft)
d	Representative flow depth entering a basin, (ft) (assumes square jet)
d_{max}	1.5 times the median stone size, (ft)
d_o	Initial depth
d_W	Conduit width, (ft)
D	Diameter, (in) (for circular culvert)
D	Height of box culvert, (ft)
e	1/12 (W), (ft) (for baffled outlet)
f	1/6 (W), (ft) (for baffled outlet)
FHWA	Federal Highway Administration
Fr	Froude Number, dimensionless
Fr_o	Outlet Froude Number
Fr_1	Froude Number at Section 1 (see Figure 10-13), dimensionless
g	Acceleration due to gravity, (32.2 ft/sec^2)
h	Energy head to be dissipated, (ft)
h_s	Scourhole depth
H	3/4 (W), (ft) (for baffled outlet)
L	Total length of supercritical flow expansion, (ft)
L_a	Apron Length, (ft)
L_s	Scourhole length
N_c	Number of chute blocks
N_s	Number of blocks at jump location
PESC	Post-Construction Erosion Prevention and Sediment Control
PI	Plasticity index from Atterburg limits
Q	Design discharge, (cfs)

SCS	Soil Conservation Service
S_0	Initial basin slope, dimensionless
S_S	Slope of chute, dimensionless
S_T	Outgoing slope of basin, dimensionless
S_v	Saturated shear strength, (lbs/in^2)
t	Time of scour, (min)
τ_c	Critical tractive shear stress, (lbs/in^2)
TCP	Temporary Construction Site Runoff Management
TW	Tailwater
USBR	United States Bureau of Reclamations
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
v_0	Outlet velocity, (ft/sec)
v	velocity, (ft/sec)
v_L	Velocity L feet downstream from brink, (ft/sec)
V_s	Scourhole volume
W	Minimum basin width, (ft)
W_B	Width of expanding culvert, (ft)
W_o	Conduit width, (ft)
W_s	Scourhole width, (ft)
y_0	Water elevation at Section 0 (see Figure 10-13), (ft)
y_1	Water elevation at Section 1 (see Figure 10-13), (ft)
y_2	Jump height, (ft)
y_e	Equivalent brink depth
z_0	Basin elevation at beginning of chute, (ft)
z_1	Basin elevation at end up chute, (ft)
z_2	Basin elevation at Section 2 (see Figure 10-13), (ft)
z_3	Basin elevation at Section 3 (see Figure 10-13), (ft)

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10.0 OUTLET PROTECTION

10.1 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 riprap 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 10.2. Recommended methods for estimating outlet erosion and scour potential are included in Section 10.3. Design details for riprap aprons, riprap outlet basins, baffled outlets, and U.S. Bureau of Reclamation (USBR) Type II basins are presented in Sections 10.4, 10.5, 10.6, and 10.7, respectively. Standard drawings for outlet protection structures used in Memphis and Shelby County can be found at the City of Memphis Engineering Division's Civil Standards Web page: http://www.cityofmemphis.org/framework.aspx?page=522.

10.2 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 10.3. 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 Fr 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.

10.3 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.

10.4 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.

10.4.1 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 conditions.

10.4.2 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.

10.4.3 Example Problems

Example 10-1. Riprap Apron Design for Minimum Tailwater Conditions

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.

Example 10-2. Riprap Apron Design for Maximum Tailwater Conditions

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.

4. 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.

10.5 Riprap Outlet Basins

A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipater 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.

10.5.1 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.

10.5.2 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 dissipater. 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 dissipater 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.

10.5.3 Example Problems

Example 10-3. Riprap Outlet Basin Design for Supercritical Flow and Minimum Tailwater Conditions

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.

10.6 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.

10.6.1 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:

   $W/d = 2.88Fr^{0.566}$ (10-2)

   or

   $W = 2.88dFr^{0.566}$ (10-3)

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.

5. Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 feet per second flowing full.

6. If the entrance pipe slopes downward, the outlet pipe should be turned horizontal for at least 3 feet before entering the baffled outlet.

7. 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 of the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

10.6.2 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

$$v = \sqrt{2\text{gh}} = \sqrt{2\left(32.2\text{ ft/sec}^2\right)\left(30\text{ ft}\right)}$$

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.

$$F_r = \frac{v}{\sqrt{gd}} = \frac{43.95\text{ ft/sec}}{\sqrt{\left(32.2\text{ ft/sec}^2\right)\left(1.85\text{ ft}\right)}} = 5.7$$

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

   $\frac{5\,feet,5inches}{2}$ + 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.

$$\frac{\pi d^2}{4} = \frac{Q}{v} \quad ; \qquad d = \sqrt{\frac{4Q}{\pi v}} = \sqrt{\frac{4(150\text{ cfs})}{\pi(12\text{ ft/sec})}} = 3.99\text{ feet}$$

Use 48-inch pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 inches.

10.7 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.

10.7.1 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:

$$W_B < W_o + 2L_T\left(\sqrt{S_T^{\ 2} + 1}\right) / 3(Fr_o) \quad (10\text{-}5)$$

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:

$$y_2 = C_1 y_1 \left[\sqrt{1 + 8Fr^2} - 1\right] / 2 \quad (10\text{-}6)$$

Where:

C_1 = TW/y_2

For USBR basins, C_1 is found using the procedure in Section 10.7.2. 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:

$$L_T = (z_o - z_1)/S_T \quad (10\text{-}7)$$

$$L_s = (z_3 - z_2)/S_s \quad (10\text{-}8)$$

$$L_B = f(y_1, Fr_1) \quad (10\text{-}9)$$

$$L = L_T + L_B + L_S = (z_o - z_3)/S_o \quad (10\text{-}10)$$

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.

      $Fr_1 = v_1 / \sqrt{g y_1}$.

   e. Calculate

   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.

10.7.2 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 10.7.1. For step 5f of Section 10.6.1, use C = 1.1 to find y_2. For Step 6 of Section 10.7.1, 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.

10.7.3 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.

10.7.4 Example Problem

Example 10-7. USBR Type II Outlet Basin Design

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 10.7.1:

   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.

10.8 Chapter Equations

$$\tau_c = 0.0001\,(S_v + 180)\tan(30 + 1.73\,PI) \qquad (10\text{-}1)$$

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

$$W/d = 2.88Fr^{0.566} \qquad (10\text{-}2)$$

$$W = 2.88dFr^{0.566} \qquad (10\text{-}3)$$

Where:

W = Minimum basin width, in feet
d = Depth of incoming flow, in feet
$F_r = v/\sqrt{gd}$ = Froude Number, dimensionless

$$Q = y_1 W_B \left[2g\left(z_o - z_1 + y_o - y_1\right) + v_o^{\,2}\right]^{0.5} \qquad (10\text{-}4)$$

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

$$W_B < W_o + 2L_T\left(\sqrt{S_T^{\,2} + 1}\right)/3(Fr_o) \qquad (10\text{-}5)$$

Where:

Fr = Outlet Froude Number
and the remaining terms are as defined in Figure 10-13

$$y_2 = C_1 y_1 \left\lfloor \sqrt{1 + 8Fr^2} - 1 \right\rfloor / 2 \quad (10\text{-}6)$$

Where:

C_1 = TW/y_2

$$L_T = (z_o - z_1)/S_T \quad (10\text{-}7)$$

$$L_s = (z_3 - z_2)/S_s \quad (10\text{-}8)$$

$$L_B = f(y_1, Fr_1) \quad (10\text{-}9)$$

$$L = L_T + L_B + L_S = (z_o - z_3)/S_o \quad (10\text{-}10)$$

$$z_3 = z_o - (L_T + L_B - z_2/S_s)\, S_o/(S_o/S_s + 1) \quad (10\text{-}11)$$

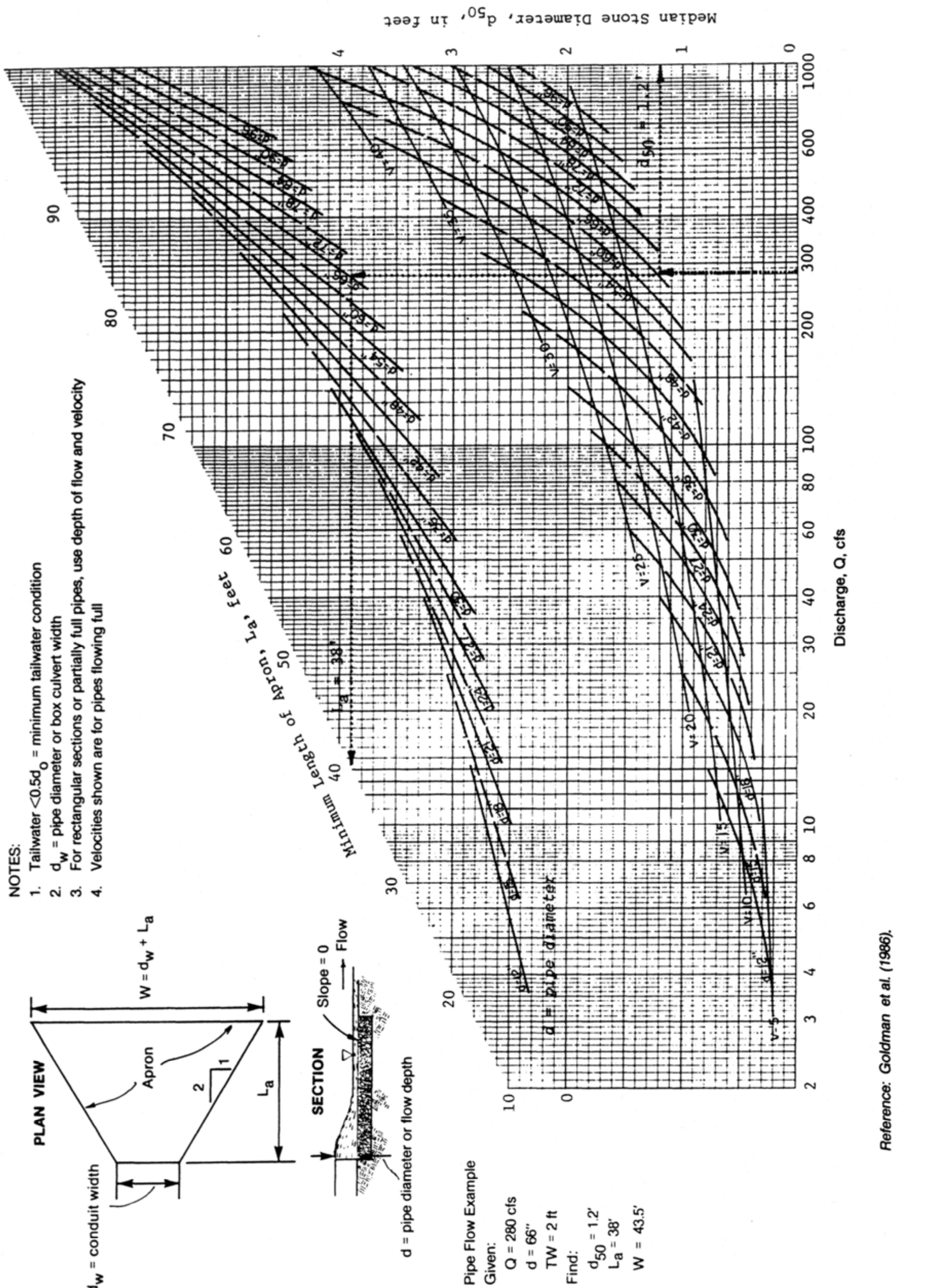

Figure 10-1
Design of Riprap Apron under Minimum Tailwater Conditions

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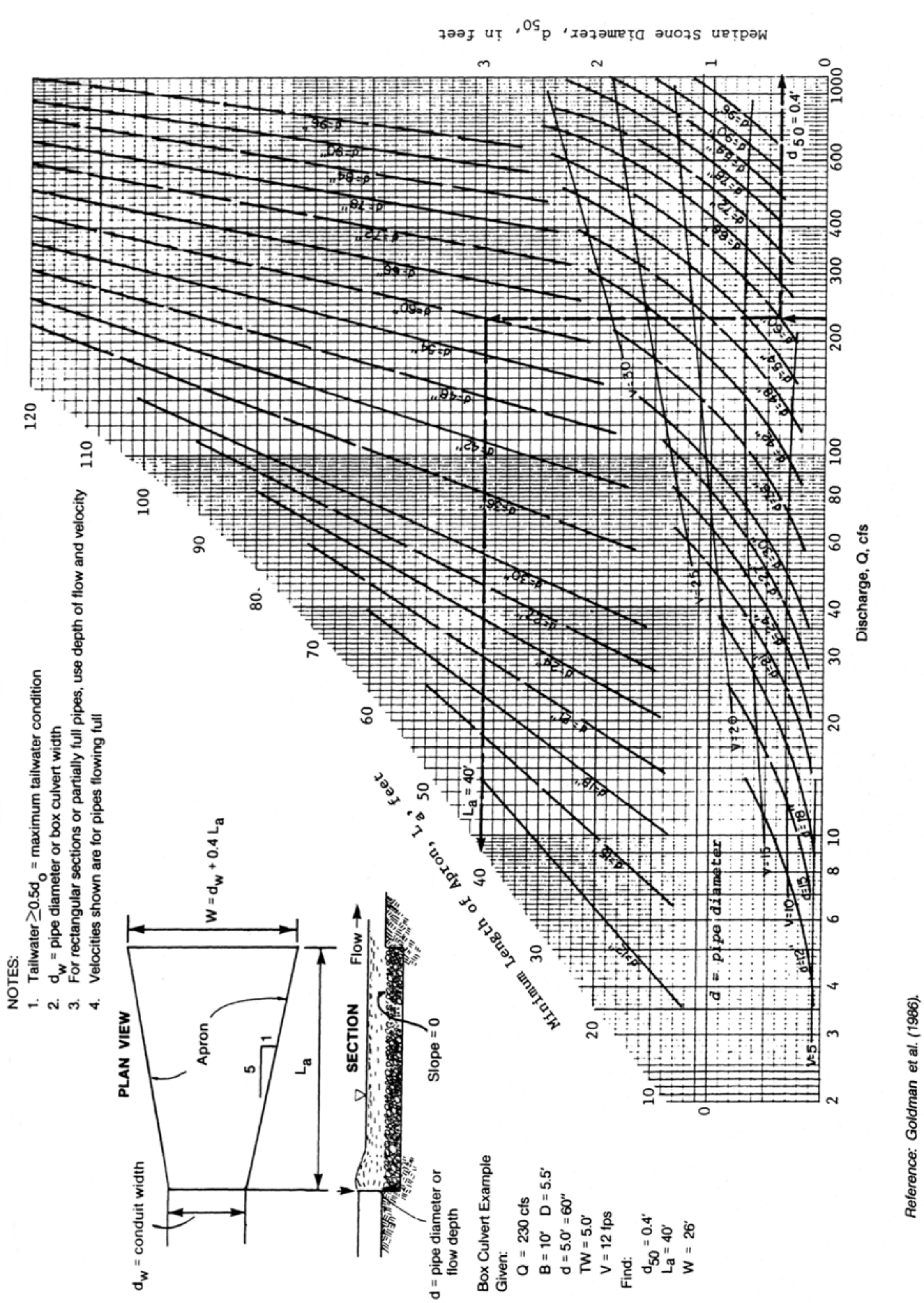


Figure 10-2
Design of Riprap Apron under Maximum Tailwater Conditions

Back to [Section 10.4.1]

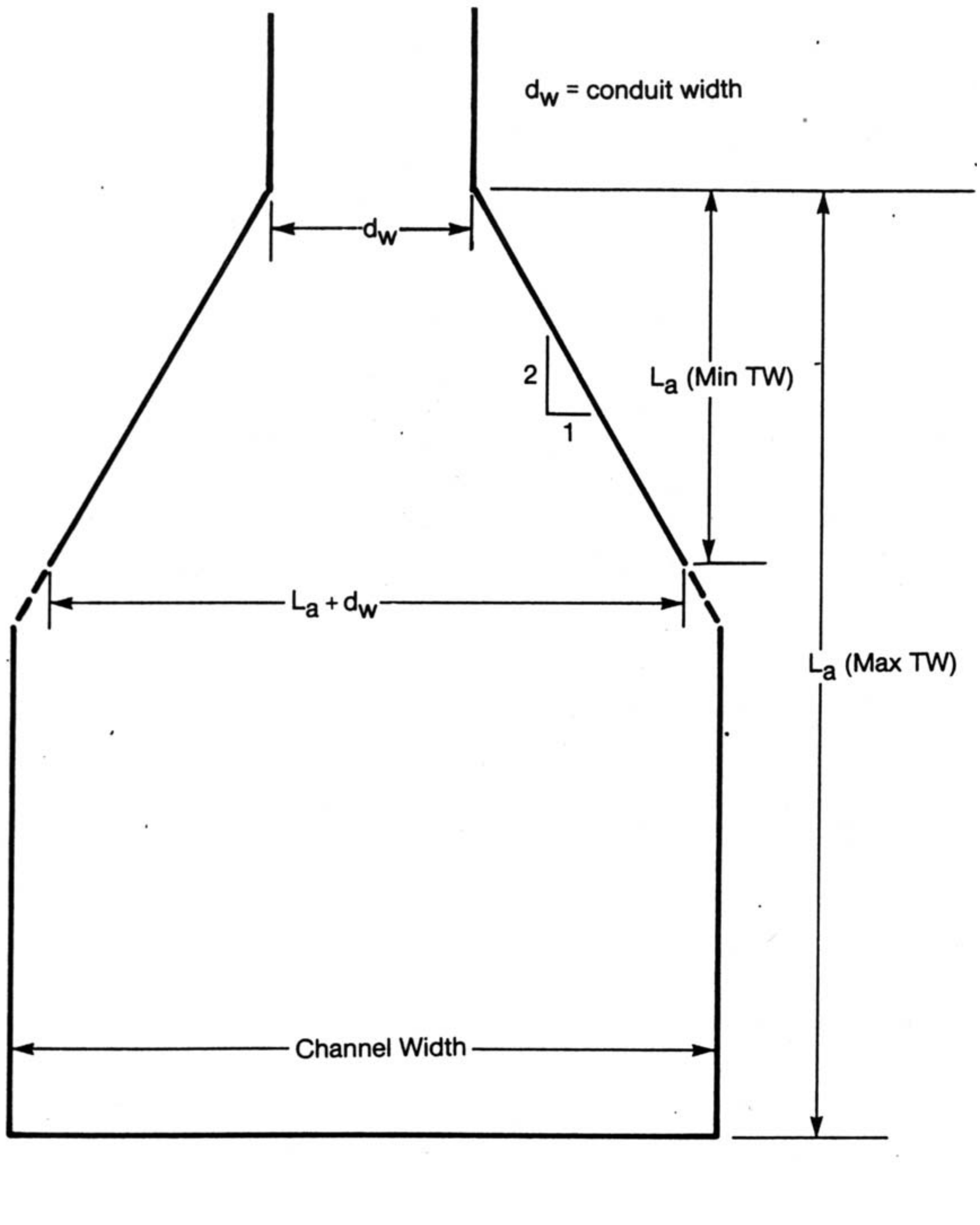
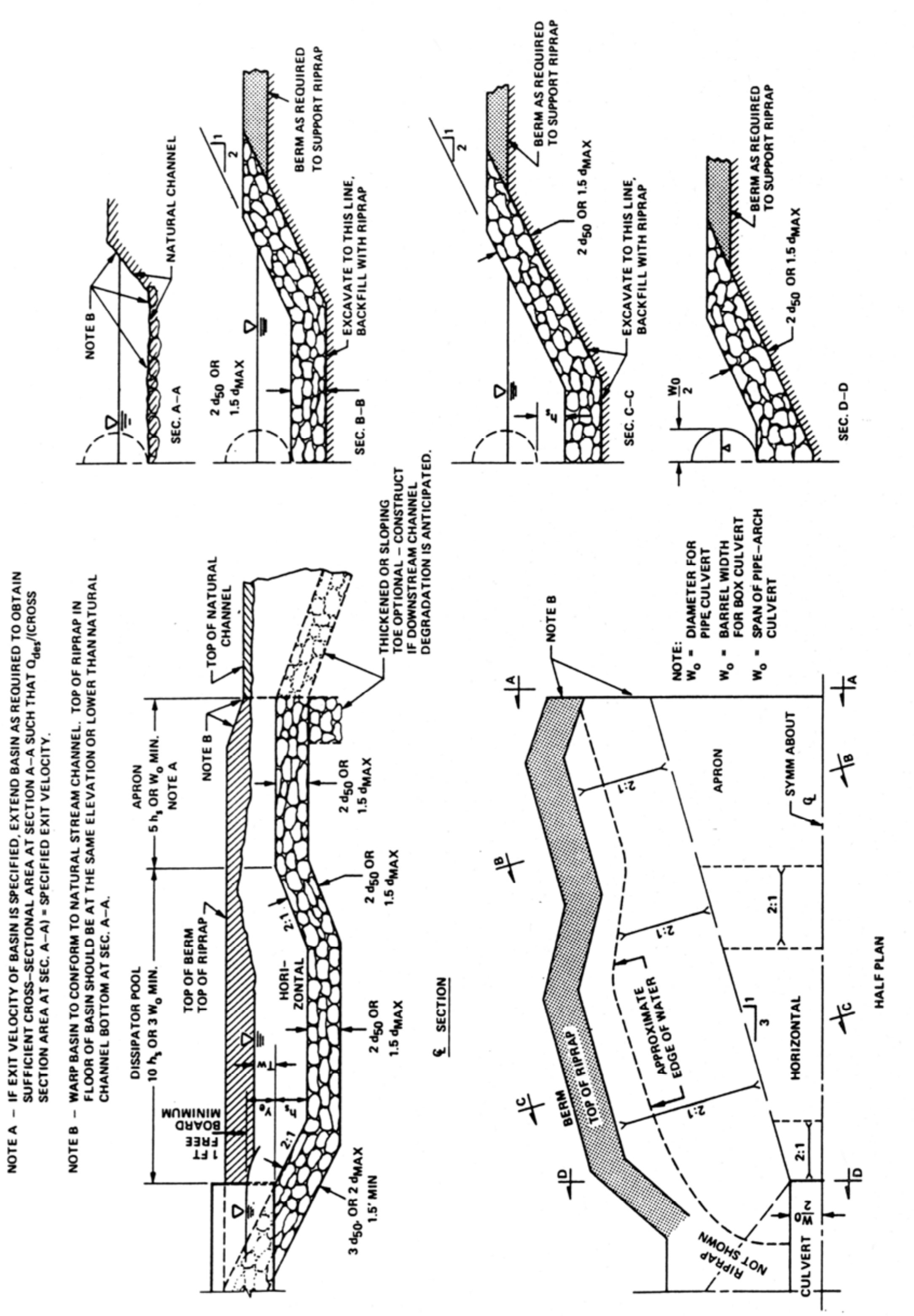


Figure 10-3
Riprap Apron Schematic for Uncertain Tailwater Conditions

Back to Section 10.4.1



Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-4
Details of Riprap Outlet Basin

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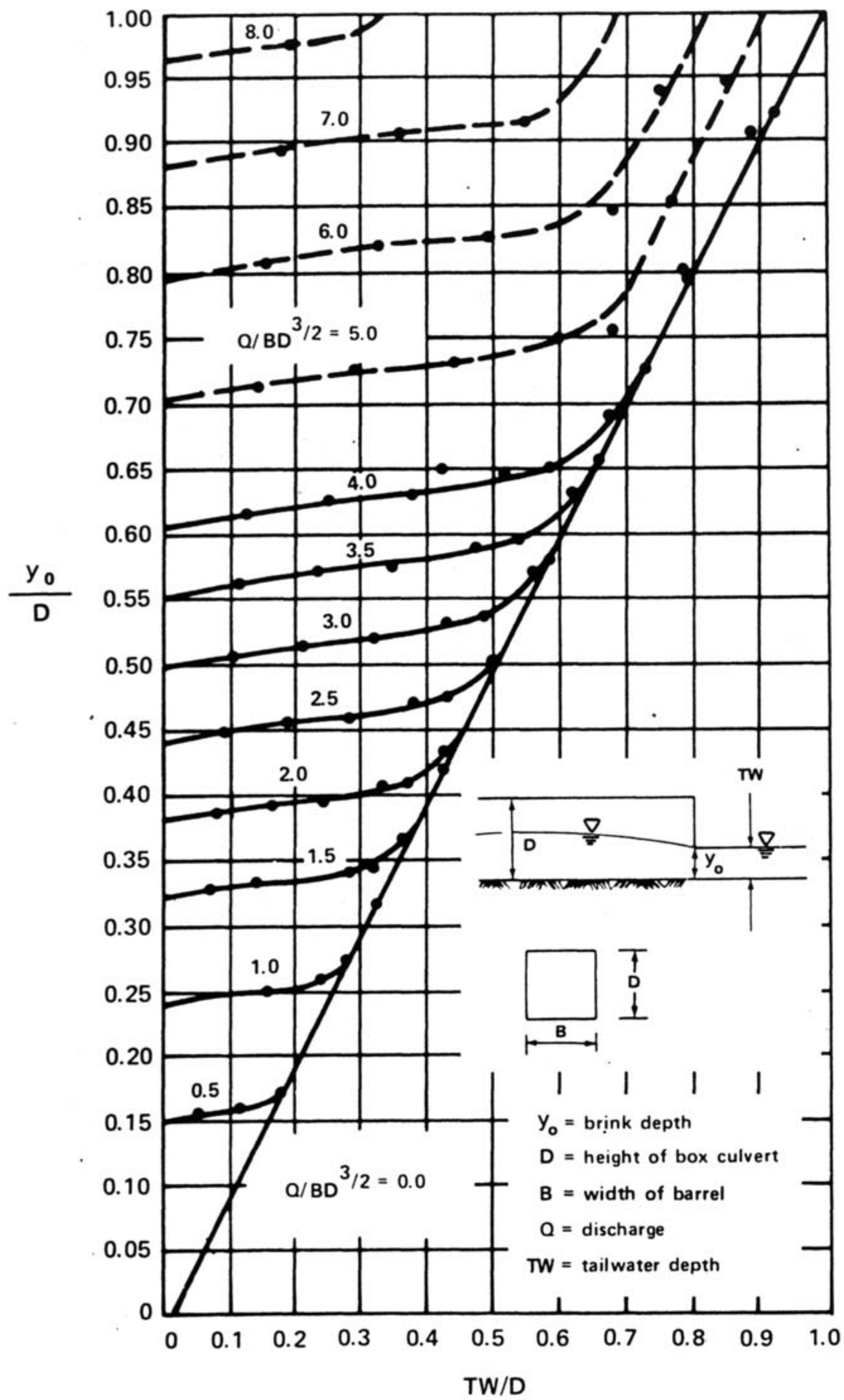

Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-5
Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

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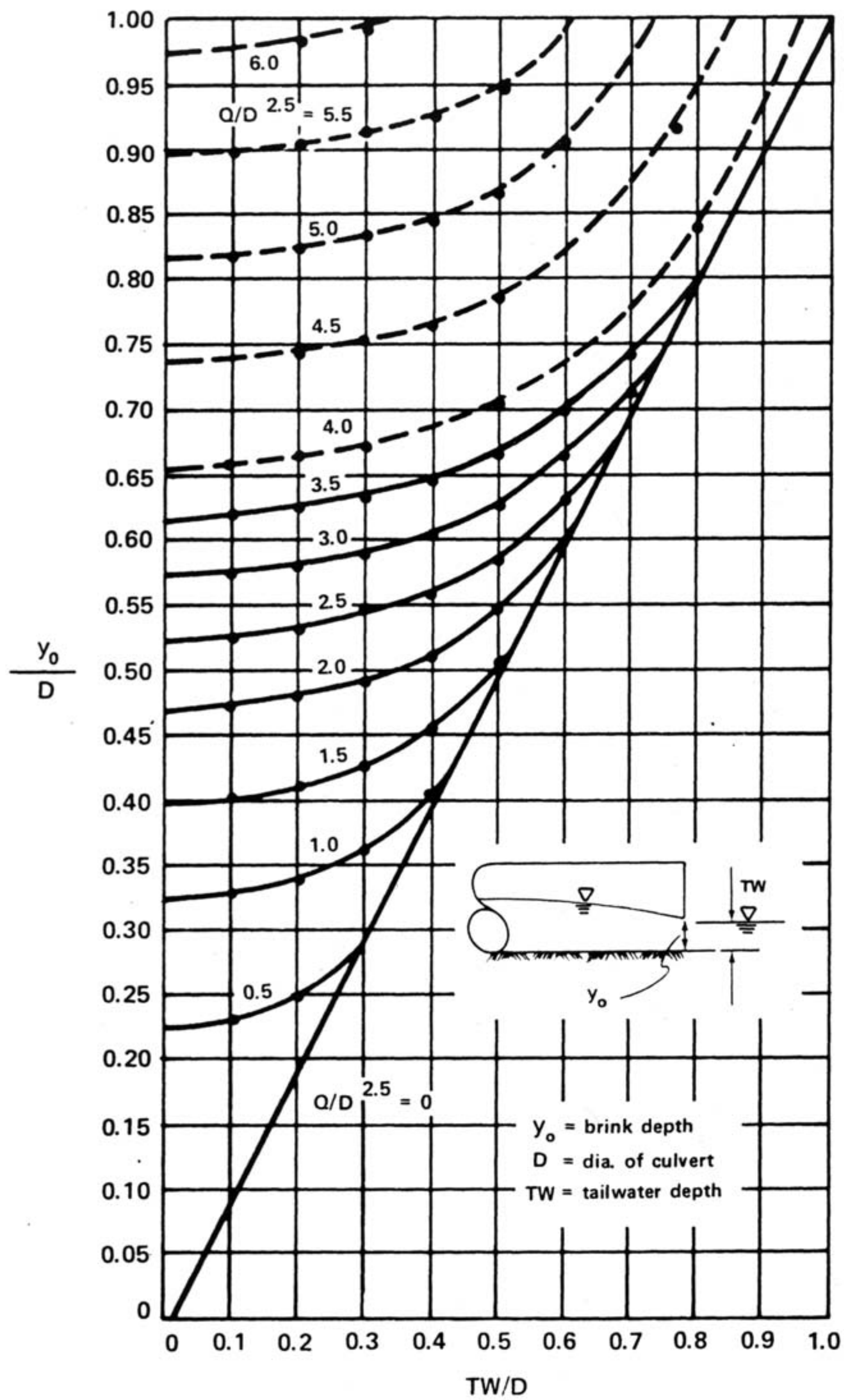

Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-6

Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

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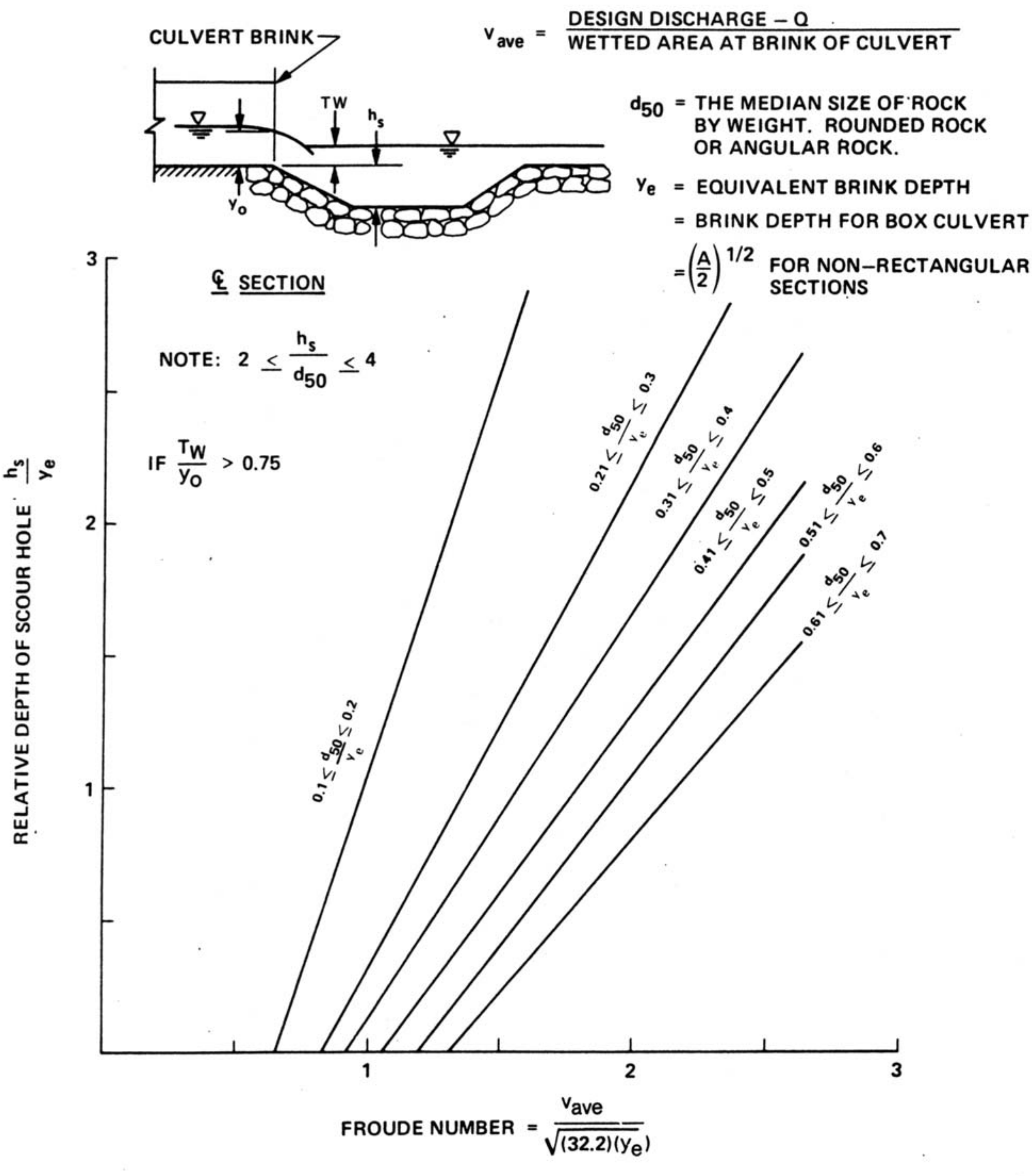
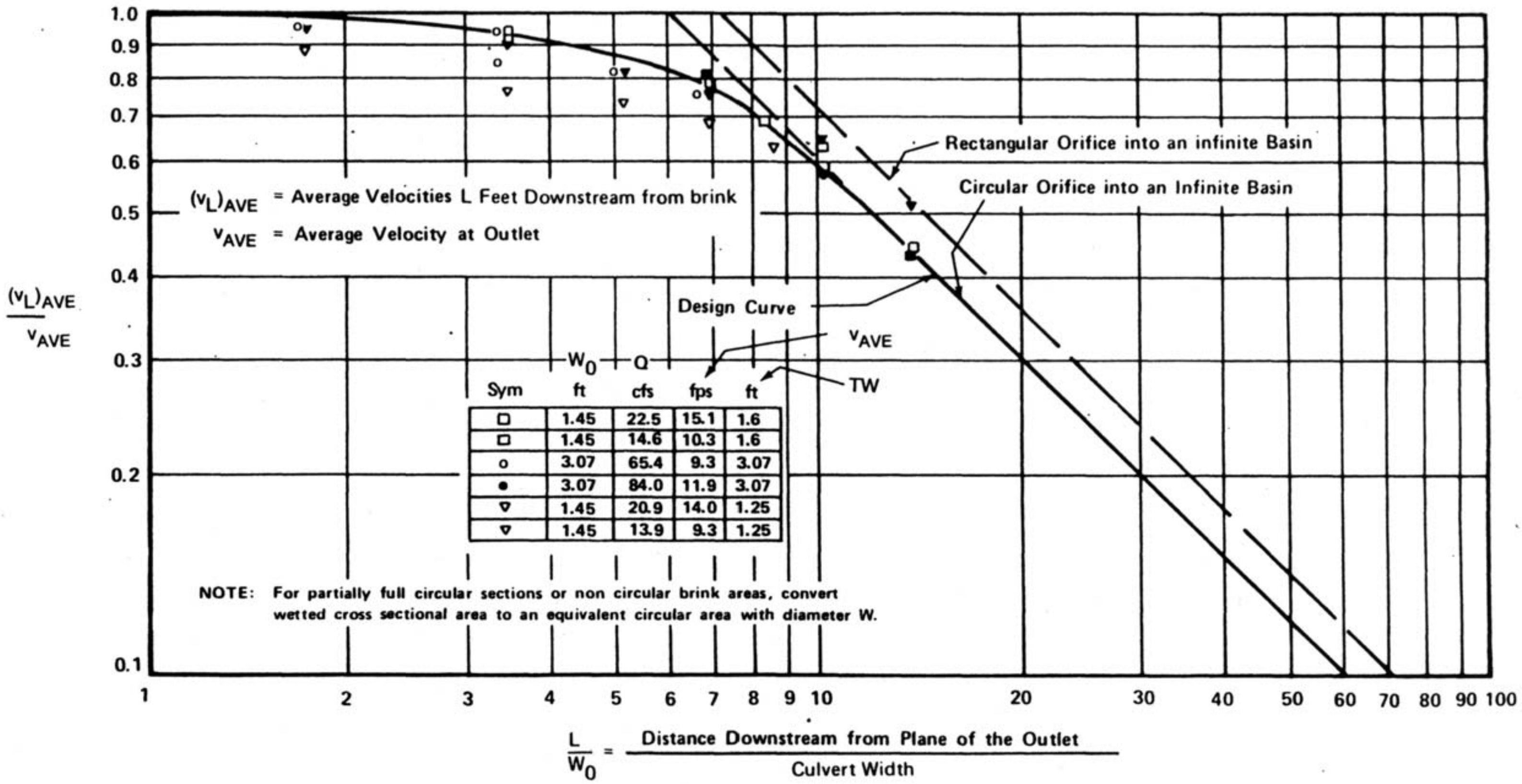


Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-7
Relative Depth of Scourhole vs. Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

Back to Section 10.5.1



Reference: *USDOT, FHWA, HEC-14 (1983).*

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.

Figure 10-8
Distribution of Centerline Velocity for Flow from Submerged Outlets

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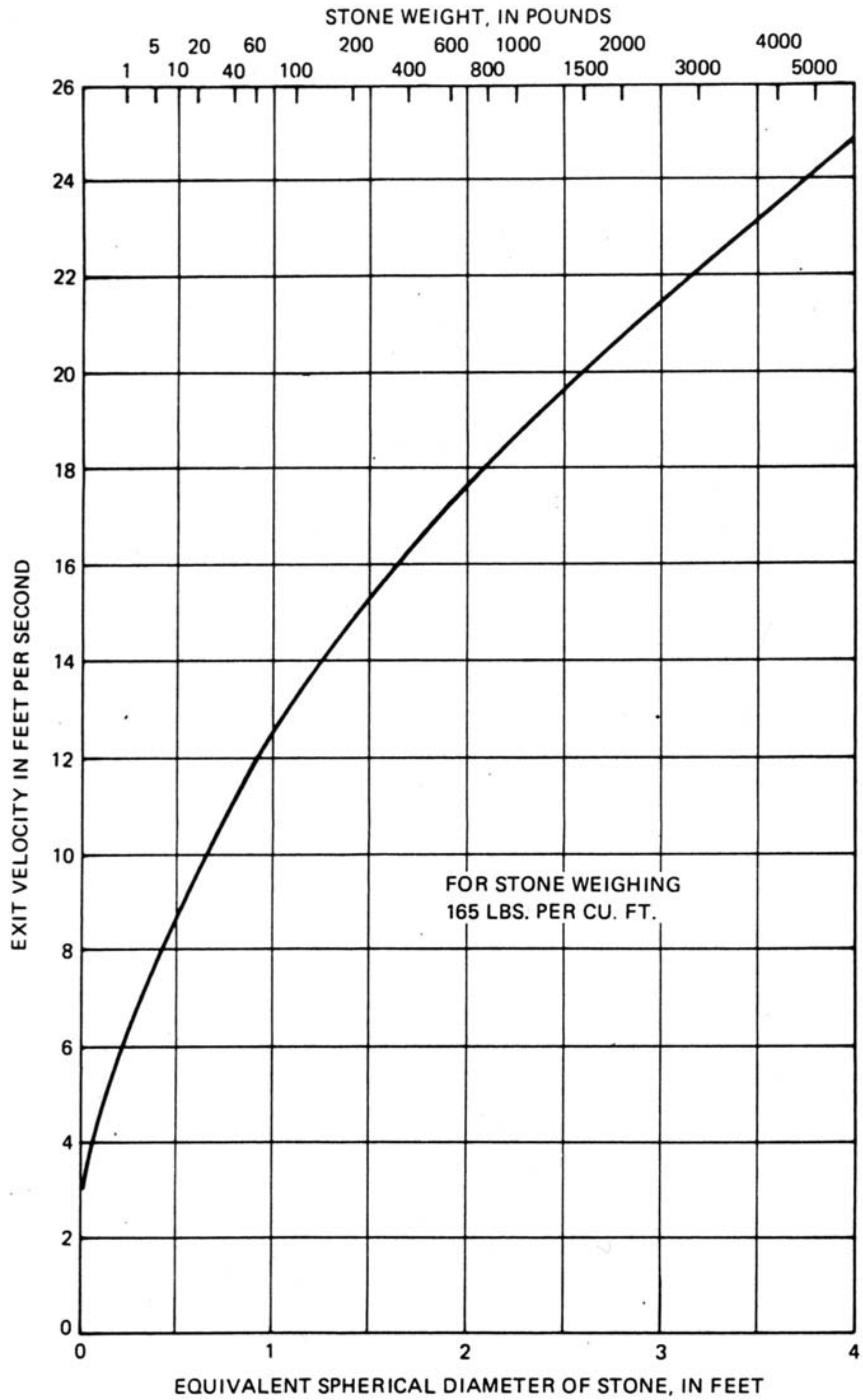

Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-9
Riprap Size for Protection Downstream of Outlet Basins

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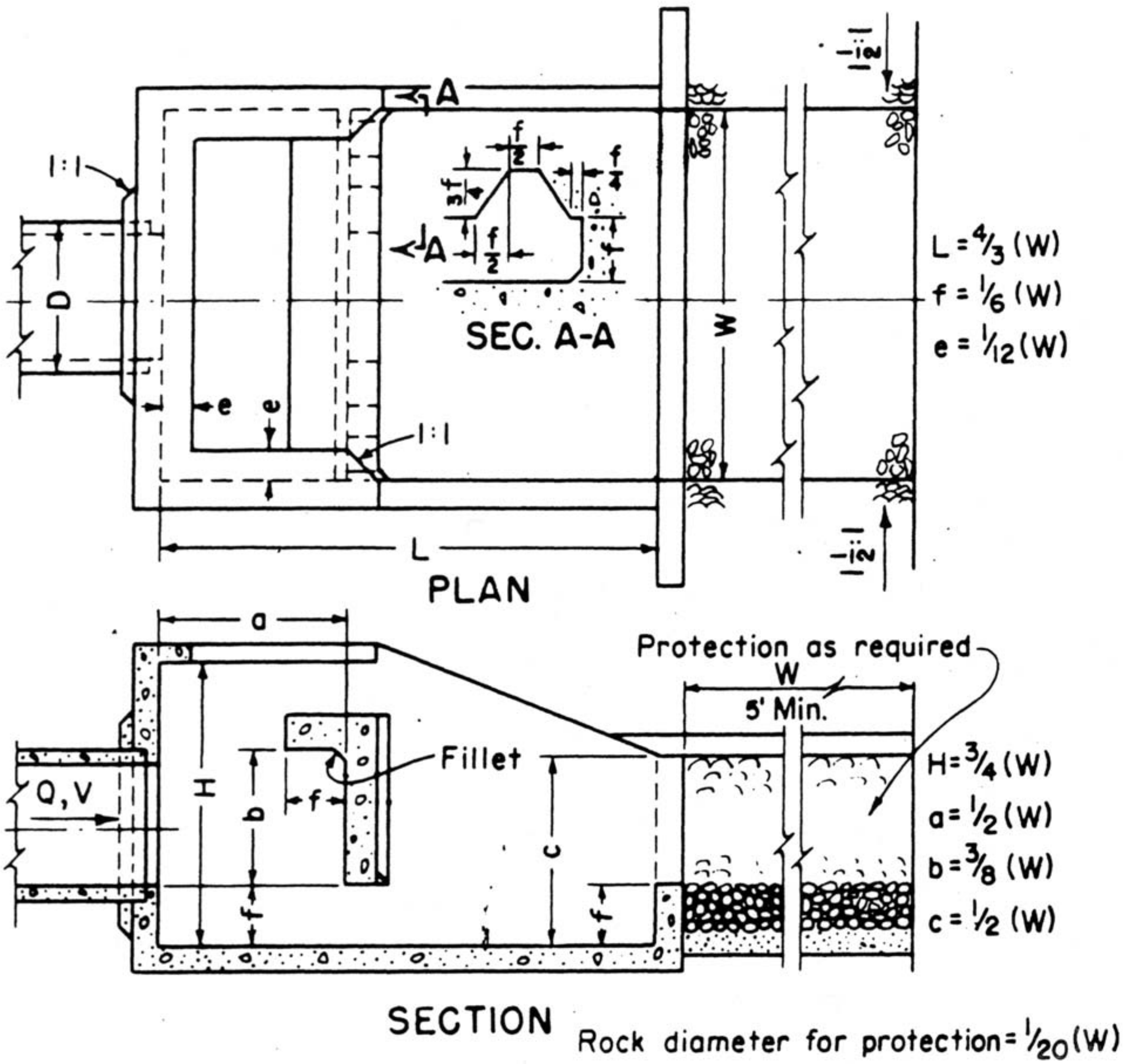

Reference: U.S. Department of the Interior (1978).

Figure 10-10
Schematic of Baffled Outlet

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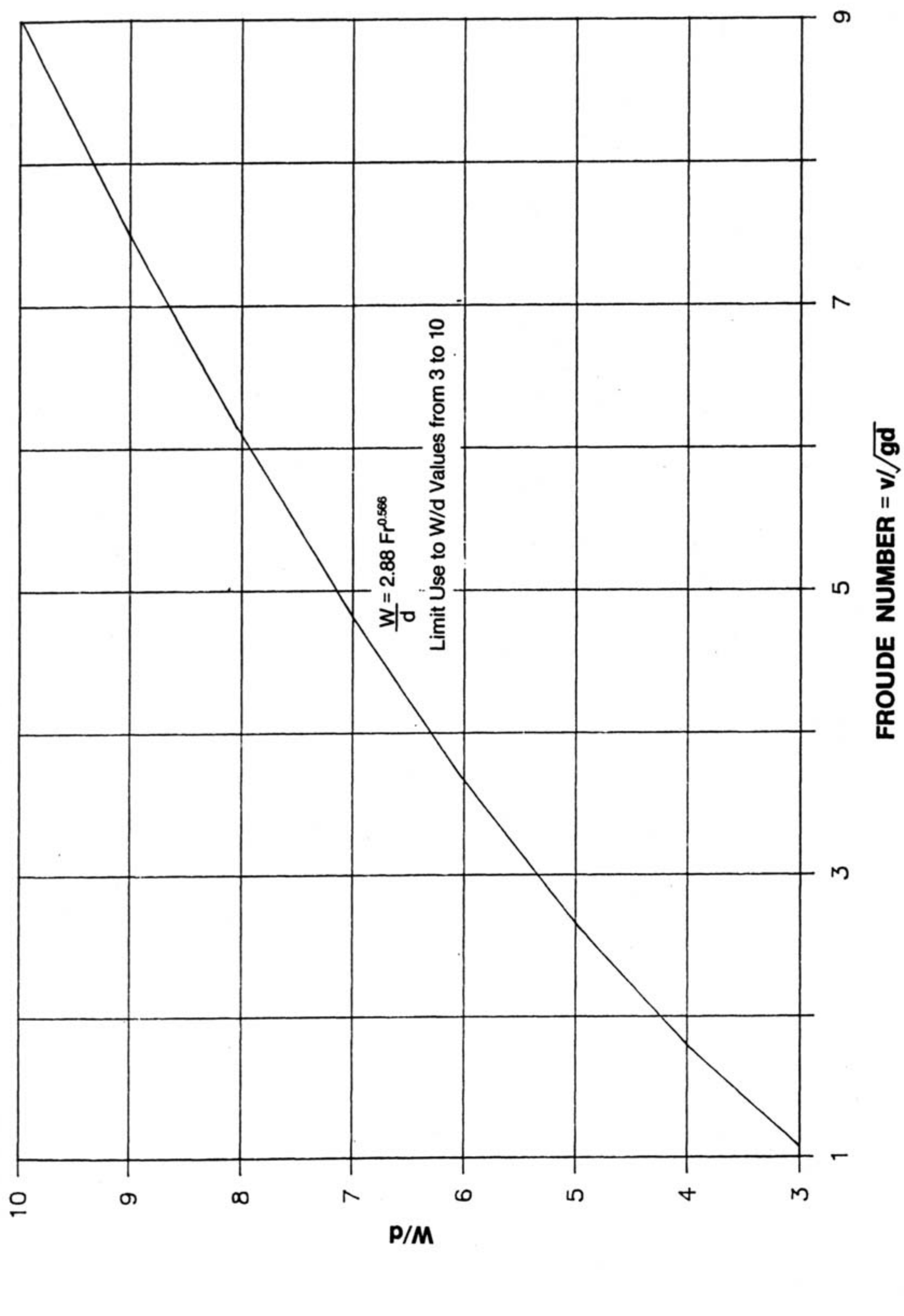
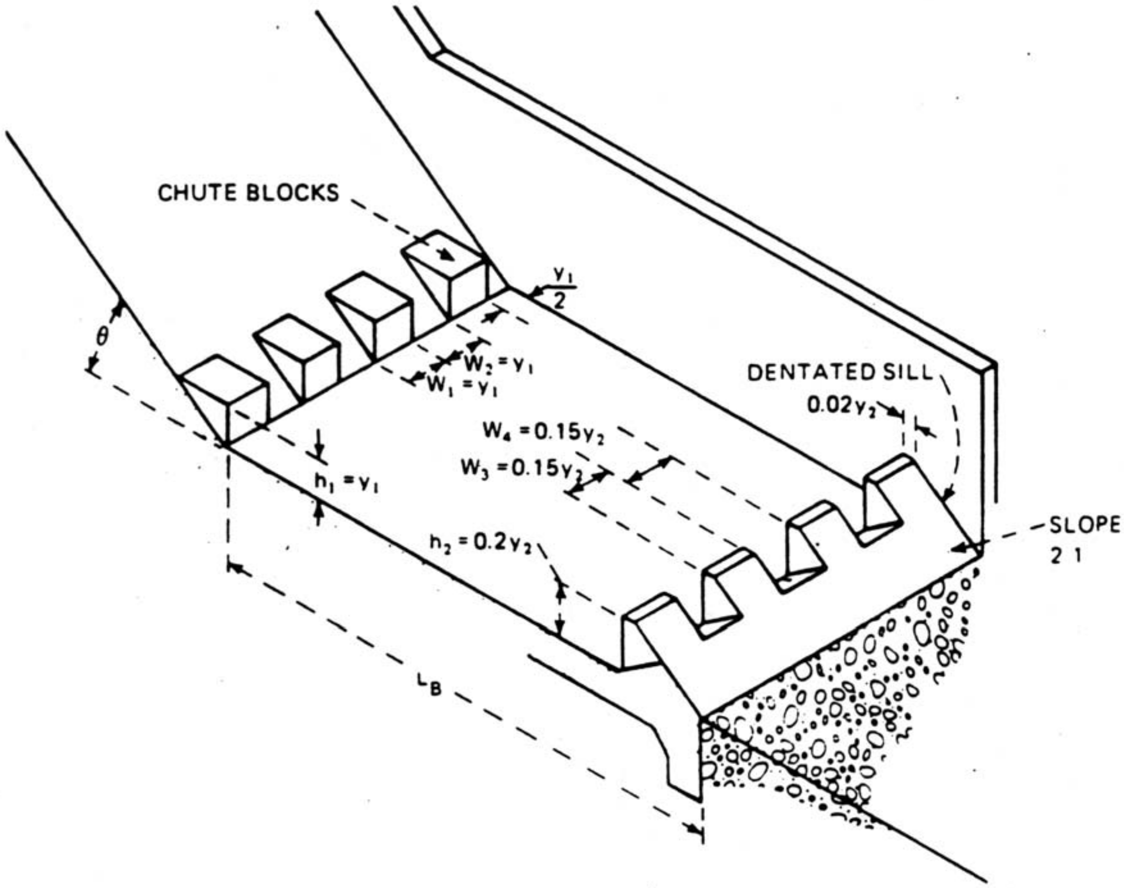
Back to Section 10.6.1



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

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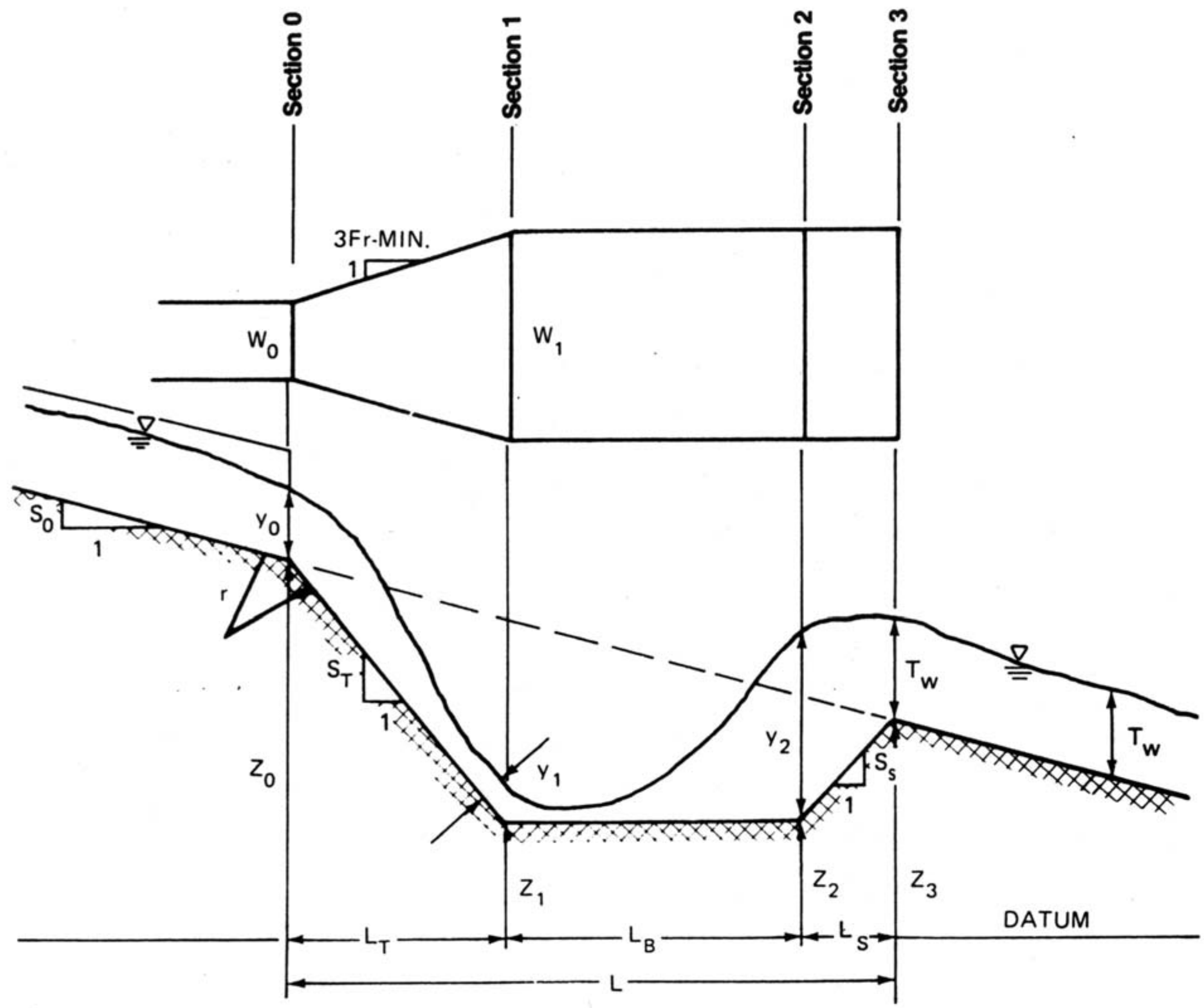

Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-13
Definition Sketch of Supercritical Flow Expansions

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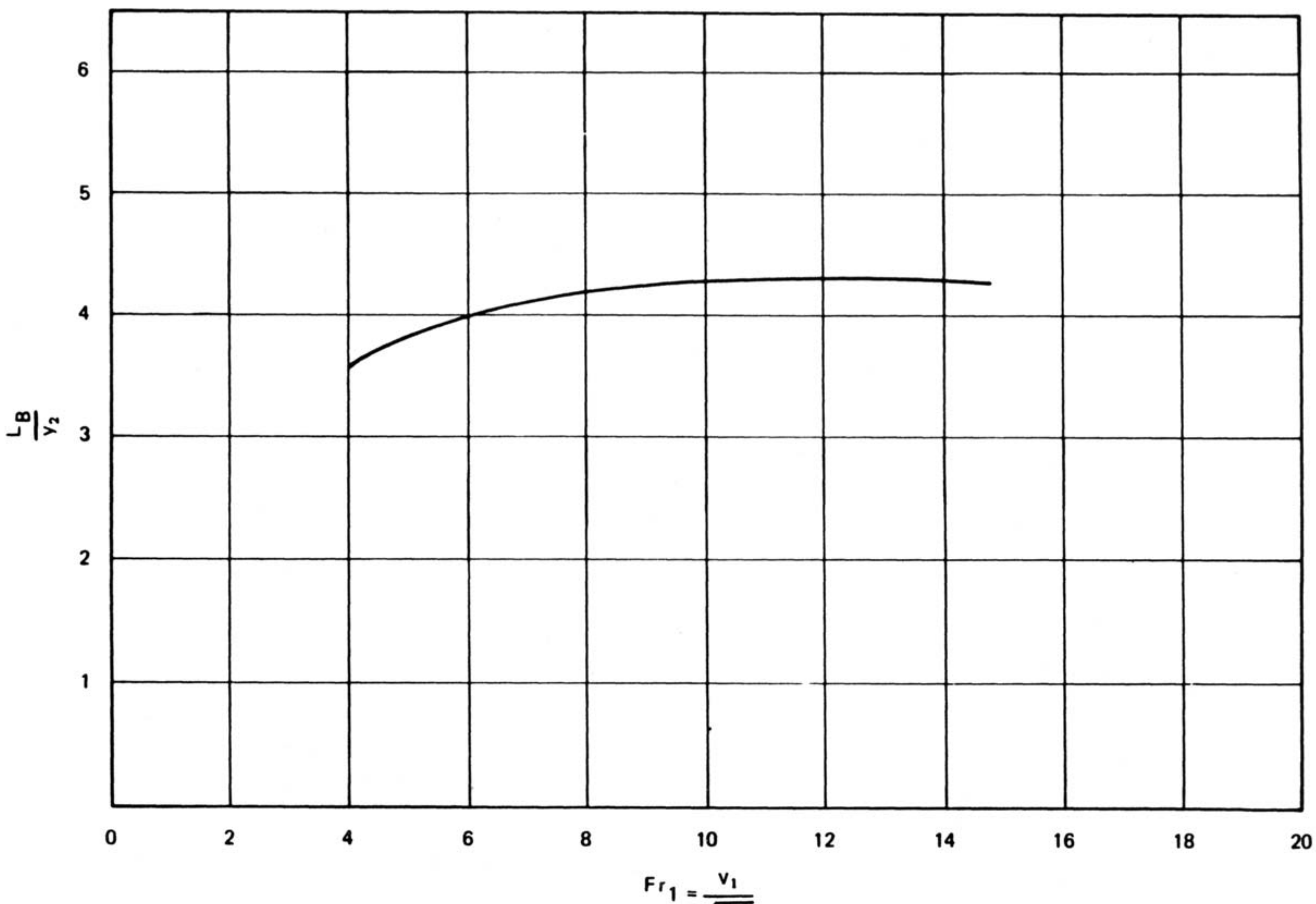

Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-14
Length of Jump on Horizontal Floor
for USBR Type II Outlet Basin

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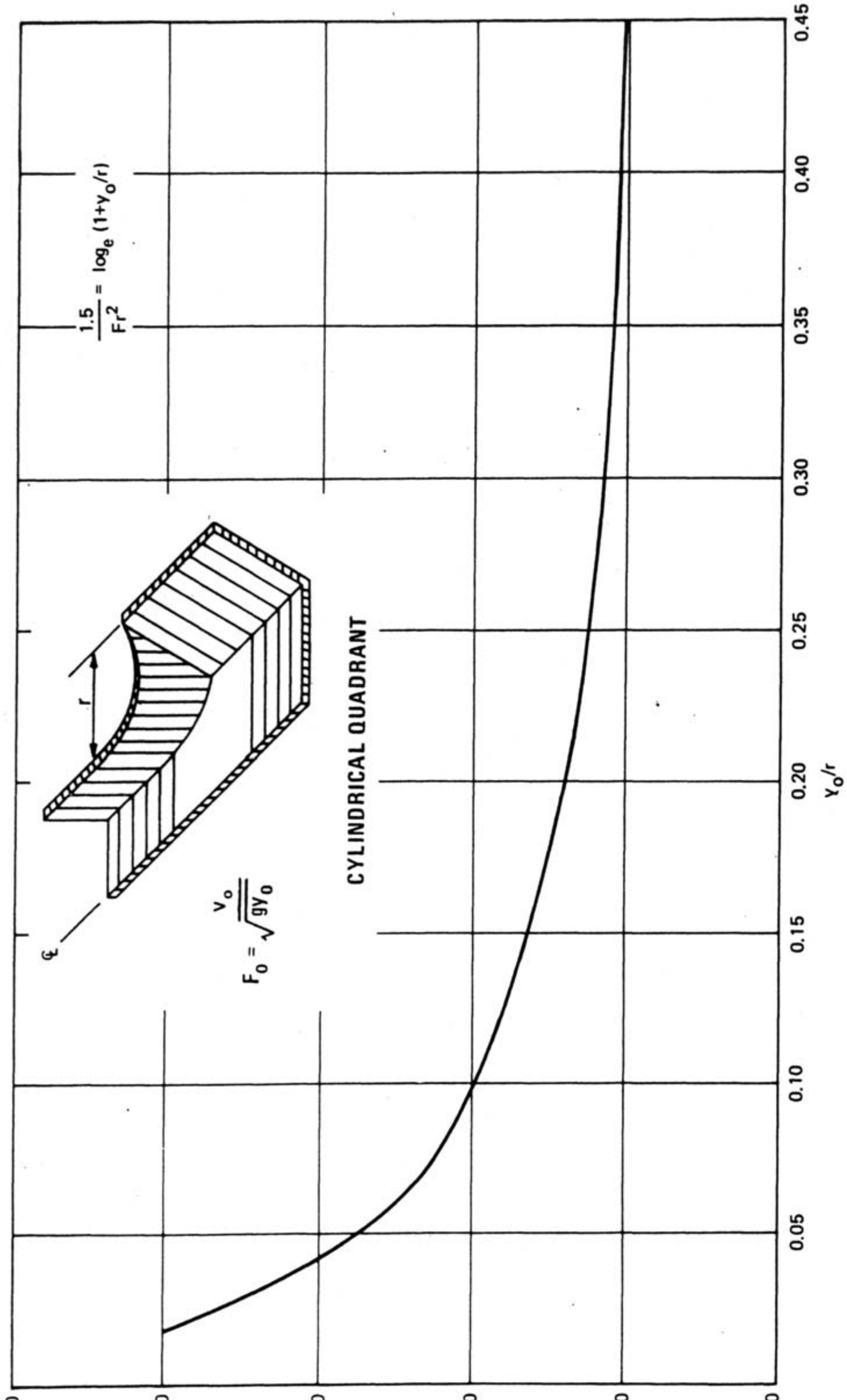

Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-15
Fr vs. y_o/r for Flow Transitions

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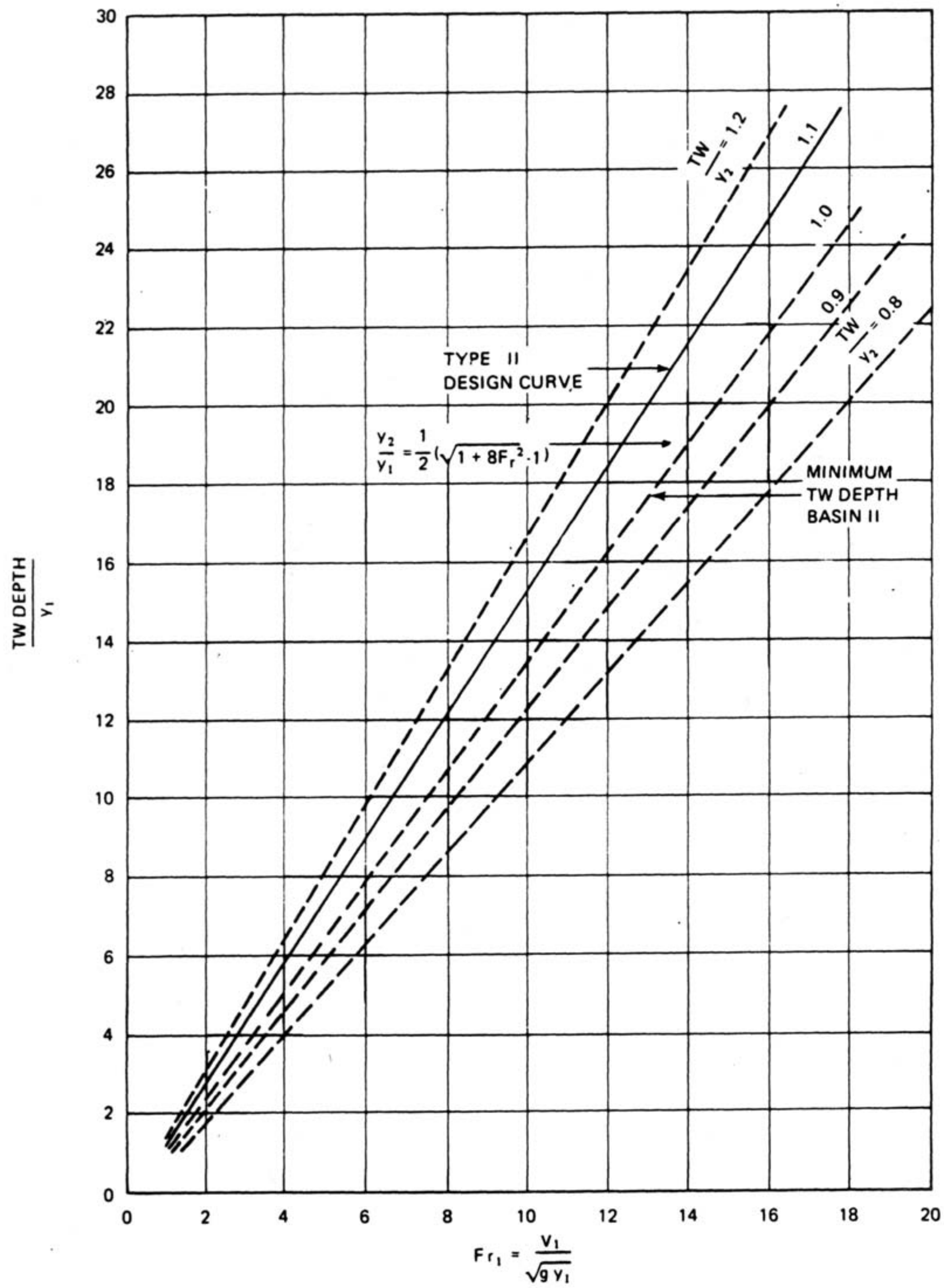

Reference: USDOT, FHWA, HEC-14 (1983).

Figure 10-16
Tailwater Depth for USBR Type II Outlet Basin

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Table 10-1
Suggested Outlet Protection Type — Based on Froude Number and Velocity

Type of Outlet Protection	Fr $\leq$ 2.5	Fr Between 2.5 and 4.5	Fr $\geq$ 4.5 and V < 50 [a]	Fr $\geq$ 4.5 and V > 50 [a]
Riprap Apron	X			
Riprap Outlet Basin	X			
Baffled Outlet	X [b]	X [b]	X [b]	
USBR Type II Basin				X

Notes:

[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 $\leq$ Fr $\leq$ 9.

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Table 10-2
Experimental Coefficients for Culvert Outlet Scour

Material	Nominal Grain Size d_{50} (mm)	Scour Equation	Depth h_s α	β	θ	α_e	Width W_s α	β	θ	α_e	Length L_s α	β	θ	α_e	Volume V_s 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.

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, or\ \frac{V_s}{D^3}\right] = \propto \left(\frac{Q}{\sqrt{g}\,D^{5/2}}\right)^{\beta} \left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-2. FOR OTHER CULVERT SHAPES. Same material as above.

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, or\ \frac{V_s}{y^3{}_e}\right] = \propto_e \left(\frac{Q}{\sqrt{g}\,y_e{}^{5/2}}\right)^{\beta} \left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.
Reference: USDOT, FHWA, HEC-14 (1983).

V-3. FOR CIRCULAR CULVERTS. Cohesive sandy clay with PI = 5-16.

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, or\ \frac{V_s}{D^3}\right] = \propto \left(\frac{\rho V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

V-4. FOR OTHER CULVERT SHAPES. Cohesive sandy clay with PI = 5-16.

$$\left[\frac{h_s}{y_e}, \frac{W_s}{y_e}, \frac{L_s}{y_e}, or\ \frac{V_s}{y_e{}^3}\right] = \propto_e \left(\frac{\rho V^2}{\tau_c}\right)^{\beta} \left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min.

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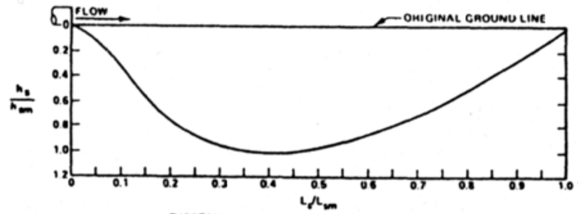
Project ________
Station ________
Designer ________
Date ________



DIMENSIONLESS CENTER-LINE PROFILE

(1) $y_e = (A/2)^{1/2}$

	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
CULVERT													

	TYPE	SIDE SLOPE	"n"	BOTTOM WIDTH	SLOPE	DISCHARGE Q	VELOCITY	FLOW AREA	TW	FREE BOARD	FR	TYPE OF MATERIAL
CHANNEL												

	EQUIVALENT DEPTH y_e	SCOUR DEPTH h_s	SCOUR WIDTH W_s	SCOUR LENGTH L_s	SCOUR VOLUME V_s	
SCOUR COMPUTATIONS						

	VELOCITY	SCOUR DEPTH	SCOUR WIDTH	SCOUR LENGTH	OTHER RESTRICTIONS
ALLOWABLE CONDITIONS					

OTHER SITE CONSTRAINTS	

Reference: USDOT, FHWA, HEC-14 (1983).

Table 10-3
Culvert, Channel, Scour, and Other Site Data Computation Form

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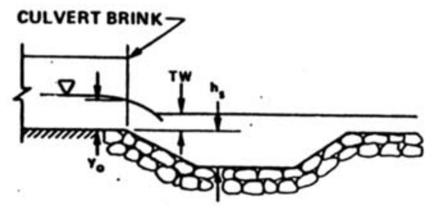

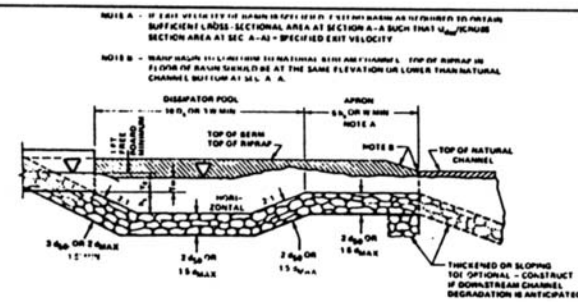

	TW	y_e	(1) TW/y_e	d_{50}/y_e	d_{50}	h_s/y_e	h_s	(2) h_s/d_{50}	
LOW TW $TW/y_e \le 0.75$									
	V ALLOWABLE	L/D_e (3)	L	V_{ave}/V_L	V_L				
HIGH TW $TW/y_e > 0.75$									

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

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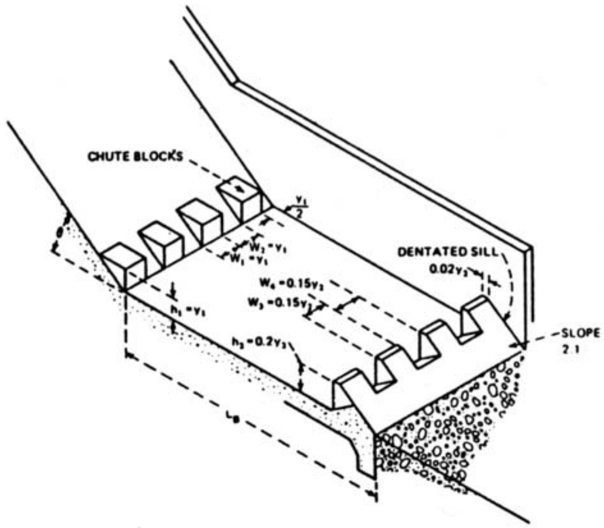

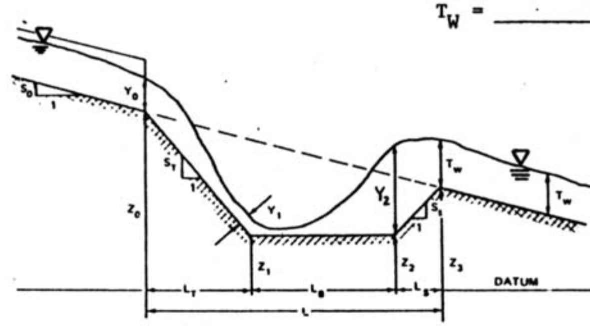

C_1 = ______
S_s = ______
S_T = ______
W_B = ______
y_2 = ______
T_W = ______

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

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