COPIES OF THESE

REGULATIONS AND TECHNICAL GUIDELINES FOR STORM WATER MANAGEMENT

MAY BE OBTAINED FROM THE OFFICE OF THE CITY ENGINEER,
CITY OF HUNTSVILLE, P. O. BOX 308, HUNTSVILLE, ALABAMA 35804-0308

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HUNTSVILLE STORMWATER MANAGEMENT MANUAL

Prepared for

HUNTSVILLE DEPARTMENT OF PUBLIC WORKS

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HUNTSVILLE STORMWATER MANAGEMENT MANUAL

PREFACE

The Huntsville Stormwater Management Manual has been prepared to establish regulations and technical guidelines on stormwater management for developers, landowners, builders, architects, engineers, and others involved in development activities. The manual is available from the Department of Public Works.

ACKNOWLEDGEMENTS

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Honorable Steve Hettinger, Mayor Sept. 1988 - present

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ABBREVIATIONS

AASHTO	American Association of State Highway Transportation Officials
AMC	Antecedent Moisture Condition
cfs	cubic feet per second
CMP	corrugated metal pipe
csm/inch	cubic feet per second per square mile per inch
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
IDF	intensity-duration-frequency
NGVD	National Geodetic Vertical Datum
SCS	Soil Conservation Service
SMM	Stormwater Management Manual
TVA	Tennessee Valley Authority
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
USGS	United States Geological Survey
WSO	Weather Service Office

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A Ordinance No. 87-769 A-1

B Stormwater Management Ordinance (to be adopted) B-1

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Appendices (Bound Separately)

1 Standard Detail Drawings

2 Ordinance No. 91-

ORDINANCE NO. 94-265

BE IT ORDAINED by the City Council of the City of Huntsville, Alabama, that the following sections of the Stormwater Management Manual of the City of Huntsville Department of Public Works, 1991 Edition, as adopted by Ordinance No. 91-375, and amended by Ordinance No. 91-432 and Ordinance No. 94-186, are hereby amended to read as follows:

Amend Section 2.1.1 to read as follows:

2.1.1 STANDARDS

To implement the objectives presented above, the following standards shall apply:

1. The calculated peak rate of stormwater runoff resulting from a ten-year return period, twenty-four (24) hour duration, type II storm distribution (as defined by the Soil Conservation Service, U.S. Department of Agriculture) shall be no greater after development of a site than before development of a site.

2. No construction, whether by private or public action, shall be performed in such a manner as to materially increase the degree of flooding in its vicinity or in other areas, except that construction fill within the floodway fringe areas and along unstudied streams shall be accomplished according to the flood plain regulations of the City of Huntsville Zoning Ordinance.

Amend Section 2.2.2 to read as follows:

2.2.2 GRADING PERMITS

Grading permits are required from the Engineer of Public Works before the commencement of any land disturbance activity; including grading, excavation, clearing, filling, or construction or paving of any parking lot or existing parking area exceeding 10,000 S.F. of impervious area, within the City of Huntsville unless exempt by the provisions of Section 2.3, GRADING PERMIT EXEMPTIONS. Exemptions shall not be construed as relieving the responsible party from other requirements within these regulations and/or flood plain requirements (see Chapter 3, Section 3.4). Approval of or exemption from the grading permit requirements shall not be construed as relieving the responsible party from the requirements of any other applicable federal, state or local laws.

Amend Section 2.3.1 to read as follows:

2.3.1 EXCAVATION OR FILL

No grading permit is required for an excavation or fill that satisfies all of the following criteria:

1. Is less than 4 feet in vertical depth at its deepest point as measured from the natural ground.
2. Will not result in a total quantity of more than 200 cubic yards of material being removed from, deposited on, or disturbed on any lot, parcel, or subdivision thereof.

3. Will not impair existing surface drainage, constitute a potential erosion hazard, or act as a source of sedimentation to any adjacent land or watercourse. This condition requires that vegetation cover be re-established immediately on all disturbed areas.
4. Will have no fill placed on a surface having a slope steeper than 5 feet horizontal to 1 foot vertical.
5. Will have no final slopes cut steeper than 3 feet horizontal to 1 foot vertical.
6. Will not contain hazardous substances.

In addition, the following activities do not require a grading permit:

1. Burial within approved human or animal cemeteries.
2. Construction of accessory structures (as defined in the Zoning Ordinance of the City of Huntsville) related to single family residences or duplex structures authorized by a building permit, provided the disturbed material or fill is handled in such a manner as to conform to the approved erosion control plan for the area or, where no such erosion control plan is in effect, that such work is performed in conformance with the basic principles of erosion and sedimentation control as contained in Chapter 10, Section 10.1.2 of this manual.
3. Paving of existing parking areas that cover less than 10,000 square feet and satisfy all the requirements for exemption.
4. Construction and paving of new parking areas less than 2,500 square feet and satisfying all the requirements for exemption.
5. Resurfacing existing paved (impervious) areas.

Amend Section 2.4.2 to read as follows:

2.4.2 REQUIRED INFORMATION

A complete application package shall include information on the following items:

1. A complete plan of the proposed development at a scale no less than 1 inch = 100 feet. This plan is to include existing and proposed contours at intervals no greater than 2 feet with at least one on-site bench mark required, including the elevation relative to National Geodetic Vertical Datum (NGVD). Contours must be drawn showing true elevations referenced to the site bench mark shown on the plans. Contour lines shall be extended to the centerline of all roads/streets bordering the sites.
2. Existing and proposed buildings, impervious surfaces (all parking areas will be considered impervious), easements, right-of-way land, and drainage structures, including inlets, catch basins, manholes, junction boxes, driveway pipes, culverts, cross drains, headwalls, outlet facilities, and sanitary sewers, with size, type, slope, invert elevations, and quantity indicated.
3. If a project will be constructed in phases, common facilities and phasing schedules.
4. Hydrologic and hydraulic calculations for appropriate design conditions and facilities, including detention/retention facilities and reservoir routing calculations as required by Chapter 9 of this manual.
5. Any proposed ditches or channel improvements, with typical section and proposed limits of change indicated.
6. Any high water marks or flood lines, either calculated or observed in the vicinity of the proposed development, and the source of said line or elevation indicated.

7. All fill areas indicated as such, with the limits and elevation indicated.
8. Drainage arrows indicating the existing and proposed direction of runoff throughout the plan.
9. Invert and top of grate elevations on all catch basins and inlets in addition to flow line elevations, stations, and percent grades of all cross drains and pipe between inlets and catch basins.
10. For flood plain areas, designated floodway along with regulatory flood elevations, cut and fill cross sections and calculations, and lowest floor elevations for buildings in the flood plain. Hydraulic calculations should be submitted, when appropriate.
11. Temporary erosion and sedimentation control measures (straw bales, silt fence, etc.) to be implemented during construction, as required by Chapter 10 of this manual.
12. Final stabilization measures proposed for all disturbed areas on the property. Areas with slopes equal to or greater than two feet horizontal to one foot vertical shall be stabilized with riprap or another equally effective stabilization measure. Stabilization for each ditch shall be shown.
13. Where special structures such as box culverts, bridges, or junction boxes are proposed, detail plans showing dimensions, reinforcement, spacing, sections, elevations, and other pertinent information.
14. A summary of the quantities of materials to be used and an estimate of the cost of construction and completion of improvements required by this manual (for which maintenance of such improvements by the City is required or anticipated) shall be provided so as to determine the amount of the bond required under Section 2.5.6 herein.
15. Plans and calculations signed and sealed by a registered engineer, landscape architect, or land surveyor, if appropriate, if application is for a grading permit. If application is for a building permit, signature and seal of a registered engineer or architect is required unless otherwise exempted.

Exemptions from the requirement to furnish existing and proposed contours, site bench mark, and the stamp of a registered engineer, architect, or land surveyor, shall be given in following specific case:

1. Construction of or an addition to a non-residential structure that has less than 2,500 square feet of floor space and less than 5,000 square feet (including structure) of additional impervious area;
2. Construction of or an addition to any single family or duplex residential structure, not located in an approved subdivision, of less than 5,000 square feet of impervious area; or
3. Construction of a swimming pool related to single-family residential or duplex structures, provided that the activities exempted herein satisfy the excavation or fill criteria of Section 2.3.1 (Swimming pools are exempt from the requirements of paragraph (1) and (2) under the excavation or fill criteria of Section 2.3.1 except for the requirement under paragraph (2) that no more than 200 cubic yards of material may be deposited on any such lot or parcel without an approved grading plan).

Applications that do not require contour elevations and an on-site bench mark as provided above shall include existing and proposed spot elevations along with a site plan showing distances of any new structures or construction from the property lines.

These exemptions shall not be construed as relieving the identified activities from compliance with the flood plain requirements of Ordinance No. 87-269, as amended, or item (10) of this Section or any other applicable regulations.

Additional information required for various facility components is covered in Chapter 3, Section 3.2.

Amend Section 3.2.5 to read as follows:

3.2.5 STORAGE SYSTEMS

The standard for determining the requirement for storage systems shall be that the calculated peak rate of stormwater runoff shall be no greater after development of a site than before development of a site. The calculations shall be made considering a ten year return period, twenty-four hour duration, type II storm distribution as defined by Soil Conservation Service (SCS), U.S. Department of Agriculture (USDA), for both before and after development site conditions. The hydraulics and design of stormwater storage systems shall be consistent with the criteria and procedures presented in Chapter 9.

Exemptions from the requirement to provide on-site retention and/or detention facilities shall be given in the following specific cases:

1. Commercial or industrial development which adds less than 10,000 square feet of impervious surface area.

2. Any development which results in less than a 2.5 cubic feet per second increase in the ten year storm peak discharge from the developed site, provided that the City Engineer may require an analysis of the downstream drainage facilities to determine hydraulic capacities of affected drainage systems. The City Engineer will require an analysis of the downstream system when the following conditions exist:

   (a) There is a history of flooding in the vicinity of the proposed development site, or there is evidence of undersized drainage facilities, or

   (b) The City Engineer has inspected the downstream drainage facilities and needs additional information to determine the capacities of those facilities.

3. Any development or construction where, based on a competent engineering study prepared by a registered professional engineer, satisfies all the following criteria:

   (a) The standard requirement in Section 2.1.1(2) is satisfied and particularly that the increase in runoff and peak discharge from the development shall not increase the degree of flooding in its vicinity or in other areas;

   (b) The increase in runoff and peak discharge from the development shall not cause an increase in erosion or sedimentation;

   (c) All downstream drainage systems and facilities are adequate to handle the increase in runoff and peak discharge from the development considering total development of the watershed. The design storm used for determining the increase in runoff and peak discharge will be in conformance with the design criteria in Chapter 3.

If paragraph 3(a) and 3(b), are met and 3(c) is not met, then the City Engineer will determine the allowable increase in peak discharge from the proposed development based on the following equation:

$$Qa = \frac{Qc - Qp}{At} (Ap)$$

Qa = allowable increase in peak discharge, cubic feet per second
Qc = hydraulic capacity of downstream system, cubic feet per second
Qp = present condition (pre-development) peak discharge, cubic feet per second
At = total watershed drainage area, acres
Ap = drainage area of proposed development, acres

4. Construction, or repair of, or addition to, any residential dwelling located on single lots within an approved subdivision.

5. Construction, or repair of, or addition to, any commercial or industrial buildings or facilities including parking areas located within an approved subdivision where an existing stormwater detention or retention facility has been designed and constructed to meet the requirements of this manual.

Amend Section 9.2.3 to read as follows:

9.2.3 GRADING AND DEPTH

To obtain sufficient storage volume, the construction of dry detention/retention facilities usually requires either excavation or the placement of earthen embankments. If vegetated embankments are used, they shall be less than 10 feet in height and shall be placed with side slopes no steeper than 3:1 (horizontal:vertical). Slopes for riprapped earthen embankments should be no steeper than 3:1. A slope stability evaluation is generally appropriate for embankment configurations with impoundment depths greater than 3 feet.
The maximum depth of stormwater detention/retention facilities should be consistent with safety and aesthetic considerations for the system. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage the growth of attached weeds (without creating undue potential for anaerobic bottom water) should be considered. A depth of from 6 to 8 feet is generally reasonable.

Other considerations when setting detention/retention facility depths include flood elevation requirements, public safety, land availability, land values, present and future land use, water table fluctuations, soil profile characteristics, maintenance, and freeboard. Aesthetically pleasing features are also an important consideration and can often be provided with little extra cost or loss of capacity. A minimum freeboard of 1 foot above the high water elevation of the design flood shall be provided, with higher values considered when addressing special concerns.

Amend Section 9.7 to read as follows:

9.7 PROTECTIVE TREATMENT

Protective treatment may be required to prevent entry to facilities that present a public safety hazard. A six (6) foot high security type chain link fence shall be provided for detention areas where one or more of the following conditions exist:

1. Rapid stage changes that would make escape practically impossible for small children

2. Water depths that either exceed 2.5 feet for more than 24 hours or are permanently wet and have side slopes steeper than 4:1 (horizontal:vertical)

3. Passage through the detention area of a low-flow watercourse or ditch with a depth greater than 5 feet or a flow velocity greater than 5 feet per second

4. Side slopes that equal or exceed 1.5:1 (horizontal:vertical)

5. Depth that equals or exceeds 3 feet and maintenance will be the responsibility of the City of Huntsville

Guards or grates may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

Fencing should be considered for dry retention areas with design depths in excess of 2.5 feet for 24 hours, unless the area is within a fenced, limited access facility.

SUMMARY

The Huntsville Stormwater Management Manual (SMM) establishes policy, permitting requirements, design regulations, and technical information for stormwater activities in the City of Hunts-ville. The manual contains ten chapters, presented in two parts as listed below:

PART 1 - POLICY, PERMITTING, AND REGULATIONS

1. Introduction
2. Policy and Permitting
3. Regulations for Facility Design and Flood Plains

PART 2 - TECHNICAL INFORMATION

4. Hydrology
5. Open Channel Hydraulics
6. Culvert Hydraulics
7. Gutter and Inlet Hydraulics
8. Storm Sewer Hydraulics
9. Storage System Hydraulics
10. Erosion and Sediment Control

The regulatory chapters in Part 1 are of particular interest to developers, builders, realtors, landowners, architects, and engineers, while the technical chapters in Part 2 are oriented to the concerns of designers and builders.

The Huntsville SMM also includes two appendices that provide copies of City ordinances relevant to stormwater management. Additional material that is appended to the manual but bound separately includes the standard detail drawings and the Stormwater Management Ordinance. The flood study reports and maps are briefly discussed in Chapter 3. The appended material is available from the Department of Public Works.

Tables and figures are located at the end of each chapter, with tables preceding figures. Each chapter is introduced by a table of contents that includes table and figure

titles. In the text, equation terms and symbols are defined immediately following each equation. All references are listed at the end of the manual.

Example problems are provided at the end of selected sections in the manual to demonstrate the application of procedures. A summary list of the example problems follows:

Problem	Title	Page No.
4-1	Rainfall Excess Using the Rational Method Runoff Coefficient	4-11
4-2	Rainfall Excess Using the SCS Curve Number	4-13
4-3	Time of Concentration Computation	4-17
4-4	Rational Method Peak Runoff Rate	4-26
4-5	TVA Regression Equation Peak Runoff Rate	4-27
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4-7	SCS Dimensionless Unit Hydrograph	4-37
4-8	Flood Hydrograph Using Unit Hydrograph Theory	4-38
5-1	Direct Solution of Manning's Equation	5-21
5-2	Grassed Channel Design Stability	5-22
5-3	Grassed Channel Design Capacity	5-23
5-4	Grassed Channel Vegetation Comparison	5-25
5-5	Riprap Design	5-28
6-1	Circular Concrete Culvert	6-18
9-1	Reservoir Routing Using Storage Indication Method	9-12

The contents of each chapter are briefly summarized below. Additional assistance for locating information can be found in the detailed table of contents presented at the front of each chapter.

PART 1 - POLICY, PERMITTING, AND REGULATIONS

Chapter 1 - Introduction

This chapter contains the following information:

Topic	Content
1. Purpose	Five items listed
2. Source of Procedures	Readily available government publications

Topic	Content
3. Limitations	Engineering judgment required
4. Updating	Available from the Department of Public Works

Chapter 2 - Policy and Permitting

This chapter contains the following regulatory information:

Topic	Content
1. Building and Grading Permit Requirements	Building permits from Inspection Department/grading permits from Department of Public Works
2. Grading Permit Exemptions	
Excavation of Fill	Specifies when a permit is not required based on six criteria
Agricultural Practices	Plowing, cultivating, etc., are exempt
Maintenance Grading	Specifies when a permit is not required based on four criteria
Public Utilities	No permit required; submit documents for review only
Department of Public Works Projects	No permit required
3. Application Procedures	
Number of Copies	Three required
Required Information	Information required by Department of Public Works is listed
Posting	Post permit until Certificate of Occupancy issued

Topic	Content
Effect	Prevents violations, requires error corrections
Other Permits	Responsibility of applicant
4. Enforcement	
Right of Entry	Authority for Engineer of Public Works
Revocation	Based on false information
Stop Work Order	For permit violation
Corrective Measures	For non-permitted activities
Penalties & Injunctions	Fines and other remedies
5. Construction Inspections	Engineer of Public Works authority
6. As-Built Certification	Requires "as-built" drawings

Chapter 3 - Regulations

This chapter contains the following regulatory information:

Topic	Content
1. Design Criteria Summary	
Return Periods	Specifies design storm to use
Manning's n Values	Specifies n values to use
2. Submittal Requirements	
Hydrologic Procedures	Calculations and documentation required
Open Channels	Requirements for open ditches
Culverts & Storm Sewers	Information required on plans

Topic	Content
Gutters & Inlets	Required intercept, spread, and spacing
Storage System	When and when not required
Erosion and sediment Control	When and when not required; information required on plans
3. Flood Elevation and Floodway Data	
FEMA Floodway Maps	Established FEMA flood elevation and floodway limits
Unpublished Data	Requirements for preparing new flood information
4. Flood Plain Requirements	
Zoning Ordinance	Floodway, Floodway Fringe regulations (see Appendix A)
Activities within Flood Plains	Restrictions for flood hazard areas
Subdivisions	Development requirements for flood plain areas identified
5. Inadequate Facilities	Requirements before new development

Topic	Content
6. Traffic-Safe Drainage Facilities	
Cross-Drainage Structures	Transversable design, structure extensions, or shielding required
Parallel Drainage Features	Structure elimination, transversable design, lateral relocation, or shielding required
Drop Inlets	On-roadway and off-roadway structure requirements

PART 2 - TECHNICAL INFORMATION

Chapter 4 - Hydrology

This chapter contains the following technical information:

Topic	Content
1. Procedure Selection	Describes six hydrologic procedures; guidelines in Table 4-1
2. Hyetographs	Development of rainfall increments
Design Storm Data	Intensity-duration-frequency curves (Figure 4-1)
SCS Type II Storm	Presented in Figure 4-2
Balanced Storm	Illustrated in Figure 4-3
3. Rainfall Excess	Converts rainfall to runoff quantities
Runoff Coefficients	Use Tables 4-3 and 4-4
Curve Numbers	Use Tables 4-5 and 4-6

Topic	Content
4. Time of Concentration	Specifies how to calculate time of concentration
Overland Flow	Use Figure 4-4
Shallow Channel Flow	Use Figure 4-5 or Manning's Equation
Main Channel Flow	Use Manning's Equation
5. Peak Runoff Rates	Methods to determine peak runoff
Gaged Sites	See Table 4-8
Rational Method	Use five-step procedure
TVA Regression Equations	Use Tables 4-9 and 4-10 and Figures 4-6 through 4-10
TVA Graphs	Use Table 4-11 and Figures 4-11 through 4-33
SCS Graphical Method	Ten limitations and six steps
Other Techniques	Subject to approval
6. Flood Hydrographs	Method for determining a flood hydrograph
Hydrograph Terminology	See Figure 4-35
Unit Hydrograph Theory	Five assumptions
Unit Hydrograph Procedure	Six steps
Synthetic Unit Hydrographs	SCS curvilinear or triangular; see Figure 4-37
SCS TR-55 Tabular Method	Limitations only; obtain original publication for procedure details

Topic	Content
7. Hydrologic Channel Routing	
Muskingum Method	Eight steps
SCS TR-55 Tabular Method	Limitations only; obtain original publication for procedure details

Chapter 5 - Open Channel Hydraulics

This chapter contains the following technical information:

Topic	Content
1. Linings	Four main classifications
Vegetative	Acceptable conditions identified
Flexible	Use rock riprap or rubble
Rigid	Use concrete or grouted riprap
Temporary	Protection required until permanent measures are established
2. Design Criteria	Open channel design values
Return Period	Use 25-year storm
Velocity Limitations	Use Tables 5-1 and 5-2
Manning's n Values	Use Table 5-3, Figure 5-2, and Equation 5-1
Side Slopes	Maximum values are specified
Riprap Lining	Guidelines for gradation, thickness, and filter material
3. Uniform Flow Calculations	Several forms of Manning's Equation
Geometric Relationships	Use Figure 5-2

Topic	Content
Direct Solutions	Tables, charts, and nomographs
Trial and Error Solutions	Equations and figures
Critical Flow Calculations	Equations and guidelines
Vegetative Design	Use two parts: design stability and design capacity
Riprap Design	Equations and figures
4. Gradually Varied Flow	General guidance
5. Rapidly Varied Flow	Conditions are identified
Hydraulic Jump	Rectangular channel procedure
Bridges	General guidance

Chapter 6 - Culvert Hydraulics

This chapter contains the following technical information:

Topic	Content
1. Fundamentals	
Inlet Control	Four variables identified
Inlet Control	Four variables identified
Outlet Control	Six variables identified
Culvert Selection	Trial and error process
2. Design Criteria	Specifies design restrictions and limitations on culverts
Discharge	Specifies return periods for five categories

Topic	Content
Headwater	Headwater restrictions and limitations
Tailwater	Four typical conditions
Length & Slope	Three factors to consider
Velocity Limitations	Minimum of 2.5 feet/second; for maximum, see Chapter 5
3. Design Calculations	Procedures for design of culverts
General Procedure	See Figure 6-1
Inlet Control	Culvert capacity; three types of calculations
Outlet Control	Three design steps to obtain culvert capacity

Chapter 7 - Gutter and Inlet Hydraulics

This chapter contains the following technical information:

Topic	Content
1. Design Criteria	
Return Period	Specifies design storm for pavement drainage
Spread	10-foot maximum
Inlet Spacing	Based on 80 percent intercept
Longitudinal Slopes	Range 0.5 to 12 percent
Cross Slope	Maximum 2 percent desirable
Combination Sections	Width of 2 or 2.5 feet

Topic	Content
Roadside Ditches	Department of Public Works approval required
Bridge Decks	Intercept flow before bridge
2. Gutter Flow Calculations	Equation used to calculate gutter flow
Uniform Cross Slopes	Use Figure 7-1
Standard Sections	Use Table 7-2
Composite Sections	Use Figures 7-1 and 7-2
3. Curb-Opening Inlets	Procedures to determine capacity
Type S	Use Table 7-3
General--Continuous Grade	Use Figures 7-3 and 7-4
Sump Conditions	Use Figures 7-5 and 7-6
4. Grate Inlets	Procedures to determine capacity
5. Combination Inlets	Procedures to determine capacity
Continuous	Capacity based on grate inlet
Sump	Capacity based on curb opening

Chapter 8 - Storm Sewer Hydraulics

This chapter contains the following technical information:

Topic	Content
1. Design Criteria	
Return Periods	Specifies design storm to use

Topic	Content
Manning's n Values	Specifies n values to use
Slope & Hydraulic Gradient	Velocity range 2.5 to 20 feet/second
Pipe Size & Length	Minimum pipe size and inlet spacing requirements
Clearances	Minimum clearance criteria for utilities and roads
Inlets	Bypass of 20 percent allowed
2. General Procedure	Iteration by four steps
3. Hydrologic Calculations	Consistent with Chapter 4
4. Hydraulic Calculations	
Pressure vs. Open Channel	Guidance in Figure 8-3
Energy Losses	Friction, pipe form, and junction losses to be considered
Open Channel	Use Manning's Equation
Pressure	Use Energy Equation

Chapter 9 - Storage System Hydraulics

This chapter contains the following technical information:

Topic	Content
1. Terminology	Retention/detention definitions
2. Design Criteria	Specifies design requirements
Release Rate	Limitations presented
Detention Volume	Based on routing calculations

Topic	Content
Grading & Depth	Requirements and considerations presented
Outlet Works	Design requirements for principal and emergency outlets
Wet Ponds	Bottom sealing and side slope requirements
3. General Procedure	Three relationships for sizing facilities presented as seven steps
4. Outlet Hydraulics	Capacities of outlet structures
Sharp-Crested--No End Contractions	Use Equation 9-1
Sharp-Crested--Two End Contractions	Use Equation 9-2
Sharp-Crested--Submerged	Use Equation 9-3
Broad-Crested	Use Equation 9-4
V-Notch	Use Equation 9-5
Orifices	Use Equation 9-6
5. Preliminary Calculations	
Storage Volume	Use Equation 9-7 for estimating storage volume
Peak Flow	Use Equation 9-8 for estimating peak outflow
6. Routing Calculations	
Storage Indication Method	Use Equation 9-9 for final design calculations
7. Protective Treatment	Requirements for fencing and safety constraints

Chapter 10 - Erosion and Sediment Control

This chapter contains the following technical information:

Topic	Content
1. Erosion & Sediment Control Plans	
Submittal Requirements	Seven requirements for plans
Basic Principles	Five basic principles
2. Erosion Control	Types and uses specified
Temporary vs. Permanent	Consistent layout for both types
Surface Stabilization	Required when land is cleared or existing conditions altered
Runoff Control	Required when activities increase or change runoff
Exposure Scheduling	Goal to provide stabilization before heavy rainfall
3. Sediment Control	Types and uses are specified
Straw Bale Barriers	Acceptable conditions for application are specified
Traps	Sketches are presented in Figure 10-3
Basins	Components are illustrated in Figure 10-4
Approximate Basin Sizing	Use Equations 10-1 and 10-2
Construction Entrances	Stabilization required at all points of access

CHAPTER 1
Introduction

CHAPTER 1
Introduction

PART 1
Policy, Permitting, and Regulations

PART 1 - POLICY, PERMITTING, AND REGULATIONS

Part 1 of the Huntsville Stormwater Management Manual will be of interest to developers, builders, realtors, landowners, architects, and engineers who need to be familiar with the regulations associated with land development or alterations to existing facilities. The following chapters are included in Part 1:

1. Introduction
2. Policy and Permitting
3. Regulations for Facility Design and Flood Plains

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Chapter 1
INTRODUCTION

1.1 PURPOSE

The purpose of the Stormwater Management Manual (SMM) is to establish regulations and provide stormwater design information for local agencies, engineers, developers, or others whose activities concern stormwater management in the City of Huntsville. The manual has been prepared to document the following items:

1. Uniform stormwater design criteria and procedures
2. Regulations for grading and building permits
3. Regulations and policies for land disturbance activities
4. Technical information to assist with the design of stormwater drainage systems
5. Published data applicable to the City of Huntsville

This manual should be used to guide persons involved in the activities mentioned above through the required procedures to obtain approval from the Engineer of Public Works for grading, building, and construction permits, and subdivision development. Zoning and Building Code requirements are the responsibility of the City of Huntsville Planning Department and Inspection Department, as appropriate. The requirements contained in this manual are in addition to Zoning and Building Code requirements. Any conflicts arising from the implementation of the requirements of this manual with enforcement of Zoning and Building Code requirements shall be resolved by appropriate methods and procedures within the City of Huntsville.

1.2 SOURCE OF PROCEDURES

Procedures identified in this manual are generally found in readily available government publications. Information from these publications has been duplicated as required to supplement contents of the manual. Complete details and additional information on the procedures can be obtained from

tional information on the procedures can be obtained from the original references.

1.3 LIMITATIONS

This manual neither replaces the need for engineering judgement nor precludes the use of information not presented. The emphasis is on desktop procedures that can be performed manually. To ensure that procedures are properly applied and adapted, the engineer must consider actual site conditions and project requirements.

1.4 UPDATING

The Huntsville SMM will be updated and revised, as necessary, to reflect up-to-date engineering practices and information applicable to Huntsville. The status and details of any updates or revisions can be obtained from the Department of Public Works. Questions and any suggestions for changes should also be addressed to the Department of Public Works.

CHAPTER 2
Policy and Permitting

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Chapter 2
POLICY AND PERMITTING

2.1 OBJECTIVES

The City of Huntsville's stormwater management program has the following objectives:

1. Protect human life and health.

2. Reduce expenditure of public money for flood control projects.

3. Reduce the need for rescue and relief efforts associated with flooding.

4. Provide for the sound use and development of flood-prone areas so as to maximize beneficial use without increasing flood hazard potential.

5. Reduce damage to public facilities and utilities such as water and sewer lines; electric, telephone, and gas facilities; and streets and bridges located in flood plains.

6. Ensure a functional stormwater drainage system that will not result in excessive maintenance costs.

7. Encourage the use of natural and aesthetically pleasing design.

8. Encourage the improvement of existing flooding problems in conjunction with new development.

9. Reduce the impact on public and private property caused by the accumulation of mud, dirt, water, debris, and other construction materials.

2.1.1 STANDARDS

To implement the objectives presented above, the following standards shall apply:

1. The calculated peak rate of stormwater runoff resulting from a ten-year return period, twenty-four (24) hour duration, type II storm distribution (as defined by the Soil Conservation Service, U.S. Department of Agriculture) shall be no greater after development of a site than before development of a site.

2. No construction, whether by private or public action, shall be performed in such a manner as to materially increase the degree of flooding in its vicinity or in other areas whether by flow restrictions, increased runoff, or by diminishing channel or overbank storage capacity.

2.2 PERMIT REQUIREMENTS

A grading permit issued by the Engineer of Public Works or approval by the Engineer of Public Works of a building permit will be required before any land disturbance activity can begin, unless the disturbance is exempted by the provisions in Section 2.3.

2.2.1 BUILDING PERMITS

Building permits are issued by the City of Huntsville Inspection Department. All improvements, repairs, or construction with respect to structures within the City of Huntsville that require a permit from the Inspection Department shall be approved by the Engineer of Public Works, unless the structure falls into one of the following categories:

1. Residential dwellings on single lots within an approved subdivision

2. Remodeling within the walls of an existing structure

The information and data that shall be furnished to the Engineer of Public Works is listed in Section 2.4.2.

2.2.2 GRADING PERMITS

Grading permits are required from the Engineer of Public

Works before the commencement of any land disturbance activity; including grading, excavating, clearing, or filling, within the City of Huntsville unless exempted by the provisions of Section 2.3. Exemption shall not be construed as relieving the responsible party from other requirements within these regulations and/or flood plain requirements (see Chapter 3, Section 3.4). Approval of or exemption from grading permit requirements shall not be construed as relieving the responsible party from the requirements of any other applicable federal, state or local laws.

2.2.3 ALTERATIONS TO EXISTING DRAINAGE

A letter of approval from the Engineer of Public Works shall be required for any alterations to existing drainage facilities maintained or proposed for maintenance by the City of Huntsville involving the removal or installation of underground pipe, including culverts, and construction or alteration of drainage ditches. No letter of approval will be required for alterations to private drainage systems with tributary areas less than 5000 square feet; however, alterations to those private drainage systems with tributary areas equal to or greater than 5000 square feet shall meet the requirements of this manual. The exception regarding alterations to private drainage systems with tributary areas less than 5000 square feet does not imply that the work has the approval of the City of Huntsville, and does not relieve the responsible party from the requirement of any other applicable federal, state, or local laws.

2.3 GRADING PERMIT EXEMPTIONS

No grading permit is required for the construction, improvement, or repair of any structure within a development for which an approved grading, drainage, and erosion control plan exists. However, any alteration to the original grading, drainage, or erosion control plan shall require approval by the Engineer of Public Works.

Any person who disturbs the natural ground cover in an area for which there is an approved grading, drainage, and erosion control plan shall comply with the requirements of such plan without exception.

Specific activities which may be conducted without obtaining

a grading permit are identified below. These exemptions shall not be construed as relieving the party responsible for the identified activities from making onsite drainage improvements that may be required or from compliance with flood plain requirements of Ordinance No. 87-269, as amended which are summarized in Chapter 3.

2.3.1 EXCAVATION OR FILL

No grading permit is required for an excavation or fill that satisfies all of the following criteria:

1. Is less than 4 feet in vertical depth at its deepest point as measured from the natural ground.

2. Will not result in a total quantity of more than 200 cubic yards of material being removed from, deposited on, or disturbed on any lot, parcel, or subdivision thereof.

3. Will not impair existing surface drainage, constitute a potential erosion hazard, or act as a source of sedimentation to any adjacent land or watercourse. This condition requires that vegetative cover be re-established immediately on all disturbed areas.

4. Will have no fill placed on a surface having a slope steeper than 5 feet horizontal to 1 foot vertical.

5. Will have no final slopes cut steeper than 3 feet horizontal to 1 foot vertical.

6. Will not contain hazardous substances.

In addition, the following activities do not require a grading permit:

1. Burial within approved human or animal cemeteries.

2. Construction of accessory structures related to single-family residences or duplex structures authorized by a building permit, provided the disturbed material or fill is handled in such a manner as to conform to the approved erosion control plan for the area or, where no such ero-

sion control plan is in effect, that such work is performed in conformance with the basic principles of erosion and sedimentation control as contained in Chapter 10, Section 10.1.2 of this manual.

2.3.2 AGRICULTURAL PRACTICES

No grading permit is required for agricultural land management practices approved by the U.S. Department of Agriculture (USDA), Soil Conservation Service (SCS), as long as the conditions of Section 2.3.1 are satisfied and existing drainage is not rerouted. Typical accepted agricultural practices include plowing, cultivation, nursery operations such as the removal of or transplanting of cultivated sod and trees, and tree cuttings at or above existing ground level that leave the stump, ground cover, and root mat intact.

2.3.3 MAINTENANCE GRADING

No grading permit is required for grading as a maintenance measure, or for landscaping on developed lots or parcels with existing structures, provided all of the following criteria are met:

1. The aggregate area affected or stripped at any one time does not exceed 10,000 square feet and is not within a natural drainageway (e.g., designated flood plain).

2. The grade change does not exceed 18 inches at any point and does not alter the direction of the drainage flow path.

3. Proper vegetative cover is re-established as soon as possible on all disturbed areas.

4. The grading does not involve a quantity of material in excess of 200 cubic yards.

2.3.4 PUBLIC UTILITIES

No grading permit is required for installation of lateral sewer lines, telephone lines, electrical lines, gas lines, or other public service facilities. Although exempt, public agencies installing such lines or other public service

facilities shall comply with the applicable standards contained in this manual. Public agencies shall submit documents to the Engineer of Public Works for consistency reviews and to allow coordination with other activities.

2.3.5 DEPARTMENT OF PUBLIC WORKS PROJECTS

No grading permit is required for projects performed by the Department of Public Works, including facilities maintenance, right-of-way projects, and storm sewer or street construction. The City of Huntsville shall design, construct and maintain all appropriate public works projects in accordance with the applicable standards contained in this manual.

2.4 GRADING AND BUILDING PERMIT APPLICATION PROCEDURES

Flow charts for the processing of building and grading permit applications and for obtaining subdivision approvals are presented in Figures 2-1, 2-2, and 2-3, respectively. Additional details of application procedures are presented below.

2.4.1 NUMBER OF COPIES

Each application for a grading or building permit for which approval of the Engineer of Public Works is required shall include three (3) copies of plans showing information required under Section 2.4.2 for review of the proposed work by the Engineer of Public Works. Plans shall be certified by a registered engineer, landscape architect, or land surveyor, as appropriate.

2.4.2 REQUIRED INFORMATION

A complete application package shall include information on the following items:

1. A complete plan of the proposed development at a scale no less than 1 inch = 100 feet. This plan is to include existing and proposed contours at intervals no greater than 2 feet with at least one benchmark required, including the elevation relative to National Geodetic Vertical Datum (NGVD).

Contours shall extend to the centerline of all roads/streets bordering the site.

2. Existing and proposed buildings, impervious surfaces, easements, right-of way land, and drainage structures, including inlets, catch basins, manholes, junction boxes, driveway pipes, culverts, cross drains, headwalls, outlet facilities, and sanitary sewers, with size, type, slope, invert elevations, and quantity indicated.

3. If a project will be constructed in phases, common facilities and phasing schedules.

4. Hydrologic and hydraulic calculations for appropriate design conditions and facilities, including detention/retention facilities and reservoir routing calculations as required by Chapter 9 of this manual.

5. Any proposed ditches or channel improvements, with typical section and proposed limits of change indicated.

6. Any high water marks or flood lines, either calculated or observed in the vicinity of the proposed development, and the source of said line or elevation indicated.

7. All fill areas indicated as such, with the limits and elevation indicated.

8. Drainage arrows indicating the existing and proposed direction of runoff throughout the plan.

9. Invert and top of grate elevations on all catch basins and inlets in addition to flow line elevations, stations, and percent grades of all cross drains and pipe between inlets and catch basins.

10. For flood plain areas, designated floodway along with regulatory flood elevations, cut and fill cross sections and calculations, and lowest floor elevations for buildings in the flood plain. Hydraulic calculations should be submitted, when appropriate.

11. Temporary erosion and sedimentation control measures (straw bales, silt fence, etc.) to be implemented during construction, as required by Chapter 10 of this manual.

12. Final stabilization measures proposed for all disturbed areas on the property. Areas with slopes equal to or greater than two feet horizontal to one foot vertical shall be stabilized with riprap or another equally effective stabilization measure. Stabilization for each ditch shall be shown.

13. Where special structures such as box culverts, bridges, or junction boxes are proposed, detail plans showing dimensions, reinforcement, spacing, sections, elevations, and other pertinent information.

14. A summary of the quantities of materials to be used and an estimate of the cost of construction and completion of improvements required by this manual (for which maintenance of such improvements by the City is required or anticipated) shall be provided so as to determine the amount of the bond required under Section 2.5.6 herein.

15. Plans and calculations signed and sealed by a registered engineer, landscape architect, or land surveyor, if appropriate, if application is for a grading permit. If application is for a building permit, signature and seal of a registered engineer is required.

Additional information required for various facility components is covered in Chapter 3, Section 3.2.

2.4.3 POSTING

Work requiring a building or grading permit shall not be commenced until the permit holder or his agent has posted the permit in a conspicuous place on the front of the premises. The permit shall be protected from the weather and be placed to allow easy access for recording entries. The building permit shall remain posted by the permit holder

until the certificate of occupancy has been issued by the Inspection Department. The grading permit shall remain posted by the permit holder until the work has been approved by the Engineer of Public Works.

2.4.4 EFFECT

Grading permits issued by the Engineer of Public Works and building permits approved by the Engineer of Public Works and issued by the City of Huntsville Inspection Department shall be construed to be a license to proceed with the work governed by this manual and shall not be construed as authority to violate, cancel, alter, or set aside any of the provisions of the permit, nor shall issuance of a permit prevent the Engineer of Public Works from thereafter requiring a correction of errors in plans, errors in construction or violations of these regulations.

2.4.5 TIME LIMITS

Unless the work governed by this manual and authorized by a grading or building permit is commenced within 6 months after the date the permit was issued, the permit shall become invalid and a new permit shall be required. If the work authorized by such permit is not completed in accordance with approved timing schedules, the permit, with respect to these regulations, shall be invalid; however, for just and reasonable cause, one extension for a period not exceeding ninety (90) days may be allowed. Requests for such a time extension shall be submitted in writing to the Engineer of Public Works, and authorization for such a time extension shall be in writing. This requirement does not replace, alter, or change any other requirements or regulations of the Inspection Department.

2.4.6 OTHER PERMITS

Approval by the Engineer of Public Works does not relieve the applicant of responsibility for obtaining any other required permits.

2.5 ENFORCEMENT

2.5.1 RIGHT OF ENTRY

The Engineer of Public Works, or any duly authorized repre-

sentatives, may enter upon the premises of any land within the City of Huntsville for which an application for either a grading or building permit has been filed for the purpose of inspecting the site before, during, and upon conclusion of construction to or other land disturbance activity to determine compliance with the requirements of this manual and the grading or building permit. The acceptance of a grading or building permit shall be construed as permission by the applicant for entry upon the premises for such inspections.

2.5.2 REVOCATION

The Engineer of Public Works may revoke an approved grading permit when informed of any false statement or misrepresentation of facts in the application or plans on which the approval is based. The Engineer of Public Works may request the revocation of an approved building permit by the Inspection Department when informed of any false statement or misrepresentation of facts in the application or plans on which approval is based. The applicant may appeal the revocation of an approved grading permit in accordance with the procedures outlined in Section 2.5.7.

2.5.3 STOP WORK ORDER

Upon notice from the Engineer of Public Works, work governed by this manual which is being performed on any site within the City of Huntsville without an approved grading or building permit, in violation of the conditions for an approved grading or building permit, or in a dangerous or unsafe manner, shall be stopped immediately. Such notice shall be in writing, shall be delivered to the owner of the property, his agent, or the person doing the work, and shall state the conditions under which the work may be resumed. The applicant may appeal the stop work order in accordance with the procedures outlined in Section 2.5.7.

2.5.4 CORRECTIVE MEASURES

Any non-permitted drainage system, construction, or fill located within a flood plain shall, upon written notice from the Engineer of Public Works, be removed at the property owner's expense. Where any violation is discovered, the responsible party shall restore the land to its state prior to the violation. The applicant may appeal the required

corrective measures in accordance with the procedures outlined in Section 2.5.7.

2.5.5 PENALTIES AND INJUNCTIONS

Any violation of these regulations shall be punishable by a fine of not more than $500.00 for each and every violation. Each day that a violation remains uncorrected shall be considered a separate offense.

In addition to all other remedies provided by law, the City of Huntsville shall have the right to injunctive relief for any violations of the requirements of this manual.

2.5.6 BOND REQUIREMENTS

The City of Huntsville shall require the providing of a performance bond to guarantee the proper construction and completion of improvements required by this manual for which maintenance of such improvements by the City is required or anticipated. The amount of the bond shall be based on an estimated cost of the required improvements provided by the applicant and approved by the Engineer of Public Works. Bond requirements may be fulfilled by the bond submitted to obtain subdivision approval, provided improvements required by this manual were considered in determining the subdivision bond amount. The bond shall be approved by the Legal Department as to form, sufficiency and manner of execution and shall be (1) a bond secured by an insurance company licensed by the State of Alabama, or (2) a cash bond or (3) an irrevocable letter of credit from an approved lending institution. Bond is not normally required for improvements which shall not be maintained by the City of Huntsville.

2.5.7 APPEALS

An applicant for a grading or building permit may appeal any decision of the Engineer of Public Works with respect to this manual. A notice of appeal must be in writing, specifying the grounds thereof, and must be filed with the Engineer of Public Works and the Stormwater Management Board within 15 days after such decision of the Engineer of Public Works. A hearing will be scheduled and the appellant given opportunity to present his appeal. The Engineer of Public Works shall also be granted the opportunity to present his

findings. Every decision of the Stormwater Management Board shall be final, subject to such remedy as any aggrieved party may have at law or in equity.

2.6 CONSTRUCTION INSPECTIONS

The Engineer of Public Works, at his discretion, shall inspect or cause to be inspected, at various intervals, all construction or grading for which a grading permit has been issued or a building permit has been approved. The Engineer of Public Works may direct that additional testing and inspection be performed. If such additional testing and inspection reveal that the construction and materials conform to the permit requirements, the costs of such additional testing and inspection shall be paid by the City of Huntsville. If such additional testing and inspection reveal that the construction and materials do not conform to the permit requirements, the costs of such additional testing and inspection shall be paid by the permit holder. All nonconforming construction and material shall be removed and replaced with conforming construction and material at no cost or obligation to the City of Huntsville.

2.7 AS-BUILT CERTIFICATION

The final step in the permit process under this manual involves as-built certification. As-built certification from a registered engineer, landscape architect, or land surveyor, as appropriate, shall be required, and the certification shall include the following items, as appropriate:

1. A letter certifying that drainage facilities, erosion and sedimentation control measures, and other improvements are complete and functional as required by the appropriate regulations and the approved permit.

2. Record drawings showing the final grades, elevations, and location of facilities.

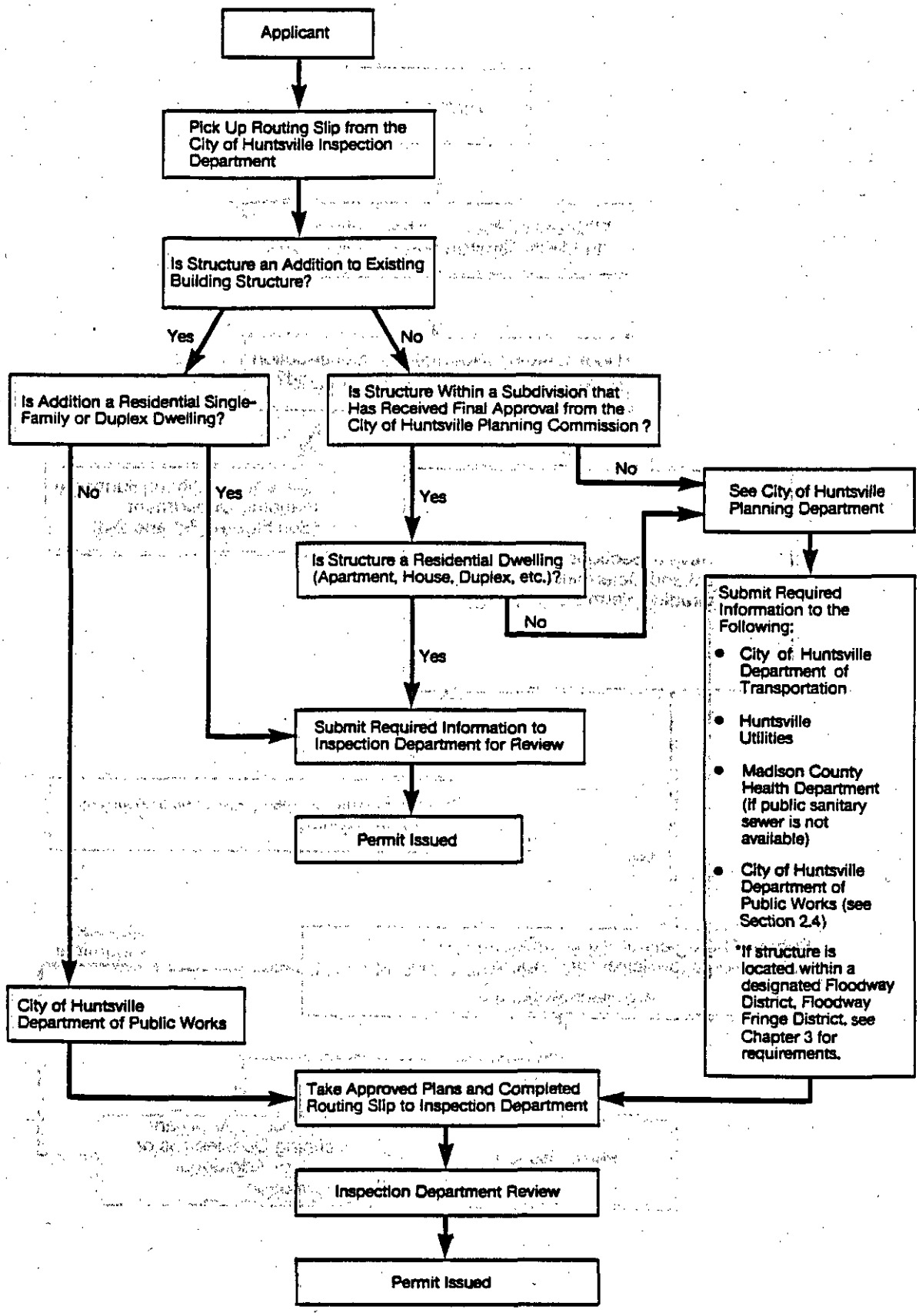


FIGURE 2-1
Building Permit Application Procedure

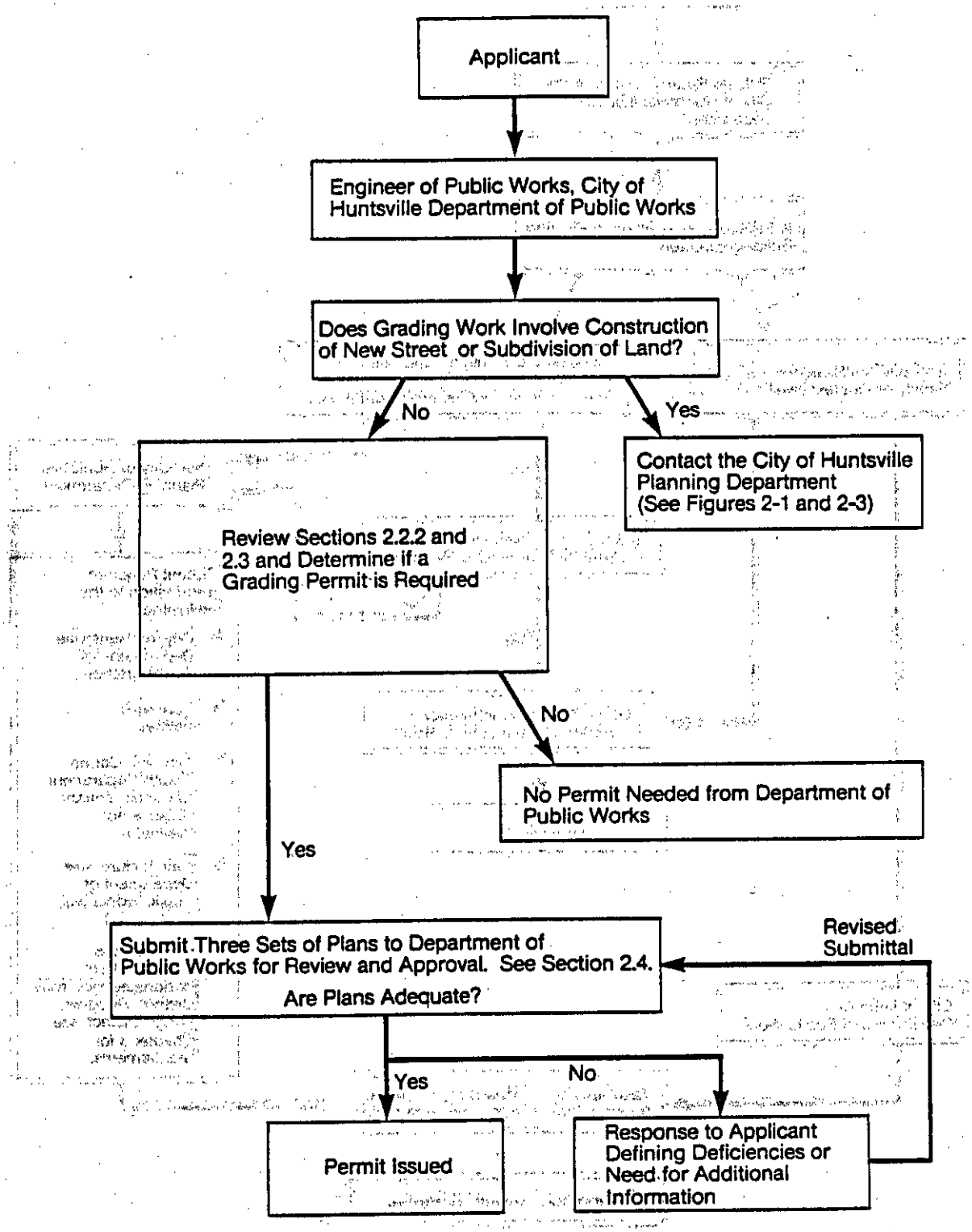
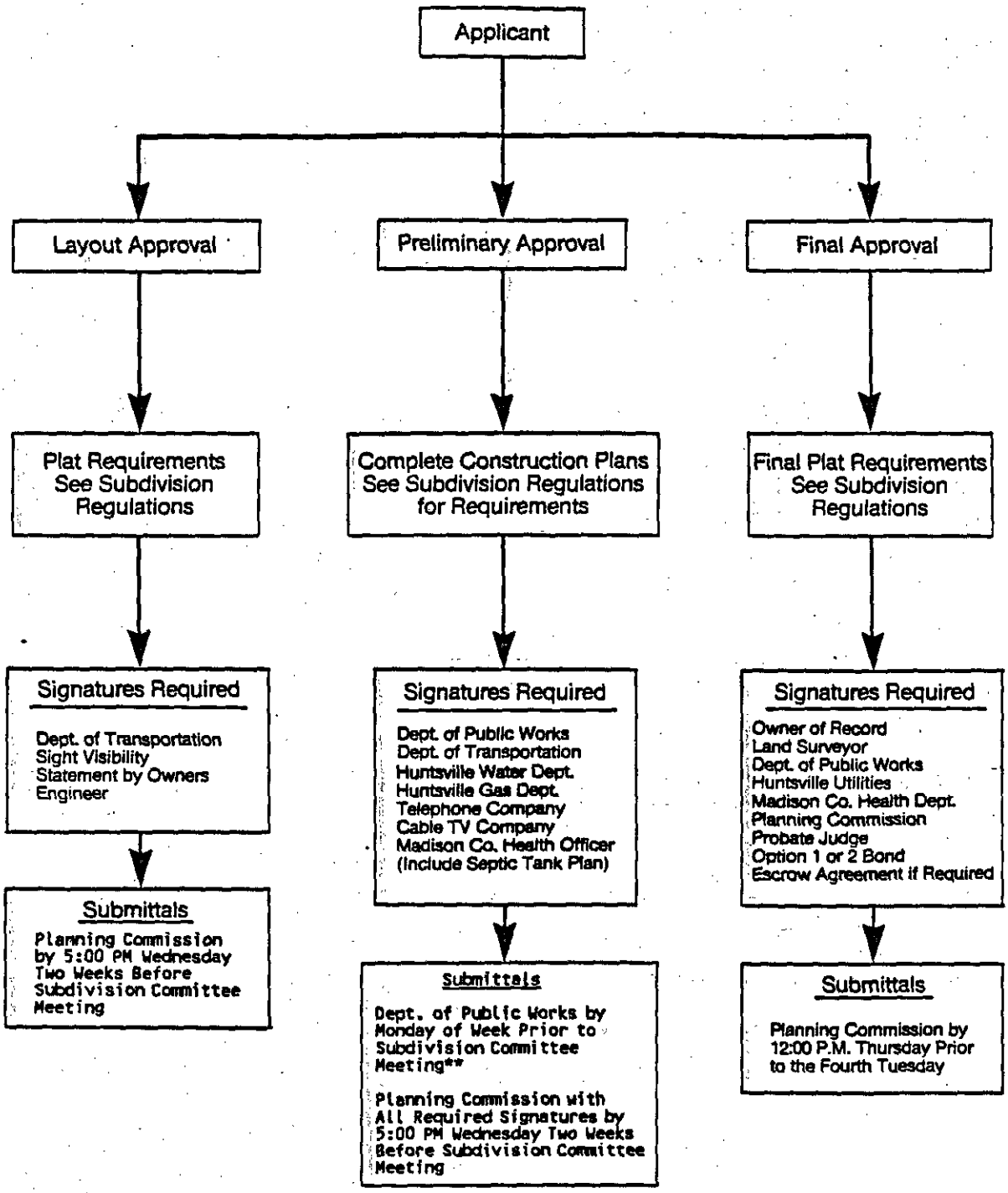
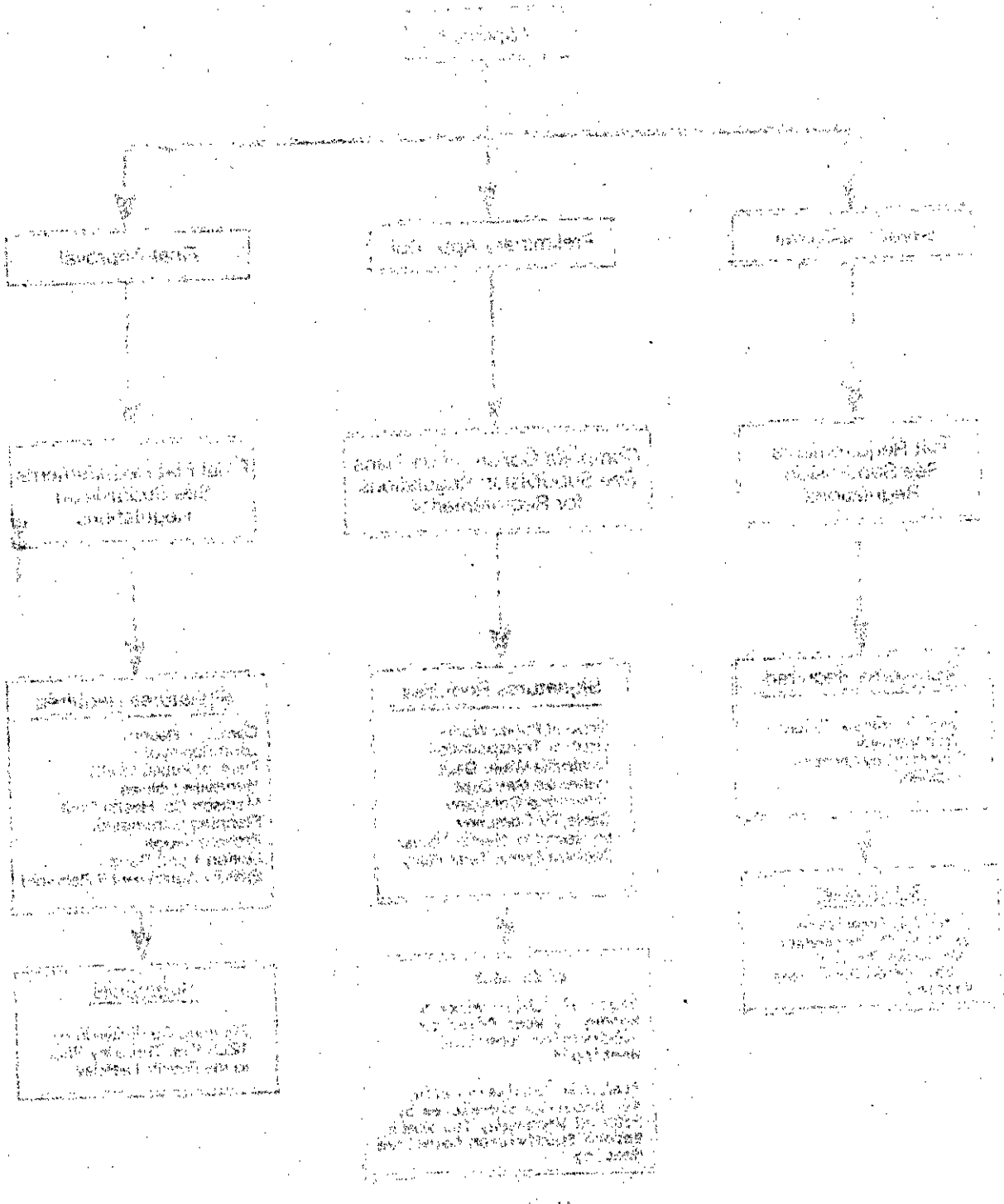


FIGURE 2-2
Grading Permit Application Procedure



**Signature of Other Authorities Not Required Before Submittal to the Dept. of Public Works

FIGURE 2-3
Subdivision Approval



CHAPTER 3
Regulations for Facility Design and Flood Plains

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Chapter 3
REGULATIONS FOR FACILITY DESIGN AND FLOOD PLAINS

3.1 DESIGN CRITERIA SUMMARY

Design criteria presented throughout the manual are summarized below. Additional details on these criteria can be found in the appropriate chapter.

3.1.1 RETURN PERIODS

Design storm return periods for components of stormwater management facilities shall be as follows:

Facility Component	Return Period	Section
Open Channels	25-year	5.2.1

Check channels, using the 100-year return period, to avoid flooding of existing buildings or exceeding the required minimum floor elevations.

Culverts and Bridges for Roads, Streets and Alleys		
1. Minor and Major Arterials	25-year	6.2.1
2. Major Collectors	25-year	6.2.1
3. Minor Collectors	10-year	6.2.1
4. Residential and Local	10-year	6.2.1
5. Roadside Ditches	10-year	6.2.1

Culvert and bridge capacities should be checked, using the 100-year return period, to ensure that overtopping flood conditions do not exceed 1 foot above the top of curb.

Pavement Drainage for Roads, Streets and Alleys		
1. Minor and Major Arterials	25-year	7.2.1
2. Major Collectors	25-year	7.2.1
3. Minor Collectors	10-year	7.2.1
4. Residential and Local	10-year	7.2.1

Pavement flooding spread limitations are summarized in Section 7.1.2. In addition, the pavement surface shall be placed above the predicted surface elevation for the 25-year return period flood of nearby streams or channel systems, and rivers. Future flood elevations shall be used where future flood information is available.

Storm Sewers for Roads, Streets and Alleys		
1. Minor and Major Arterials	25-year	8.1.1
2. Major Collectors	25-year	8.1.1
3. Minor Collectors	10-year	8.1.1
4. Residential and Local	10-year	8.1.1

The impact of a 100-year return period flood shall be analyzed considering adjacent property and buildings, emergency access, and public safety.

3.1.2 MANNING'S n VALUES

Manning's n values for components of stormwater management facilities shall be as follows:

Facility Component	Value	Section
Open Channels		
1. Vegetative Class C	See Equa. 5-1	5.2.3
2. Vegetative Class D	See Equa. 5-2	5.2.3
3. Vegetative Class E	See Equa. 5-3	5.2.3
4. Natural Channels	See Tab. 5-5	5.2.3
5. Rigid, Bare Soil, Rock Cut, Temporary, and Riprap	See Tab. 5-3	5.2.3
Culverts		
1. Circular Concrete	0.012	6.2.4
2. Circular Corrugated Metal Pipe (CMP)	0.024	6.2.4
3. Structural Plate CMP	0.0328-0.0302	6.2.4
4. Box Concrete	0.012	6.2.4
5. Oval Concrete	0.012	6.2.4
6. Arch CMP	0.024	6.2.4
7. Structural Plate Arch CMP	0.0327-0.0306	6.2.4
8. Arch Concrete	0.012	6.2.4
Pavement Drainage		
1. Curb and Gutter	0.016	7.1.3
Storm Sewers		
1. Circular Concrete	0.012	8.1.2
2. Box Concrete	0.012	8.1.2
3. Arch Concrete	0.012	8.1.2
4. Oval Concrete	0.012	8.1.2

5.	Circular CMP	0.024	8.1.2
6.	Arch CMP	0.024	8.1.2

3.2 SUBMITTAL REQUIREMENTS

All data, drainage computations, and plans shall be submitted to the Engineer of Public Works and shall be signed and sealed by a registered professional engineer. If deficiencies are identified by the Engineer of Public Works after reviewing data, computations, and plans, additional information shall be submitted, as requested.

3.2.1 HYDROLOGIC PROCEDURES

Hydrologic data and computations used to determine peak discharges and/or flood hydrographs for the design of drainage facilities shall be consistent with the information presented in Chapter 4. The guidelines summarized in Table 4-1 shall be used for selecting appropriate hydrologic procedures. Hydrologic calculations shall be based on existing conditions and development under current zoning classifications within the watershed. Design discharge values available to the Engineer of Public Works from other projects or studies may be used in the design and analysis of selected stormwater management facilities, as appropriate.

The following minimum information shall be submitted to the Engineer of Public Works:

1. Drainage area maps on a scale of 1 inch = 200 feet or less.

2. Soil maps

3. Run-off coefficients, rainfall intensities, curve numbers, design storms, rainfall excess, overland flow calculations, time of concentration, and other pertinent data in determining peak run-off rates.

4. Peak design discharges, included on plans to allow easy determination of the design discharges for each component of the drainage system or stream; the return period shall be indicated by subscript on each peak design discharge.

5. Calculations, along with hard copies of computer input and output files, as applicable.

3.2.2 OPEN CHANNELS

This section addresses the regulations for proposed and existing open channels and watercourses within and downstream of the proposed development. The required section and lining material for open channels shall be consistent with the criteria and procedures presented in Chapter 5.

Existing Facilities

Improvements may not be required for existing City maintained open drainage systems unless it is determined by hydraulic calculations that the proposed development increases flooding and erosion potential. The Engineer of Public Works may require the submittal of data to support any consideration for not improving existing facilities. Improvements may include, but are not limited to, widening, deepening, paving, grassing, and piping the existing watercourse.

Existing open drainage courses shall not be altered, except when the following conditions requiring improvements occur:

1. The existing drainage course has insufficient capacity to carry the design flow (see Section 3.1.1 and Chapter 5).

2. The existing drainage course requires rerouting because of the development design and layout.

3. The existing drainage course is erratic in alignment, which may create drainage and maintenance problems.

4. At street crossings, the existing drainage course must be improved as required to tie into the proposed drainage structures.

Public utility and drainage easements shall be dedicated along existing drainage courses within the proposed development. Offsite public utility and drainage easements shall be required along existing drainage courses. More detailed information is contained in the section on easements found later in this section.

Existing drainage courses that collect and convey storm water run-off from the proposed development shall be analyzed downstream to ensure that the proposed development does not increase the potential for flooding, erosion, and sedimentation along the drainage course (see Section 3.5).

Any drainage construction offsite from the proposed development shall be accomplished within dedicated public utility and drainage easements.

Alterations to Existing Facilities.

If the drainage course lies within the Floodway District or Floodway Fringe District as shown on the maps identified by the title Flood Boundary and Floodway Maps, City of Huntsville, Alabama, Madison County, any alterations to the drainage course shall comply with all the requirements of the Federal Emergency Management Agency (FEMA) and the City of Huntsville Ordinance No. 87-269, as amended (see Appendix A).

No alteration will be permitted that increases the Regulatory Profile Elevations along the existing drainage course.

Hydraulic Engineering Requirements.

The water surface elevation and floodway widths, as determined for a 100-year return period, shall be established along drainage courses to determine minimum floor elevations and floodway boundaries.

Proposed Facilities

The regulations below pertain to proposed open drainage channels and ditches required for transport of stormwater run-off. Existing watercourses and drainage channels that require improvements will also be covered by the following regulations:

1. The proposed open channels shall be sized to carry the design flow (see Section 3.1.1 and Chapter 5).

2. The water surface elevation and floodway widths, as determined for a 100-year return period, shall be established along drainage courses to determine

minimum floor elevations and floodway boundaries. See Section 3.4 for additional standards and requirements on drainage courses.

3. Proposed open channels shall be protected from erosion by lining the side slopes and bottom if the velocities expected in the channel will create an erosion problem. Acceptable permanent channel lining materials are concrete, riprap, and vegetation (see Chapter 5). Until permanent stabilization is established, a temporary lining of synthetic (e.g., geotextile) or natural (e.g., straw mulch) material shall be provided. Maximum side slopes are as follows:

Lining Material	Maximum Side
Concrete, Grouted Riprap	1:1
Ungrouted Riprap, Rock Lined, Solid Sod in Non-Residential Subdivision	2:1
Vegetative in Non-Residential Subdivision	3:1
Vegetative in Residential Subdivision, Includes Solid Sod	4:1

The lining shall extend upslope a sufficient distance to contain the design storm flow with appropriate freeboard (generally 20 percent of design storm flow depth).

4. Open channel drainage will not be permitted along side lot lines. Underground pipes and facilities shall be provided along the side lot lines to the back easement line. The Engineer of Public Works will consider exceptions to this requirement for the following reasons:

a. The existing or proposed easements will provide the City adequate access to the open channel.

b. The existing or proposed channel requires a 42-inch concrete pipe (or equivalent) or greater to carry the design flow and the channel is designed to prevent erosion and sedimentation problems.

c. The open channel is an existing channel or drainage course located in steep terrain and consisting primarily of natural rock.

5. In addition to these requirements, open channel drainage will not be allowed when the channel meets the following conditions:

a. Channel depth exceeds 12 feet in depth along side or back lot lines in a subdivision development.

b. Channel creates a dangerous situation for the public due to depth and/or location.

c. Channel creates access problems for City maintenance of public utilities.

d. Channel lies in close proximity to existing building structures.

e. Channel is is less than 200 feet in length and can be replaced using underground pipe.

f. Channel lies along streets with curb and gutter.

Easements

A utility and drainage easement shall be required along all drainage courses located within a development or subdivision. The easement shall be of sufficient width to contain the entire drainage course as well as providing adequate space for access for maintenance purposes. The easement shall also provide ample width to preserve the natural integrity of the drainage course and to prevent building of structures within areas reserved for the flow of stormwater run-off. Minimum widths shall be as specified below:

1. Vegetative or flexible linings: 10 feet from top of bank on each side.

2. Rigid linings: 5 feet from top of bank on each

side.

Easements shall be provided of sufficient width to allow for future widening and improvements to any drainage course when required by the Engineer of Public Works.

Plan Requirements

1. Design discharges along the channel at most upstream points in the subdivision or development and at points when additional pipes or ditches are flowing into the channel

2. Channel alignment

3. Channel dimensions and side slopes, and lining material

4. Profile showing flow line, slope, and natural and finished ground elevations at channel centerline

5. Cross section every 50 feet along centerline

6. Structural details

7. Water surface elevations

3.2.3 CULVERTS AND STORM SEWERS

Hydrologic data and computations shall conform to the requirements of Chapters 6 and 8 as appropriate. Hydraulic calculations shall establish capacities for storm sewer and culvert facilities.

Plans submitted to the Engineer of Public Works that show proposed storm sewer and culvert facilities shall include the following information:

1. Pipe or culvert size

2. Class of pipe and material

3. Pipe length and slope

4. Invert flow line elevations

5. Inlet locations

6. Design discharge

7. Existing drainage facilities

8. Profile drawings indicating finished ground elevations

9. Structural details

All drainage culverts and storm sewers shall be reinforced concrete. Class II reinforced concrete pipe may be used in non-traffic areas. Class III reinforced concrete pipe shall be used in traffic areas with appropriate bedding requirements, unless the trafficloads dictate use of higher strength pipe.

Hydraulic grade line data shall be provided. Storm sewer pipe under streets shall extend at least to the back lot line easement unless otherwise approved by the Engineer of Public Works, as allowed in Section 3.2.2.

3.2.4 GUTTERS AND INLETS

Gutter and inlet capacities shall be determined when new streets are designed. The gutter and inlet hydraulics of streets and roadways shall be consistent with the criteria and procedures presented in Chapter 7.

Inlets shall be placed so that at least 80 percent of the gutter flow is intercepted (i.e., maximum bypass is 20 percent). The maximum length of longitudinal curb and gutter section without a "Type S" inlet shall be 300 feet. Calculations, as detailed in Chapter 7, shall be made to ensure that new development and construction that increases run-off to existing streets does not violate maximum spread limitations, as contained in Section 7.1.2. If the existing spread on the street equals or exceeds the maximum allowed before considering the proposed development, the run-off from the new development shall be transported by pipe to an existing storm sewer system.

The following information shall be provided in the plans:

1. Design flow along the curb and gutter

2. Inlet locations, intercepted flow, and bypass flow

3. Street profiles and typical sections.

3.2.5 STORAGE SYSTEMS

The standard for determining the requirement for storage systems shall be that the calculated peak rate of stormwater run-off shall be no greater after development of a site than before development of a site. The calculations shall be made considering a ten year return period, twenty-four hour duration, type II storm distribution as defined by the Soil Conservation Service (SCS), U.S. Department of Agriculture (USDA), for both before and after development site conditions. The hydraulics and design of stormwater storage systems shall be consistent with the criteria and procedures presented in Chapter 9. Storage systems will be required to limit the adverse impacts of increased run-off. Retention and/or detention facilities shall be provided in the following specific cases:

1. Undersized facilities, channels, and ditches are located downstream of the proposed development.

2. The implementation of detention and/or retention facilities has been determined to effectively reduce the chance of downstream damages caused by increased run-off.

Exemptions from the requirement for retention and/or detention facilities shall be given in the following specific cases:

1. Comercial or Industrial development which adds less than 10,000 square feet of impervious surface, provided all of the criteria presented in Section 2.3.1 are satisfied.

2. Any development where, based upon a competent engineering study, use of such a facility will result in a greater potential for flooding, provided that existing facilities, channels, and ditches will not exceed capacity. The study shall be prepared by a registered professional engineer and

shall include hydrologic and hydraulic calculations in support thereof.

3. Construction or repair of, or addition to any residential dwelling located on single lots within an approved subdivision.

4. Construction or repair of, or addition to any commercial or industrial buildings or facilities including parking areas located within an approved subdivision where an existing stormwater detention or retention facility has been designed and constructed to meet the requirements of this manual.

Detention and/or retention facilities that are not required may be allowed, provided such facilities have no detrimental effect on upstream, downstream, and adjacent drainage facilities and further provided that the calculated peak rate of stormwater run-off shall be no greater after development than before development. The hydraulics and design of stormwater storage systems shall be consistent with the criteria and procedures presented in Chapter 9.

A maintenance plan approved by the Engineer of Public Works will be required for any detention and/or retention facility. Public utility and drainage easements of adequate width shall be required to provide access to the facility for maintenance activities.

The following information shall be provided in the plans:

1. Pre-development and post-development design flood hydrographs for the 100-year and 10-year storm return periods.

2. Stage-storage data

3. Stage-discharge data

4. Construction details

5. Design criteria

6. Structural details for spillway and outlet facilities.

3.2.6 EROSION AND SEDIMENT CONTROL

An erosion and sediment control plan must be approved by the Engineer of Public Works for all land alteration activities unless exempted (see Chapter 10). Any activity requiring a grading permit shall include an erosion and sediment control plan. Development of an erosion and sediment control plan shall be consistent with criteria and procedures presented in Chapter 10.

Plans submitted to the Engineer of Public Works for approval in the building permit process shall include an erosion and sediment control plan, unless no land disturbance activity is associated with the building.

All subdivision plans submitted to the Engineer of Public Works for approval shall include an erosion and sediment control plan.

Erosion and sediment control plans are not required for single residential or commercial lots within subdivision developments. This does not relieve the builder from controlling the erosion of dirt, rock, debris, and building materials onto public or private property and into drainage systems and streets. The builder or contractor is also not relieved from any fines or penalties that may be issued against him for violation of City ordinances restricting deposits of debris on any street, alley, or public grounds within the City (Section 20, City of Huntsville Code of Ordinances).

If the Engineer of Public Works determines that the type or extent of the work being performed or planned will not cause adverse impacts to the surrounding property, the requirement for an erosion and sediment control plan may be waived.

The erosion and sediment control plan shall include a time schedule detailing the sequence of erosion and sediment control measures along with the sequence of construction. The components of an approved erosion and sediment control plan shall be implemented and maintained to ensure that the control measures are functioning as intended.

The Engineer of Public Works shall require the following information on the erosion and sediment control plans:

1. A complete plan of the proposed development at a

scale no less than 1 inch = 100 feet. This plan is to include existing and proposed contours at intervals no greater than 2 feet, with at least one benchmark required.

2. Existing and proposed buildings, impervious surfaces, and drainage structures, including inlets, catch basins, manholes, junction boxes, driveway pipes, culverts, cross drains, headwalls, outlet facilities, and sanitary sewers.

3. Any proposed drainage improvements such as channels, ditches, drainage pipes, and structures.

4. All fill areas indicated as such, with the limits and elevations indicated.

5. Drainage arrows indicating the existing and proposed direction of run-off throughout the plan.

6. Temporary erosion and sediment control measures to be implemented during construction, including a time sequence schedule for the construction and erosion control plan.

7. Measures the builder or contractor will use to keep construction materials, including dirt, mud, and debris, off the streets.

3.3 FLOOD ELEVATION AND FLOODWAY DATA

3.3.1 FEMA FLOODWAY MAPS

The FEMA Floodway and Floodway Fringe District boundaries are established on the maps entitled Flood Boundary and Floodway Maps, City of Huntsville, Alabama, Madison County (Community Panels 010153 0001-0050), composed of the following panels issued by the U.S. Department of Housing and Urban Development, Federal Insurance Administration:

Panel Number	Effective Date
10, 15, 20 and 30	Nov. 1, 1979
25, 35, 40 and 50	Sept. 27, 1985

Information pertaining to the development of these maps is presented in the Flood Insurance Study (FEMA, 1985), which covers the incorporated area of the City of Huntsville, Madison County, Alabama, excluding the Redstone Arsenal.

Flood elevations and floodway limits were developed for the following streams using detailed methods:

Stream	Miles
Tennessee River	332.00 - 333.47
Unnamed Tributary to Tennessee River	0.63 - 3.20
Aldridge Creek	2.10 - 10.04
Huntsville Spring Branch	9.75 - 14.64
Pinhook Creek	14.64 - 19.07
East Fork Pinhook Creek	0.00 - 0.99
Blue Spring Creek	0.00 - 0.98
Dallas Branch	0.00 - 1.71
Unnamed Tributary to Pinhook Creek (Dallas Branch Bypass)	0.00 - 1.37
Fagan Creek	0.00 - 1.37
Broglan Branch	0.00 - 3.52
Dry Creek	0.00 - 0.86
McDonald Creek	3.67 - 8.33
Unnamed Tributary to McDonald Creek	0.01 - 1.00
Sherwood Branch	0.00 - 3.52
Indian Creek	13.08 - 16.69
Betts Spring Branch	2.26 - 3.50
Bradford Creek	1.82 - 2.96

Approximate methods were used on four ponding areas in the west-central portion of the city, on McDonald Creek upstream of mile 8.33, in the ponding areas west of Ardmore Highway at Brandontown, and at the point where Triana Boulevard nears Southern Railway. These areas were studied by approximate methods because of the lack of existing or expected development along their flood plains.

A summary of the peak discharge data used to develop the Flood Boundary and Floodway Map data is presented in Table 3-1.

3.3.2 UNPUBLISHED DATA

If a project is located in an unnumbered A zone on the FEMA maps or areas not covered by the FEMA maps, the applicant shall provide base flood elevation and floodway data as documented in a Flood Plain Report when the project is greater than 5 acres.

In addition, a Flood Plain Report shall be required for areas outside unnumbered A zones or areas not covered by the FEMA maps when the stream has a drainage area of 1 square mile or greater. Approximate methods for flood level determination may be used if approval is granted by the Engineer of Public Works.

The Flood Plain Report shall document procedures and input data and provide plan and profile maps with flood elevations and floodway limits. Regulatory flood elevations and floodway limits shall be established using peak discharge rates based on existing conditions under current zoning classifications. The plan view shall show the flooding limits for 100- and 500-year return periods, floodway base line, cross section stations, and adjacent boundaries. The profile should show stream invert, cross section stations, and computed water surface elevations. The report should also show the drainage divides on the plan and the ultimate zoning categories used.

Base flood elevation and floodway data submitted by the applicant for areas previously without such data, or where channel improvements and/or mitigation measures result in changes to reported information, shall be reviewed by the Engineer of Public Works and, if acceptable, shall be processed for adoption as regulatory flood plain management

data. When the base flood elevation and floodway data submitted by the applicant result in a deviation from the data presented on the FEMA maps, such deviations may become official following review and approval by both the Engineer of Public Works and FEMA, if required (i.e., original information applies until revisions are approved).

3.4 FLOOD PLAIN REQUIREMENTS

3.4.1 ZONING ORDINANCE

Uses permitted within the flood plain shall be in accordance with Ordinance No. 87-269, as amended, Article 62 of the Zoning Ordinance of the City of Huntsville, Alabama, as presented in Appendix A. The regulations and controls set forth shall be applied within the areas designated by the Zoning Ordinance. However, nothing shall prohibit the application of Ordinance No. 87-269, as amended, to lands that can be demonstrated by generally accepted engineering methods to lie within any flood plain.

Under the Zoning Ordinance, it shall be unlawful for any person, firm, or corporation to carry out any mining, filling, grading, paving, excavation, or drilling operations in an area designated as a Floodway District or Floodway Fringe District (see Section 3.3.1) without having first obtained a permit for such activity from the Engineer of Public Works.

No building permit will be issued where any part of the property lies within any flood hazard area until plans for such improvements have been reviewed and approved by the Engineer of Public Works.

3.4.2 UNSTUDIED AREAS

Development along streams not included in the FEMA Floodway and Floodway Fringe District study area shall also conform to the requirements of the City of Huntsville Zoning Ordinance, Article 62, Floodway-Floodway Fringe District Regulations (see Appendix A), latest revision. Additional requirements for an unstudied area or areas where regulatory elevations and floodway boundaries have not been determined are also given in Article 62 and Section 3.3.3.

3.4.3 ACTIVITIES WITHIN FLOOD PLAINS

All land development, construction, and building activities are prohibited within Floodways, except as allowed by Zoning Ordinance Article 62 (see Appendix A).

All construction or substantial improvements of any structure shall have the lowest floor, including basement, elevated to at least 1 foot above the FEMA base flood elevation (see Section 3.3.1).

3.4.4 SUBDIVISIONS

Land to be subdivided shall be developed in compliance with the latest version of the Huntsville Subdivision Regulations except that land located within FEMA Floodway Boundaries shall not be subdivided or developed.

3.5 INADEQUATE FACILITIES

If it is determined that any proposed development has inadequate storm sewer facilities downstream of the proposed development and outside its property lines, the Engineer of Public Works shall not approve the development until the problem is resolved. "Inadequate storm sewer facilities" shall mean those insufficient to carry the design flow and meet the design criteria in this manual and those not located within public utility and drainage easements. Storm sewer facilities shall include pipes, culverts, ditches, channels, watercourses, and retention/detention ponds.

3.6 TRAFFIC-SAFE DRAINAGE FACILITIES

The City of Huntsville recognizes that design standards change constantly and become more efficient and safer with time. The standards contained herein have been adopted to ensure that construction of new roadways, reconstruction of existing roadways, and construction of driveway turnouts to existing roadways is accomplished in a manner that maximizes motorists' safety at minimum additional cost.

Clear zone distance data from the American Association of State Highway Transportation Officials (AASHTO) (1988) are presented in Table 3-2.

Parallel and transverse pipes and culverts and drop inlets are common drainage system elements that shall be designed, constructed, and maintained with both hydraulic efficiency and roadside safety in mind.

In general, the following options, listed in order of preference, are applicable to all drainage features:

1. Eliminate non-essential drainage structures.

2. Design or modify drainage structures so they are transversable or present a minimal hazard to an errant vehicle.

3. If a major drainage feature cannot be redesigned or relocated, shield it by a suitable traffic barrier if it is in a vulnerable location.

The rest of this section identifies the safety problems associated with pipes, culverts, and drop inlets and offers recommendations concerning the location and design of these features to improve their safety characteristics without adversely affecting their hydraulic capabilities. The information presented applies to all roadway types and projects. Typical construction details for providing traffic-safe drainage facilities are contained in the standard drawings (Appendix 3).

3.6.1. CROSS-DRAINAGE STRUCTURES

Cross-drainage structures are designed to carry water underneath the roadway embankment and vary in size from 15-inch concrete or corrugated metal pipes to multi-barreled concrete box culverts or structural plate pipes with clear spans of 10 feet or more. The potential hazard is either a fixed object protruding above an otherwise transversable embankment or an opening into which a vehicle can drop. The design engineer shall minimize roadside hazards at cross drains with one of the following methods:

1. Using a transversable design

2. Extending the structure so it is less likely to be hit

3. Shielding the structure

Each of these options are discussed below.

Transversable Designs

The design engineer should attempt to provide embankments as smooth or transversable as practical for a given facility. Transversable, non-recoverable slopes should be rounded at top and bottom and may provide a relatively flat runout area at the bottom. If a slope is generally transversable, the preferred treatment for any cross-drainage structure is to extend (or shorten) it to intercept the roadway embankment and to match the inlet or outlet slope to the embankment slope. For small culverts, no other treatment is required.

Single structures and end treatments wider than 36 inches can be made transversable for passenger size vehicles by using bar grates or pipes to reduce the clear opening width. Figure 3-1 shows recommended sizes to support a full size automobile and is based on a 30-inch bar spacing. It is important to note that the toe of the embankment slope and the ditch or streambed area immediately adjacent to the culvert must be fairly transversable if the use of a grate is to have any significant safety benefits. Normally, grading within the right-of-way limits can produce a satisfactory runout path.

For median drainage where flood debris is not a concern and mowing operations are frequently required, much smaller openings between bars can be tolerated. Figure 3-2 shows a median drainage inlet design that can be used where cover is insufficient to construct a drop inlet directly across the cross-drainage pipe.

Extension of Structures

For intermediate sized pipes and culverts for which inlets and outlets cannot readily be made transversable, the structure can be extended at or just beyond the appropriate clear zone. However, the extended culvert headwall may become the only significant man-made fixed object hazard immediately at the edge of the clear zone along the section of roadway under design. In that case and if the roadside is generally transversable to the right-of-way line elsewhere, extending the culvert to the edge of the clear zone may not be the best alternative. This is particularly

true on freeways and other high speed, controlled access facilities.
Conversely, if the roadway has numerous fixed objects, both natural and man-made, at the edge of the clear zone, extending individual structures to the same minimum distance from traffic may be appropriate. However, the preferred safety treatment is usually to redesign the inlet/outlet so it is no longer a fixed object hazard.

Shielding

Major drainage structures are costly to extend and often have end sections that cannot be made trans-versable. In such cases, shielding with an appropriate traffic barrier is often the most effective safety treatment. The traffic barrier will be longer and closer to the roadway than the structure opening and more likely to be hit than a shielded culvert located further from the traveled way. Nevertheless, a properly designed, installed, and maintained barrier system may provide an increased level of safety for errant motorists.

3.6.2 PARALLEL DRAINAGE FEATURES

Parallel or side drain culverts are typically used under driveways, field entrances, access ramps, intersecting side roads, and median crossovers. As with cross-drainage structures, the designer's primary concern should be to design generally transversable slopes and to match the culvert opening with adjacent slopes. Embankment slopes that can be struck at 90 degrees by vehicles run off the road should be constructed with 6:1 slopes or flatter for locations susceptible to high speed impacts (speed limit areas of 40 miles per hour or greater). On low volume roads with low speed limits, a 3:1 slope or flatter shall be used. With these guidelines, safety treatment options are similar to those for cross-drainage structures. The design engineer involved in the design of parallel drainage features shall minimize the hazard of the drainage structures by using one of the following methods:

1. Eliminate the structure.

2. Use a transversable design.

3. Move the structure laterally to a less vulnerable

location.

4. Shield the structure.

Eliminate the Structure

Unlike cross-drainage pipes and culverts, which are essential for proper drainage and operation of a road or street, parallel pipes can sometimes be eliminated by constructing an overflow section on the field entrance, driveway, intersecting side road, or median crossover. This treatment will be appropriate only at low volume locations where it does not decrease the sight distance available to drivers entering the main road.

Transversable Designs

Embankments shall be designed with consideration given to their effect on the roadside environment. Parallel drainage structures shall match the selected side slope and shall be made transversable when they are located in a vulner-able position relative to main road traffic. The addition of pipes and bars parallel to traffic can prevent a wheel from catching in the culvert opening. The bottom bar or pipe should be set at a maximum of 4 inches above the culvert invert. A possible transversable design for a parallel culvert is shown in Figure 3-3. When ditch grades permit, the inlet end may use a drop inlet type design to reduce the length of grate required.

In locations where headwater depth is limited, a larger pipe should be used or the parallel drainage structure may be positioned outside the clear zone.

Relocate the Structure

Some parallel drainage structures can be moved laterally further from the traveled way. This approach often allows the transverse embankment slope within the selected clear zone to be flattened. If the embankment at the new culvert locations is transversable and likely to be encroached upon by either main road or side road traffic, safety treatment will be applied. The inlet or outlet shall match the embankment slope.

Shielding

In cases where the embankment cannot be made transversable or the structure is too large to be effectively safety treated and it cannot be relocated, it may be necessary to shield the hazard with a traffic barrier. Specific information is found in AASHTO (1988).

3.6.3 DROP INLETS

Drop inlets can be classified as on-roadway or off-roadway structures. On-roadway inlets are usually located on or alongside the shoulder of a street or highway and include curb-opening inlets, grated inlets, slotted drain inlets, or combinations. Since on-roadway inlets are installed flush with the pavement surface, they do not constitute a hazard to motorists. In addition, they must be capable of supporting vehicle wheel loads and present no hazard to pedestrians or bicyclists.

Off-roadway drop inlets are used in medians of divided roadways and sometimes in roadside ditches. They shall be designed and located to present a minimal obstacle to errant motorists. This can be accomplished by building them flush with the ditch bottom or slope on which they are located. No portion of the design shall project high enough to cause vehicular snagging, obstruction, or instability.

The opening shall be treated to prevent a vehicle wheel from dropping into it but unless pedestrians are a consideration, grates with openings as small as those used for pavement drainage are not necessary. Neither is it necessary to design for a smooth ride over the inlet. It is sufficient to prevent wheel snagging and the resulting sudden deceleration or loss of control.

Table 3-1
SUMMARY OF PEAK DISCHARGES FROM THE FEMA FLOOD INSURANCE STUDY

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Year	50-Year	100-Year	500-Year
Tennessee River					
River Mile 332.53	25,610	286,000	325,000	346,000	400,000
Unnamed Tributary to Tennessee River					
River Mile 0.63	6.78	1,890	3,090	3,690	4,970
River Mile 2.35	3.91	1,350	2,080	2,400	3,240
River Mile 3.68	1.67	700	1,100	1,300	1,700
Aldridge Creek					
River Mile 0.00	21.7	4,600	7,500	8,400	12,000
River Mile 6.62	11.4	3,000	4,800	5,500	7,800
River Mile 10.04	3.46	1,300	2,100	2,400	3,300
Huntsville Spring Branch					
River Mile 9.75	46.9	12,000	18,000	20,500	28,000
River Mile 12.25	41.8	11,000	17,000	19,000	26,000
River Mile 13.93	36.8	10,000	15,500	17,500	28,500
River Mile 14.54	27.0	8,000	12,500	14,500	19,000
Pinhook Creek					
River Mile 14.68	22.5	6,972	10,957	12,947	16,929
River Mile 16.33	15.9	5,080	7,905	9,249	12,117
River Mile 16.98	9.10	3,249	5,316	6,299	8,265
River Mile 19.07	5.63	2,200	3,500	4,200	5,500
Blue Spring Creek					
River Mile 0.00	2.53	1,000	1,500	1,800	2,400
River Mile 0.98	1.83	750	1,150	1,420	1,820
Dallas Branch					
River Mile 0.00	3.73	2,600	4,100	4,700	6,800
River Mile 1.71	2.40	1,300	1,990	2,320	3,350

Table 3-1
(Continued)

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Year	50-Year	100-Year	500-Year
Unnamed Tributary to Pinhook Creek (Dallas Branch Bypass)					
River Mile 0.00	0.82	590	1,550	2,080	3,790
River Mile 0.84	0.62	460	1,400	1,910	3,590
Fagan Creek					
River Mile 0.00	4.36	2,600	4,200	5,000	7,400
River Mile 1.37	3.25	1,530	2,390	2,850	4,110
Broglan Branch					
River Mile 0.15	9.70	3,949	6,827	8,505	13,854
River Mile 2.70	6.48	2,800	5,000	6,100	10,000
River Mile 3.52	0.80	870	1,190	1,290	1,740
Dry Creek					
River Mile 0.00	4.86	2,000	3,700	4,700	7,700
River Mile 1.02	4.03	1,650	2,980	3,990	6,500
McDonald Creek					
River Mile 3.11	11.3	3,200	4,900	5,700	7,700
River Mile 5.16	8.12	2,400	3,700	4,400	5,900
River Mile 6.56	5.31	1,600	2,500	3,000	4,100
River Mile 8.26	1.38	530	860	1,050	1,400
Unnamed Tributary to McDonald Creek					
River Mile 0.02	1.55	860	1,200	1,300	1,700
River Mile 0.81	0.51	310	460	530	670
Sherwood Branch					
River Mile 0.00	3.16	1,100	1,700	2,100	2,700
River Mile 2.28	1.65	600	950	1,160	1,630
River Mile 3.52	1.19	480	750	930	1,200

Table 3-1
(Continued)

Flooding Source and Location	Drainage Area (square miles)	Peak Discharges (cfs) 10-Year	50-Year	100-Year	500-Year
Indian Creek					
River Mile 12.80	49.0	6,800	11,000	13,000	18,000
River Mile 17.52	39.6	5,731	9,344	11,052	15,181
Betts Spring Branch					
River Mile 1.22	8.29	2,100	3,400	4,100	5,700
River Mile 2.90	2.76	900	1,400	1,790	2,300
Bradford Creek					
River Mile 0.85	15.5	3,320	5,520	6,740	9,350
River Mile 3.22	13.3	3,000	4,900	6,000	8,200

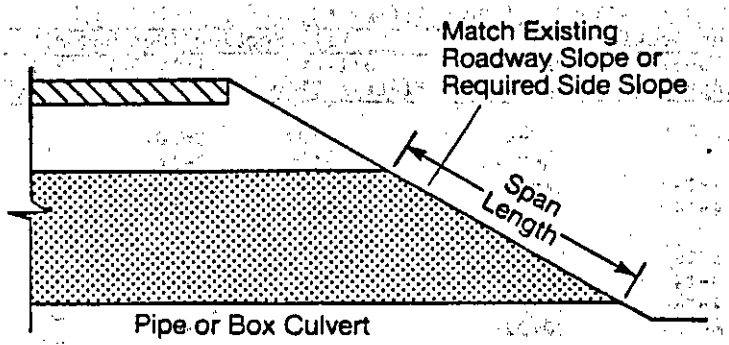
Table 3-2
CLEAR ZONE DISTANCE DATA

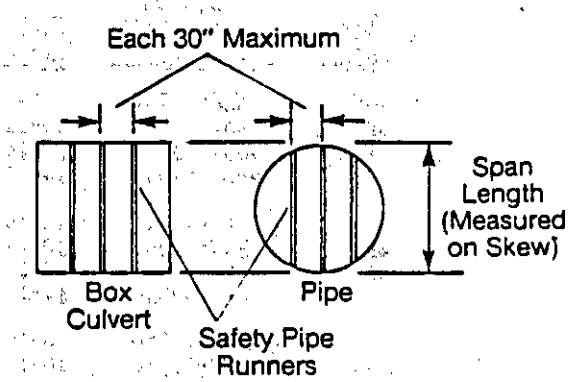
Speed (mph)	Design Average Daily Traffic	Clear Zone Distances (in feet from edge of traveled way) Cut Section 3:1 Slope	Cut Section 4:1 to 5:1 Slope	Cut Section 6:1 or Flatter Slope	Fill Section (Unrounded) 6:1 or Flatter Slope	Fill Section (Unrounded) 4:1 to 5:1 Slope	Fill Section (Unrounded) 3:1 Slope[a]
40 or Less	Under 750	7-8	7-8	7-8	7-8	8-10	–
	750-1,000	10-12	10-12	10-12	10-12	12-14	–
	1,500-6,000	12-14	12-14	12-14	12-14	14-16	–
	Over 6,000	14-16	14-16	14-16	14-16	16-18	–
45-50	Under 750	8-10	8-10	10-12	10-12	12-14	–
	750-1,000	10-12	12-14	14-16	12-14	16-20	–
	1,500-6,000	12-14	14-16	16-18	16-18	20-24	–
	Over 6,000	14-16	18-20	20-22	18-20	24-28	–
55	Under 750	8-10	10-12	10-12	12-14	14-18	–
	750-1,000	10-12	14-16	16-18	16-18	20-24	–
	1,500-6,000	14-16	16-18	20-22	20-22	24-30	–
	Over 6,000	16-18	20-22	22-24	22-26	26-32	–

Reference: American Association of State Highway Transportation Officials (1988).

Note: Clear zone is defined as a roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. The clear zone should not be viewed as a discrete, exact distance, but as the center of a zone that should then be analyzed on a site-specific basis.

[a]Since recovery is less likely on the unshielded, transversable 3:1 slopes, fixed objects should not be present in the vicinity of the toe of those slopes.





Example Safety Pipe Runner Sizes

Span Length (feet)	Safety Pipe Diameter (inches)
up to 12	3.0
12 to 16	3.5
16 to 20	4.0
20 or less with center support	3.0

FIGURE 3-1
Recommended Cross-Drain Safety Pipe Runner Sizes for Various Span Lengths

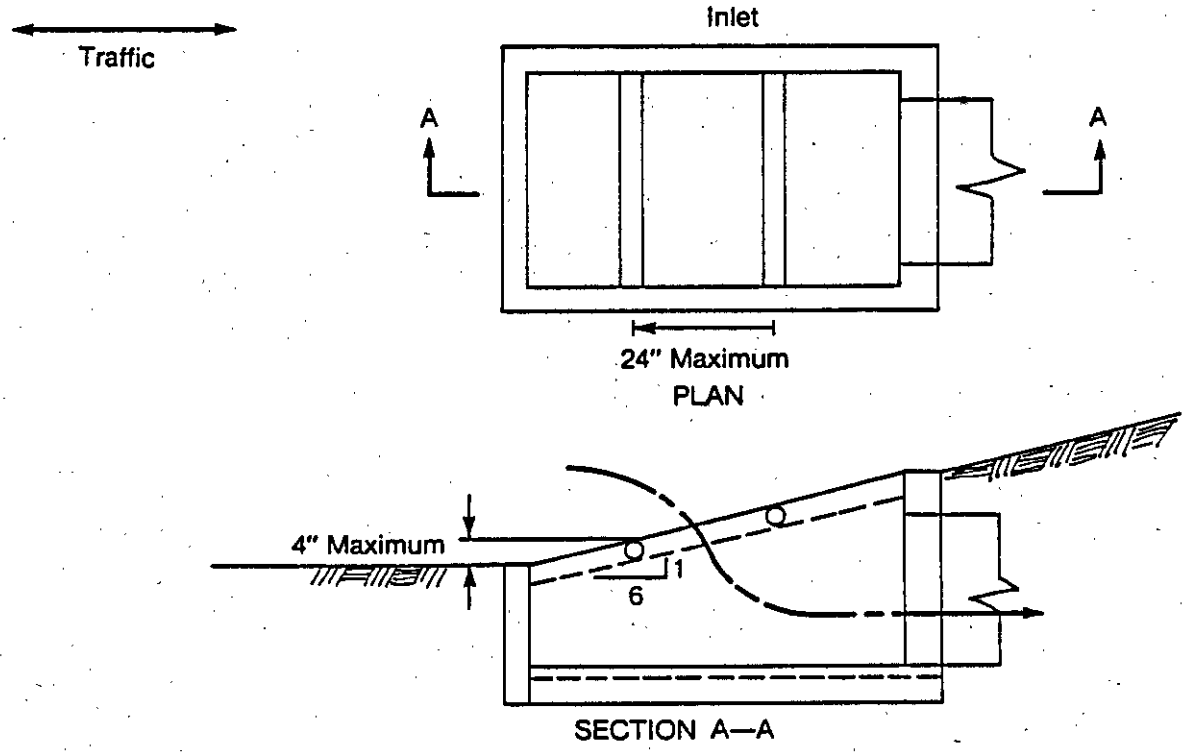
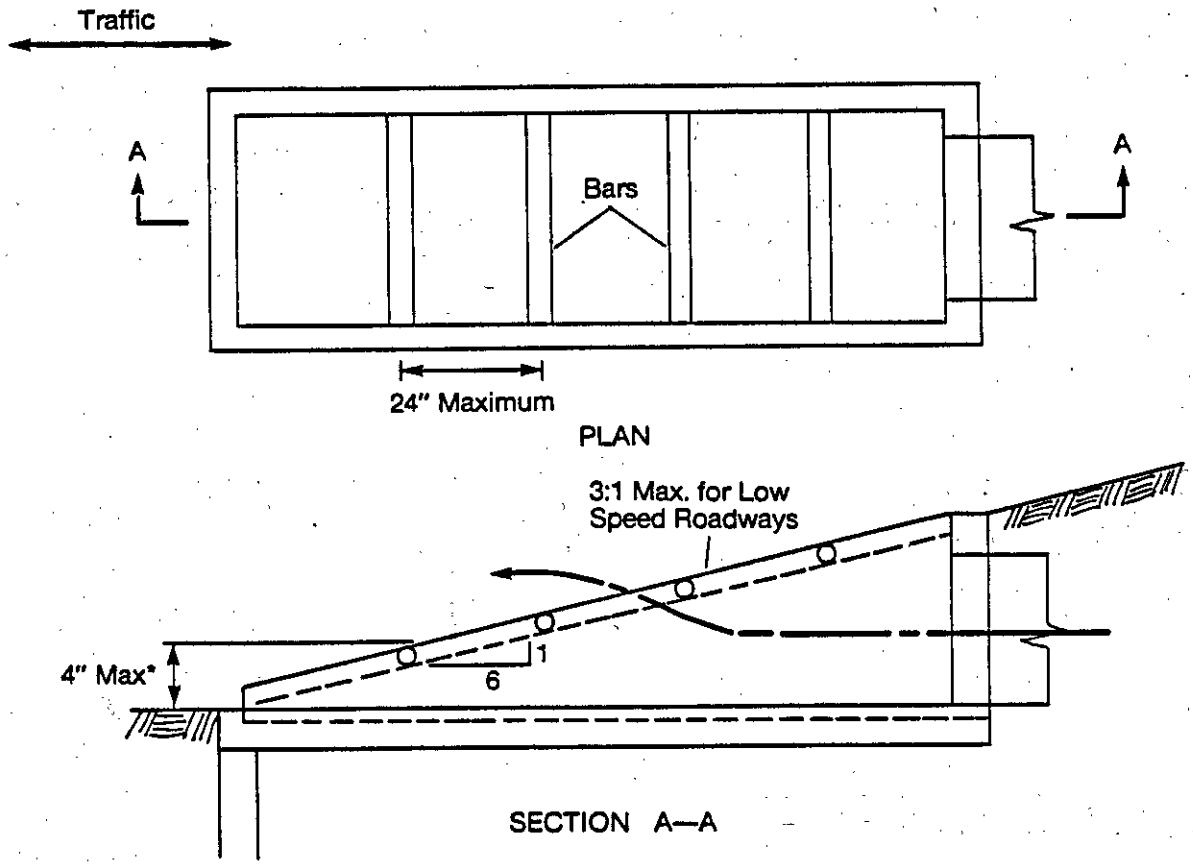
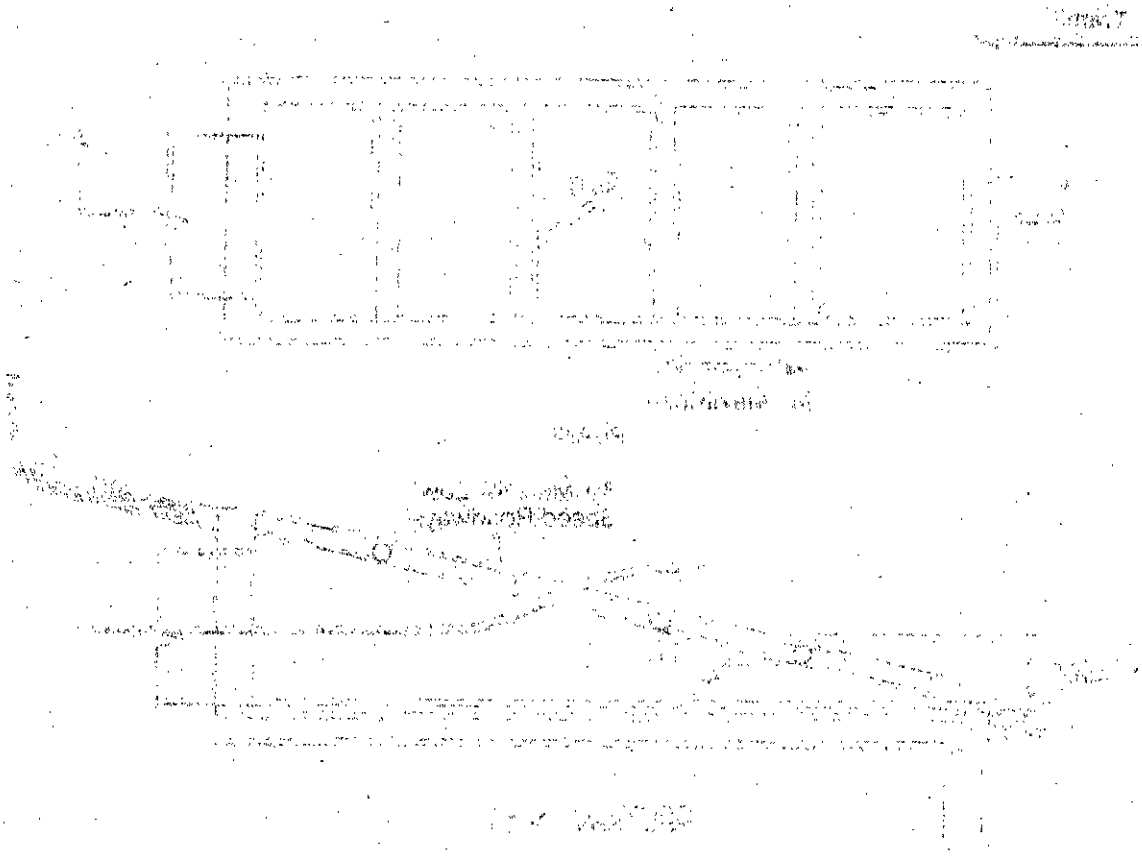


FIGURE 3-2
Transversable Median Cross Drains for Insufficient Cover



*4" max. applies to both inlet and outlet on two way roads and only to the side facing traffic on divided highways.

FIGURE 3-3
Inlet/Outlet Transversable Example for Parallel Drainage



PART 2
Technical Information

PART 2 - TECHNICAL INFORMATION

Part 2 of the Huntsville Stormwater Management Manual is oriented to the technical concerns of designers and builders. The following chapters are included in Part 2:

4. Hydrology
5. Open Channel Hydraulics
6. Culvert Hydraulics
7. Gutter and Inlet Hydraulics
8. Storm Sewer Hydraulics
9. Storage System Hydraulics
10. Erosion and Sediment Control

CHAPTER 4
Hydrology

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Chapter 4
HYDROLOGY

4.1 PROCEDURE SELECTION

The guidelines in Table 4-1 shall be followed 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), a flood hydrograph is usually required.

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

The Rational Method (see Section 4.5.2) is subject to the following limitations:

1. Estimates of peak design flows only.

2. Time of concentration, t_c, of 5 minutes $\leq t_c \leq$ 30 minutes.

3. Drainage area, DA $\leq$ 50 acres.

In addition, results should be compared using other methods, and approval by the Engineer of Public Works is required.

The SCS TR-55 (1986) graphical method (see Section 4.5.5) is subject to the following limitations:

1. Estimates of peak design flows only.

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

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

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

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

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

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

10. No consideration of hydrologic channel routing.

The SCS TR-55 (1986) tabular method (see Section 4.6.5) can be used to estimate flood hydrographs and to approximate the effects of hydrologic channel routing, subject to the following limitations:

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

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

3. DAs of individual subareas 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 = 484.

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

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

TVA regression equations (see Section 4.5.3) have been prepared for small, ungaged, urban watersheds in Huntsville. The TVA regression equations are subject to the following limitations:

1. Estimates of peak design flows only.

2. DA between 60 and 3,200 acres.

3. Imperviousness between 3 to 50 percent.

4. Inapplicable when extensive channelization or regulation is incorporated into the watershed.

TVA graphs (see Section 4.5.4) provide flow frequency estimates for modeled streams that are applicable only to those stream reaches that were included in the modeling.

Unit hydrograph theory (see Sections 4.6.2 and 4.6.3) provides a generally applicable procedure for developing flood hydrographs using a basin-specific unit hydrograph and an appropriate rainfall hyetograph. Many computer models use unit hydrograph theory. With careful development of a basin-specific unit hydrograph, this versatile method can be adapted to a wide range of conditions.

Computer modeling is appropriate when limitations of the simpler methods are exceeded, complex situations are being studied, or more detailed information is required.

4.2 HYETOGRAPHS

The rainfall data in Section 4.2.1 represent average intensity or volume over a particular duration. When a flood hydrograph is to be developed, however, a time variable distribution or hyetograph is required. Two methods are described in Sections 4.2.2 and 4.2.3 for constructing the design storm hyetograph: the SCS Type II storm distribution (24-hour duration) and the balanced storm approach (any duration).

4.2.1 DESIGN STORM DATA

Hydrologic design storm data should be collected by the following procedure:

1. Select an appropriate procedure for hydrologic calculations using information in Section 4.1.

2. Determine the type of precipitation data that are needed. Generally, either intensity-duration-frequency (IDF) curves or hyetographs for historic or design storm conditions are used.

IDF curves and data for Huntsville are given in Figure 4-1 along with depth-duration-frequency data up to a 24-hour duration. These data were obtained from TP-40 (Hershfield, 1961) and HYDRO-35 (Frederick et al., 1977). Additional data for durations of 2, 4, 7, and 10 days can be obtained from Miller (1964).

Huntsville area climatological data are collected by the National Weather Service Office (WSO). The WSO at the Huntsville/Madison County Airport, Carl T. Jones Field, is located 11 miles southwest of the center of Huntsville. All official weather reports and data for Huntsville currently come from Jones Field.

The National Climatic Center evaluates and summarizes collected climatological data on a daily, monthly, and annual basis. Publications summarizing the climatological data collected are available at a small fee to the general public from the U.S. Department of Commerce, National Climatic Center, Federal Building, Asheville, North Carolina 28801, or from the WSO at Jones Field.

Monthly and annual precipitation averages for Huntsville from 1959 to 1986 are shown in Table 4-2. The average annual precipitation depth for this record was 56.07 inches. The maximum and minimum precipitation depths of 72.97 and 41.81 inches occurred in 1975 and 1981. The average monthly precipitation depths vary from a low of 3.41 inches in October to a high of 6.71 inches in March.

4.2.2 SCS TYPE II STORM

The SCS has developed a set of 24-hour storm distributions representing average conditions for different regions of the United States. The Type II storm is appropriate for Huntsville and its cumulative mass curve is presented in

Figure 4-2. The curve is applicable to the 24-hour rainfall depth for any recurrence interval.

The SCS dimensionless hyetograph approach takes the following steps:

1. Select a design storm time step and return period.

2. Obtain the 24-hour precipitation depth, P_{24}, from Figure 4-1 for the return period selected in Step 1.

3. Use the time step selected above and tabulate P_x/P_{24} ratios for each time step of the 24-hour storm, using Figure 4-2.

4. Multiply the P_x/P_{24} ratios from Step 3 by the 24-hour precipitation depth from Step 2. Results of these calculations provide a cumulative rainfall hyetograph. Incremental hyetograph data can be prepared by finding the difference between each cumulative depth interval.

4.2.3 BALANCED STORM

The balanced storm approach provides a precipitation hyetograph that has the same return period for each time interval within the total storm duration. This approach places the largest increment of rainfall for the selected time step at the midpoint of the total storm duration. For example, the midpoint for a 24-hour storm is 12 hours. Smaller depth increments are arranged symmetrically about the largest value, with the second largest value placed before and the third largest value placed after the largest. This "before and after" process is illustrated in Figure 4-3 and continues until the entire hyetograph is developed.

The data from Section 4.2.1 can be used to develop a site-specific balanced storm for Huntsville using the procedure described below. The balanced storm is nearly identical to the 24-hour SCS Type II storm, except that the SCS storm distribution is slightly more intense for durations between 15 minutes and 1 hour of the midpoint.

1. Select the design storm time step and return period. The time step should correspond with the unit hydrograph duration if unit hydrograph procedures will be used (see Section 4.6.3).

2. Use the time step and return period from Step 1 to obtain precipitation depth data. Depth-duration frequency data can be obtained from Figure 4-1 and the increments of precipitation determined from the following steps:

   a. Find the intensity at each multiple of the time step (time intervals) to the duration of the storm.

   b. Determine the rainfall volume for each time interval (volume = intensity x time interval).

   c. Determine the increment of precipitation depth for each time step by subtracting the volume of the previous time interval from the volume of the present time interval.

3. Select the largest increment of precipitation depth from Step 2 and place it at the midpoint of the total storm duration. For a 24-hour storm, the midpoint is 12 hours.

4. Arrange smaller depth increments symmetrically about the largest depth. The second largest value is placed before the largest and the third largest value is placed after the largest.

5. Continue placing sequential depth increments before and after the largest value until the entire storm duration has been considered.

An example of the procedure for developing a balanced storm is presented in Section 4.6.

4.3 RAINFALL EXCESS

Rainfall excess is the depth of precipitation that runs off an area during or immediately following a rainstorm, or the

water depth remaining when the following abstractions are subtracted from the total precipitation:

1. Evaporation
2. Infiltration
3. Transpiration
4. Interception
5. Depression storage

Because the complexity of the actual process precludes a detailed determination of each abstraction, several methods are available to approximate the combined effects based on watershed characteristics.

Either the runoff coefficient or the SCS curve number can be used to estimate rainfall excess. Each approach is expressed mathematically as shown below:

Runoff Coefficient

$$R_T = C_T P_T \tag{4-1}$$

SCS Curve Number

$$R_T = \frac{\left(P_T - 0.2S\right)^2}{P_T + 0.8S} \tag{4-2}$$

$$S = \frac{1000}{CN} - 10 \tag{4-3}$$

where:

R_T = Rainfall excess for return period T, in inches

C_T = Runoff coefficient for return period T, dimensionless (see Section 4.3.1)

P_T = Precipitation depth for return period T, in inches

S = Maximum soil storage, in inches

CN = Watershed curve number (see Section 4.3.2)

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

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

4.3.1 RUNOFF COEFFICIENTS

The runoff coefficients for various land uses, soil types, and watershed slopes in Table 4-3 apply when a design storm with a return period of 10 years or less is considered. Runoff coefficients can be taken directly from the table for homogeneous land use. For mixed land uses, a weighted C value should be calculated as follows:

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

where:

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

n = Total number of areas with uniform runoff coefficients

C_i = Runoff coefficient for subarea i

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

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

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

$$C_T = C_{10} X_T \quad (4\text{-}5)$$

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 (see Table 4-3)

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

The value of C_T should never be increased above 1.0.

4.3.2 SCS CURVE NUMBERS

The following procedure determines the SCS curve number based on soil survey information published by the SCS:

1. Identify soil types using the SCS soil survey report for Madison County, Alabama (USDA, SCS, 1958).

2. Assign a hydrologic group to each soil type. The SCS has classified more than 4,000 soil series into four hydrologic soil groups, denoted by the letters A, B, C, and D. Soils in the A group have the lowest runoff potential; soils in the D group have the highest. Two letters can be assigned to certain wet soils that may be drained or undrained; the first letter applies to the drained condition and the second to the undrained. The following criteria are used for assigning dual groups:

   a. Soils are rated D in their natural condition.

   b. Drainage is feasible and practical.

   c. Drainage improves the soil hydrologic group by at least two classes (e.g., D to B).

   The hydrologic soil group classification considers only the soil properties that influence the

minimum rate of infiltration obtained for a bare soil after prolonged wetting.

3. Identify drainage areas with uniform soil type and land use conditions.

4. Use tables to select curve number values for each uniform drainage area identified in Step 3. A curve number value for Antecedent Moisture Condition (AMC) II can be selected using Tables 4-5 and 4-6. Table 4-5 provides curve numbers for selected urban and suburban land uses; Table 4-6 gives information on rural land uses.

   Several special factors should be considered when curve numbers are developed for an urban area, including the degree to which heavy equipment may compact the soil and the degree of surface and subsurface soil mixing caused by grading. In addition, the amount of barren pervious area (with little sod established) should be evaluated. Any one of these factors could move a soil normally placed in hydrologic group A or B to group B or C, respectively.

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

$$\overline{CN} = \frac{\sum_{i=1}^{n} CN_i A_i}{A_T} \qquad (4\text{-}6)$$

where:

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

n = Total number of areas with uniform soil type and land use conditions

CN_i = Curve number for subarea i with a given soil type and land use conditions (from Tables 4-5 and 4-6)

A_i = Land area for subarea i with uniform soil type and land use conditions, in acres or square miles

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

4.3.3 EXAMPLE PROBLEMS

Example 4-1. Rainfall Excess Using the Rational Method Runoff Coefficient

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

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

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

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

4. Industrial site on sandy soil (hydrologic soil group D) = 10 acres

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

1. From Figure 4-1, the 25-year, 24-hour rainfall depth is 6.21 inches.

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

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

as follows:

a. From Table 4-3, for rolling (2-7 percent) woodland areas on sandy soil and assuming mid-range values, $C_1 = 0.17$

b. From Table 4-3, for rolling (2-7 percent) woodland areas on clay soil and assuming mid-range values, $C_2 = 0.22$

c. From Table 4-3, for rolling (2-7 percent) multi-family residential areas on sandy soil and assuming mid-range values, $C_3 = 0.70$

d. From Table 4-3, for rolling (2-7 percent) industrial areas on sandy soil and assuming mid-range values, $C_4 = 0.85$

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

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

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

$$C_{25} = \bar{C}\, X_{25}$$

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

4. The rainfall excess is computed with Equation 4-1:

$$R_{25} = 0.58\ (6.21)$$

$$R_{25} = 3.6 \text{ inches}$$

Example 4-2. Rainfall Excess Using the SCS Curve Number

Using the watershed and proposed development from Example 4-1, calculate the rainfall excess for proposed conditions from a 25-year, 24-hour storm by the SCS curve number method.

1. From Figure 4-1, the 25-year, 24-hour rainfall depth is 6.21 inches.

2. The composite weighted curve number is computed from Equation 4-6 (repeated below)

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

as follows:

a. From Table 4-6, for woodlands with good ground cover on hydrologic soil group A, $CN_1 = 30$

b. From Table 4-6, for woodlands with good ground cover on hydrologic soil group D, $CN_2 = 77$

c. From Table 4-5, for residential areas with 1/8-acre average lot size on hydrologic soil group B, $CN_3 = 85$

d. From Table 4-5, for industrial areas on hydrologic soil group D, $CN_4 = 93$

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

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

3. From Equation 4-3, the maximum soil storage in inches is

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

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

$$R_{25} = \frac{[6.21 - 0.2(3.51)]^2}{6.21 + 0.8(3.51)}$$

$$R_{25} = \frac{(5.51)^2}{9.02}$$

$$R_{25} = 3.4 \text{ inches}$$

4.4 TIME OF CONCENTRATION

The time of concentration, t_c, is the time required for a hydraulic wave to travel across the watershed. It is often approximated as the time required for runoff to travel from the hydraulically most remote part of the watershed to the point of reference.

To calculate the time of concentration of a watershed, at least three runoff components should be considered:

1. Overland
2. Shallow channel (typically rill or gutter)
3. Main channel

The Velocity Method is a segmental approach that can be used to account for each of these components. The average velocity for each flow segment being evaluated is considered and a travel time calculated using the following equation:

$$t_i = \frac{L_i}{(60)\ v_i} \qquad (4\text{-}7)$$

where:

t_i = Travel time for flow segment i, in minutes

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

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

The time of concentration is then calculated, expressed as:

$$t_c = t_1 + t_2 + t_3 + \ldots t_i \qquad (4\text{-}8)$$

where:

t_c = Time of concentration, in minutes

t_1 = Overland flow travel time, in minutes

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

t_3 = Main channel travel time, in minutes

t_i = Travel time for the ith segment, in minutes

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

4.4.1 OVERLAND FLOW

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

$$t_1 = 0.93 \left(\frac{L^{0.6} n^{0.6}}{I^{0.4} S^{0.3}} \right) \qquad (4\text{-}9)$$

where:

t_1 = Overland flow travel time, in minutes

L = Overland flow length, in feet

n = Manning's roughness coefficient for overland flow (see Table 4-7)

I = Rainfall intensity, in inches/hour

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

Manning's n values reported in Table 4-7 were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations.

Equation 4-9 generally entails a trial and error process using the following steps:

1. Assume a trial value of rainfall intensity, I.

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

3. Use t_1 from Step 2 in Equation 4-8 to find the actual rainfall intensity for a storm duration of t_c (see Section 4.2.1).

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

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

$$t_1 = 0.42\left(\frac{(nL)^{0.8}}{P_2^{0.5}\,S^{0.4}}\right) \qquad (4\text{-}10)$$

where:

t_1 = Overland flow travel time, in minutes

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

and the remaining terms are as defined in Equation 4-9

Equation 4-10 is based on a single rainfall intensity from a 2-year, 24-hour rainfall event. For many cases, this approximate method will yield acceptable results; however, for very flat or very steep watersheds, overland flow travel time should be checked using the iterative method for Equation 4-9.

The overland flow time has been predicted by the Federal Aviation Administraion after analysis of airport areas to be as follows:

$$t = \frac{1.8(1.1-C)(L_o)^{.5}}{S^{1/3}}$$

t = overland flow time, minutes
c = rational runoff coefficient
S = watershed slope (avg.), percent
L_o = longest distance of overland flow to the collection point, feet

For irregurlarly shaped drainage areas, it will be necessary to evaluate several alternative overland flow distances.

4.4.2 SHALLOW CHANNEL FLOW

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

4.4.3 MAIN CHANNEL FLOW

Average velocities for main channel flow should be evaluated using bankfull conditions and the open channel procedures presented in Chapter 5. More than one main channel flow segment can be used where necessary.

4.4.4 EXAMPLE PROBLEM

Example 4-3. Time of Concentration Computation

The hydrologic flow path of the watershed described in Example 4-1 is about 2,000 feet in length with a total elevation change of about 40 feet. This flow path may be divided into the following three segments:

Segment No.	Type of Flow	Segment Length (ft)	Elevation Change (ft)	Slope (%)
1	Overland (woodland)	250	25	10
2	Shallow channel	750	13	1.7
3	Main channel	1,000	2	0.20
	n = 0.025			
	Width = 10 feet			
	Depth = 2 feet			
	Approximately rectangular channel			

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

1. Compute the overland flow travel time, t_1, using the SCS TR-55 method from Equation 4-10 (repeated below).

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

From Figure 4-1, the 2-year, 24-hour rainfall depth, P_2, is 3.78 inches.

From Table 4-7, for woodlands, n = 0.45.

$$t_1 = 0.42 \frac{(0.45 \times 250)^{0.8}}{(3.78)^{0.5} (0.10)^{0.4}}$$

t_1 = 24 minutes

2. Compute the shallow channel flow travel time.

From Figure 4-5, for a slope of 1.7 percent, the flow velocity is 2.6 feet/second.

By Equation 4-7,

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

$$t_2 = 4.8 \text{ minutes}$$

3. Compute the main channel flow travel time.

The flow velocity is given by Manning's Equation (Equation 5-5),

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

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

$$R = (2 \times 10)/[2(2)+10]$$

$$R = 20/14 = 1.43 \text{ feet}$$

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

$$v = 3.4 \text{ feet/second}$$

By Equation 4-7,

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

$$t_3 = 4.9 \text{ minutes}$$

4. Compute the watershed time of concentration.

From Equation 4-8,

$$t_c = 24 + 4.8 + 4.9$$

$$t_c = 34 \text{ minutes}$$

5. Check the time of concentration using the kinematic wave equation (Equation 4-9, repeated below).

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

From Step 1, n = 0.45. From Figure 4-1, for t_1 = 24 minutes from Step 1, the 10-year return frequency rainfall intensity is 4.05 inches per hour. Assuming the rainfall intensity to be 4.05 inches per hour, Equation 4-9 gives

$$t_1 = 0.93 \frac{(250)^{0.6}(0.45)^{0.6}}{(4.05)^{0.4}(0.10)^{0.3}}$$

t_1 = 18 minutes

From Step 2, t_2 = 4.8 minutes. From Step 3, t_3 = 4.9 minutes. From Equation 4-8,

t_c = 18 + 4.8 + 4.9

t_c = 28 minutes

From Figure 4-1, for t_c = 28 minutes, the 10-year return frequency rainfall intensity is 3.75 inches per hour.

Trial rainfall intensity and computed rainfall intensity do not agree.

6. Repeat computation assuming rainfall intensity is 3.75 inches per hour.

$$t_1 = 0.93 \frac{(250)^{0.6} (0.45)^{0.6}}{(3.75)^{0.4}(0.10)^{0.3}}$$

t_1 = 18.6 minutes

t_c = 28 minutes

From Figure 4-1, for t_c = 28 minutes, rainfall intensity is about 3.75 inches/hour.

Trial rainfall intensity and computed rainfall intensity agree.

7. Use t_c = 28 minutes.

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

4.5 PEAK RUNOFF RATES

4.5.1 GAGED SITES

Streamflow and flood frequency data for gaged watersheds are available from the U.S. Geological Survey (USGS) and TVA for selected streams, as presented in Table 4-8. In the event that streamflow measurements have not been analyzed to develop appropriate flood frequency curves, guidelines presented by the U.S. Water Resources Council (1981) should be followed.

The major drainage basins in Huntsville are Aldridge Creek, Spring Branch, and Indian Creek. Smaller drainage basins are tributary to Dry Creek, McDonald Creek, Bradford Creek, and Betts Spring Branch. The Aldridge Creek Valley watershed, which drains an area of 21.7 square miles, begins in Huntsville near U.S. Highway 431 and flows south to the Tennessee River. The valley is gently sloping, with broad flood plains and a number of small tributaries. Development in the valley is primarily residential and has occurred to some extent in the flood plain.

The Huntsville Spring Branch Valley drains 85.7 square miles and is a broad, undulating-to-rolling belt traversing highly developed municipal and community property. The numerous tributaries flowing into Huntsville Spring Branch include Pinhook Creek, Fagan Creek, Dallas Branch, and Broglan Branch. Pinhook Creek has been highly developed for a number of years.

The southern portion of the City is affected by the Tennessee River. The system of reservoirs operated by the TVA on the Tennessee River and its tributaries now comprises 33 major projects. Out of the 21 multi-purpose projects, 11 are on tributaries and 5 are on the main river upstream from Huntsville. Fourteen of these upstream projects reserve storage space for the control of flood flows. Their combined effect is to greatly alleviate flood damage threats at many locations in the Tennessee River basin.

4.5.2 RATIONAL METHOD

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

$$Q_T = C_T I_{t_c} A$$

where:

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

C_T = Runoff coefficient for return period T, expressed as the dimensionless ratio of rainfall excess to total rainfall

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

t_c = Time of concentration (see Section 4.4), in minutes

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

The following procedure is recommended for using the Rational Method:

1. Collect watershed data.

2. Calculate time of concentration using information in Section 4.4.

3. Use the IDF curves in Figure 4-1 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 4.2.

5. Compute the peak runoff rate for the return period T, using Equation 4-11.

4.5.3 TVA REGRESSION EQUATIONS

Flood frequency regression equations were developed by the TVA (1986) as part of an urban flood study for small, ungaged, urban watersheds in Huntsville with drainage areas between 60 and 3,200 acres and with imperviousness between 3 and 50 percent. These equations are applicable only where

the impact from manmade structures and channelization is insignificant.

The TVA regression equations and the TVA graphical flow frequency estimates presented in Section 4.5.4 were compared with results from USGS regression equations (Olin and Bingham, 1977). Similar results were obtained from watersheds with an imperviousness of about 20 percent, but the TVA relationships yielded better estimates for other imperviousness ranges. As a result, the TVA relationships are more representative of urban conditions in Huntsville and should be used instead of the USGS equations.

The TVA regression equations for the 2-, 10-, 50-, 100-, and 500-year return frequency flood events are shown in Table 4-9. Graphical solutions to these equations are presented in Figures 4-6 through 4-10. Tributary drainage area and imperviousness are the only watershed characteristics required as input for the equations. The tributary area, in square miles, should be determined from the best available sources, including the USGS 1"=2,000' topographic maps and the City's 1"=200' maps. Imperviousness, in percent, should be determined from actual or proposed impervious surfaces such as rooftops and paved areas. Unless specific development plans are available to provide better estimates, imperviousness associated with future development may be estimated using the typical values presented in Table 4-10.

4.5.4 TVA GRAPHS

Graphical flow frequency estimates were developed by the TVA (1986) for the selected stream reaches listed in Table 4-11 and apply only to the stream reach for which each was developed. The estimates, which are based on future development conditions, are presented in Figures 4-11 through 4-33 as a function of stream mile. Other hydrologic methods are required to develop flood hydrographs or to account for routing through stormwater facilities.

4.5.5 SCS TR-55 GRAPHICAL METHOD

The SCS has developed a graphical peak discharge method, presented in SCS TR-55 (1986), for estimating the peak runoff rate from watersheds with a single homogeneous land use.

The method is based on the results of computer analyses performed using TR-20 (USDA, SCS, 1983) and is subject to the following limitations:

1. Estimates of peak design flows only.

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

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

4. Presence of a single homogeneous subbasin. The procedure was developed using a DA of 1 square mile.

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

10. No consideration of hydrologic channel routing.

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

$$Q_t = q_u A_m R_T F_p \qquad (4\text{-}12)$$

where:

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

q_u = Unit peak discharge (from Figure 4-34), in cubic feet per second per square mile per inch (csm/inch)

A_m = Drainage area, in square miles

R_T = Rainfall excess for return period T, in inches (see Equation 4-2)

F_p = Pond and swamp adjustment factor (see Step 5 below)

Computation using the graphical peak discharge method proceeds as follows:

1. The 24-hour rainfall depth is determined from Figure 4-1 for the selected return frequency.

2. The runoff curve number, CN, is estimated using the procedures of Section 4.3.2 and rainfall excess is estimated using Equation 4-2.

3. The CN value is used to determine the initial abstraction, I_a, from Table 4-12 and the ratio I_a/P is then computed.

4. The watershed time of concentration is computed using the procedures in Section 4.4 and is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 4-34. If the ratio I_a/P lies outside the range shown in Figure 4-34, 0.1 to 0.5, another peak discharge method should be used.

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

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

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

6. The peak runoff rate is computed using Equation 4-12. The same procedure should be used to estimate pre- and post-development times of concentration when computing pre- and post-development peak discharge.

Since the 1986 version of TR-55 includes extensive revisions to the 1975 version, the earlier version is no longer appropriate for use. 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.

4.5.6 OTHER TECHNIQUES

Techniques other than those listed here may be used for computation of design flow rates, subject to the approval of the Public Works Department.

4.5.7 EXAMPLE PROBLEMS

Example 4-4. Rational Method Peak Runoff Rate

Use the Rational Method to compute the peak runoff rate from the watershed in Example 4-1 for a 25-year, 24-hour storm event.

1. Total area of watershed = 50 acres.

2. From Example 4-1, the 25-year runoff coefficient, C_{25}, is 0.58.

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

   a. By TR-55 (Equation 4-10), t_c = 34 minutes

   b. By kinematic wave equation (Equation 4-9), t_c = 28 minutes

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

a. For t_c = 34 minutes, I = 4.0 inches/hour

b. For t_c = 28 minutes, I = 4.4 inches/hour

5. The peak runoff rate is computed from Equation 4-11 as follows:

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

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

$$Q_{25} = 116 \text{ cfs}$$

b. Using the kinematic wave equation for t_c:

$$Q_{25} = (0.58)(4.4)(50)$$

$$Q_{25} = 128 \text{ cfs}$$

Example 4-5. TVA Regression Equation Peak Runoff Rate

The 50-acre watershed in Example 4-1 has an estimated imperviousness of about 30 percent resulting from urbanization. Determine the 10-, 50-, and 100-year peak runoff rates using the TVA regression equations.

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

a. Area = 50 acres or 0.078 square miles

b. Imperviousness = 30 percent

The area is slightly less than the lower limit of applicability (60 acres) and the imperviousness is within the range of applicability (3 to 50 percent). Although the area is less than 60 acres, use the regression equations for comparison.

2. From Table 4-9, the peak runoff rates are as follows:

a. $Q_{10} = 365\ (0.078)^{0.69}\ (30)^{0.23}$

$$Q_{10} = 137 \text{ cfs}$$

b. $Q_{50} = 680\ (0.078)^{0.69}\ (30)^{0.15}$

$Q_{50} = 195$ cfs

c. $Q_{100} = 855\ (0.078)^{0.69}\ (30)^{0.13}$

$Q_{100} = 229$ cfs

Example 4-6. SCS TR-55 Graphical Method Peak Runoff Rate

Use the SCS graphical peak discharge method to compute the peak runoff rate from a 25-year, 24-hour storm event from the watershed in Example 4-1. Use the watershed time of concentration computed in Example 4-3.

1. From Figure 4-1, the 25-year, 24-hour rainfall depth is 6.21 inches.

2. From Example 4-2, the composite curve number, $\overline{CN}$, is 74.

3. From Example 4-2, Step 3, the soil storage, S, is 3.51 inches.

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

$$R_{25} = \frac{[6.21 - 0.2(3.51)]^2}{6.21 + 0.8(3.51)}$$

$R_{25} = 3.4$ inches

5. From Table 4-12, the initial abstraction, I_a, is 0.703 inches.

6. The I_a/P ratio is

$I_a/P = 0.703/6.21$

$I_a/P = 0.11$

7. From Figure 4-34, for the time of concentration, t_c, from Example 4-3 of 34 minutes (0.57 hours) and with an I_a/P ratio of 0.11, the unit peak discharge, q_u, is 500 csm/inch of runoff.

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

9. The peak runoff rate is computed using Equation 4-12, as follows:

$$Q_{25} = (500)(50/640)(3.4)(1.0)$$

$$Q_{25} = 133 \text{ cfs}$$

4.6 FLOOD HYDROGRAPHS

Following a discussion of hydrograph terminology, flood hydrograph procedures for unit hydrograph theory and the SCS TR-55 (1986) tabular method are discussed below.

4.6.1 HYDROGRAPH TERMINOLOGY

A flood hydrograph is a continuous plot of the surface runoff flow rate versus time. Although historical streamflow data are preferred, they are not generally collected for the typical watershed; thus, synthetic methods for developing flood hydrographs are often required. If historical data are not available for a given basin, then data from a similar basin may be appropriate.

A typical hydrograph resulting from an isolated period of rainfall has the following major components, as illustrated in Figure 4-35:

1. Rising limb
2. Crest segment
3. Falling limb or recession curve

The shape of the rising limb is influenced primarily by the characteristics of the storm that produced the surface runoff. The point of inflection on the recession curve of the hydrograph is commonly assumed to mark when surface inflow to the channel system ceases (see Figure 4-36). Thereafter, the recession curve represents the withdrawal of water from storage within the watershed. As a result, the recession curve is largely independent of the storm and is

influenced instead by watershed characteristics, such as channel slope and storage availability.

The hydrograph terminology used throughout this manual is presented in Figure 4-35, along with appropriate SCS hydrograph equations. A rainfall excess hyetograph, which in this case is a single block of rainfall excess over duration, D, is shown in the upper part of Figure 4-35. The runoff hydrograph is presented directly below the rainfall excess hyetograph.

The area enclosed by the hyetograph and by the runoff hydrograph represents the same volume, Q, of direct runoff. The maximum flow rate of the hydrograph is the peak flow, q_p, while the time from the start of the hydrograph to q_p is the time to peak, t_p. The total time duration of the hydrograph is known as the time base, t_b. The watershed lagtime, t_L, is defined as the time from the center of mass of the rainfall excess to the runoff hydrograph peak. The following equation summarizes the relationship between the time parameters for the rising limb of a direct runoff hydrograph:

$$t_p = \frac{D}{2} + t_L \qquad (4\text{-}13)$$

where:

t_p = Time to peak or time of rise of the runoff hydrograph

D = Duration of runoff-producing rainfall

t_L = Watershed lagtime

The recession time for a hydrograph is the difference between the time base and the time to peak and can be expressed as:

$$t_r = t_b - t_p \qquad (4\text{-}14)$$

where:

t_r = Hydrograph recession time

t_b = Hydrograph time base

t_p = Time to peak or time of rise of the runoff hydrograph

4.6.2 UNIT HYDROGRAPH THEORY

The concept of a unit hydrograph, published by L. K. Sherman in 1932, provides a widely accepted basis for converting rainfall excess from a watershed to a runoff hydrograph. Although the tools and data available for developing unit hydrographs have become more extensive since Sherman first proposed unit hydrograph theory, the concept has not changed.

A unit hydrograph is defined as a runoff hydrograph that is produced by 1 inch of rainfall excess distributed uniformly over a watershed and occurring at a uniform rate during a specified period of time. Sherman originally used "unit" to denote the specified duration of rainfall excess for a particular unit hydrograph. The word "unit" is often misinterpreted as 1 inch or a "unit depth" of effective rainfall excess rather than as a "unit of time" for rainfall excess as originally intended.

The following assumptions constitute the basis of unit hydrograph theory:

1. The rainfall excess is uniformly distributed within its unit duration or specified period of time.

2. The rainfall excess is uniformly distributed in space over a particular watershed.

3. The time base for a direct runoff hydrograph due to a rainfall excess of unit duration is constant.

4. The ordinates of the direct runoff hydrographs, when a common unit duration is considered, are directly proportional to the total volume of direct runoff represented by each hydrograph (principle of linearity or superposition).

5. For a given drainage basin, the hydrograph of runoff caused by a unit duration and volume of

rainfall excess is invariable (principle of time invariance).

These assumptions cannot be precisely applied to natural precipitation and drainage basin characteristics. However, experience has shown that the unit hydrograph method gives results that are sufficiently accurate for most drainage problems.

The two fundamental assumptions that must always be considered when applying unit hydrograph theory are the principle of linearity (Assumption 4) and the principle of time invariance (Assumption 5). Theoretically, each increment of rainfall excess can be routed through the subject watershed in accordance with the principle of linearity. In practice, this means that the product of a rainfall excess volume and the sequence of unit hydrograph ordinates (i.e., runoff rates in cubic feet per second per inch of rainfall excess) produces an estimate of the runoff hydrograph for that volume of rainfall excess. In addition, the principle of linearity allows individual runoff hydrographs developed from a sequence of individual rainfall excess volumes (i.e., a design storm of rainfall excess increments arranged in units of time equal to the unit duration) to be superimposed and added when estimating a total runoff hydrograph. The principle of time invariance requires that the hydrologic characteristics of the drainage basin be fixed or specified for a particular unit hydrograph. Land development and channel improvements are typical activities that violate the principal of time invariance.

4.6.3 UNIT HYDROGRAPH PROCEDURE

A flood hydrograph can be developed through the following steps using the unit hydrograph procedure:

1. Develop a unit hydrograph for the subject watershed using an appropriate procedure (see discussion below).

2. Develop a design storm hyetograph using the time interval for which the unit hydrograph was developed and an appropriate procedure (as presented in Section 4.2).

3. Develop a rainfall excess hyetograph using an appropriate procedure as presented in Section 4.3.

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

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

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

$$V = \frac{12\ \Delta t\ \Sigma q_i}{A(43,560)} \qquad (4\text{-}15)$$

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

4.6.4 SYNTHETIC UNIT HYDROGRAPHS

Unit hydrographs should be developed using observed rainfall and streamflow records when they are available. Procedures for deriving unit hydrograph parameters from observed data are well-documented in publications by Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), Chow (1964), and the

USDOT, FHWA (HEC-19, 1984). When observed data are not available for deriving unit hydrograph parameters, as is often the case, synthetic procedures are required. The SCS dimensionless unit hydrograph approach is presented below.

Two types of dimensionless unit hydrographs have been developed by the SCS; the first has a curvilinear shape as shown in Figure 4-37 and the second is a triangular approximation to that curvilinear shape (see Figure 4-36). In both cases, once the time to peak and peak flow for a particular unit hydrograph have been defined, the entire shape of the unit hydrograph can be estimated using the dimensionless unit hydrograph ratios and mass data, as shown in Table 4-13.

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

1. Estimate the time of concentration, t_c, using an appropriate method (see Section 4.4).

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

$$\Delta D = 0.133\ t_c \qquad (4\text{-}16)$$

where:

ΔD = Incremental duration of runoff producing rainfall, in minutes

t_c = Time of concentration, in minutes

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

$$t_p = \frac{\Delta D}{2} + 0.6\ t_c \qquad (4\text{-}17)$$

where:

t_p = Time to peak, in minutes

ΔD = Incremental duration of runoff producing rainfall, in minutes

t_c = Time of concentration, in minutes

4. Calculate the peak flow rate, q_p, as follows.

$$q_p = 60 \frac{B\ A}{t_p} \quad (4\text{-}18)$$

where:

q_p = Peak flow rate, in cfs

B = Hydrograph shape factor, ranging from 300 for flat swampy areas to 600 in steep terrain. SCS standard B value = 484.

A = Drainage area, in square miles

t_p = Time to peak, in minutes

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

6. Using Table 4-13 or Figure 4-37, find the q/q_p ratio for the appropriate t/t_p ratios calculated in Step 5.

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

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

The SCS triangular dimensionless unit hydrograph procedure is identical to the curvilinear procedure presented above. However, to draw the required unit hydrograph, only t/t_p ratios of 0, 1, and 2.67 are needed. When applying the triangular dimensionless unit hydrograph, the time of concentration, t_c, is computed using Equation 4-16, the time to peak, t_p, is computed using Equation 4-17, and the time base, t_b, is computed as follows:

$$t_b = 2.67\ t_p \quad (4\text{-}19)$$

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 the time to peak and peak flow rate for that unit hydrograph. Therefore, a triangular unit hydrograph would probably produce approximately the same synthetic runoff hydrograph as a curvilinear unit hydrograph.

4.6.5 SCS TR-55 TABULAR METHOD

The tabular method developed by SCS can be used to estimate flood hydrographs from watersheds that can be divided into relatively homogeneous land uses. The method is based on the results of computer analyses performed using TR-20 (USDA, SCS, 1983) and can be used to approximate the effects of hydrologic channel routing. Details for applying the procedure are contained in SCS TR-55 (1986). Limitations of the tabular method are summarized as follows:

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

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

3. DAs of individual subareas 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 = 484.

7. The reach travel time, t_T, is 0 to 3 hours.

Because the 1986 version of TR-55 includes extensive revisions to the 1975 version, the earlier version is no longer appropriate. 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. Micro-

computer diskettes with TR-55 procedures are also available under catalog number PB87-101598.

4.6.6 OTHER METHODS

Other methods of developing flood hydrographs may be used, subject to approval by the Public Works Department.

4.6.7 EXAMPLE PROBLEMS

Example 4-7. SCS Dimensionless Unit Hydrograph

Develop a synthetic unit hydrograph for the watershed in Example 4-1 using the SCS curvilinear approach.

1. From Example 4-3, the watershed time of concentration, t_c, is 34 minutes.

2. From Equation 4-16, the incremental duration of runoff producing rainfall, ΔD, is

   $\Delta D = 0.133\ (34)$
   $\Delta D = 4.5$ minutes or 0.08 hours

3. From Equation 4-17, the time to peak, t_p, is

   $$t_p = \frac{4.5}{2} + 0.6(34)$$

   $t_p = 23$ minutes

4. From Equation 4-18, the unit hydrograph peak flow rate, q_p, is

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

   $q_p = 99$ cfs

5. From Figure 4-37 or Table 4-13, determine the q/q_p ratio for appropriate t/t_p ratios and calculate the unit hydrograph ordinates by multiplying the q/q_p ratio by q_p as follows:

Time, t (hours)	t/t_p (t/0.4 hours)	q/q_p (q/99 cfs)	q (cfs)
0.00	0.00	0.000	0
0.08	0.20	0.100	10
0.16	0.40	0.310	31
0.24	0.60	0.660	65
0.32	0.80	0.930	92
0.40	1.00	1.000	99
0.48	1.20	0.930	92
0.56	1.40	0.780	77
0.64	1.60	0.560	55
0.72	1.80	0.390	39
0.80	2.00	0.280	28
0.88	2.20	0.207	20
0.96	2.40	0.147	15
1.04	2.60	0.107	11
1.12	2.80	0.077	8
1.20	3.00	0.055	5
1.28	3.20	0.040	4
1.36	3.40	0.029	3
1.44	3.60	0.021	2
1.52	3.80	0.015	1
1.60	4.00	0.011	1
1.68	4.20	0.008	1
1.76	4.40	0.005	0
1.84	4.60	0.002	0
1.92	4.80	0.001	0
2.00	5.00	0.000	0
			660

6. Check that the unit hydrograph volume equals 1 inch using Equation 4-15:

$$V = \frac{12(0.08)(3,600)(660)}{(50)\ (43,560)}$$

$$V = 1.05 \approx 1 \text{ (close enough)}$$

Example 4-8. Flood Hydrograph Using Unit Hydrograph Theory

Develop a synthetic runoff hydrograph for a 25-year, 2-hour design storm for the watershed described in Example 4-1 using the unit hydrograph developed in Example 4-7.

1. Develop a balanced storm hyetograph and cumulative mass curve using the IDF curve for a 25-year storm from Figure 4-1, as shown in Table 4-14.

2. Develop a rainfall excess hyetograph using the SCS curve number approach. From Example 4-2, CN is 74 and S is 3.51 inches. From Figure 4-1, the 25-year, 2-hour rainfall depth is 3.78 inches.

   Apply Equation 4-2 to the cumulative depth as follows to obtain the rainfall excess hyetograph:

Time, t (hours)	Cumulative Depth (inches)	Cumulative Rainfall Excess (inches)	Rainfall Excess Hyetograph (inches)
0.00	0.00	0.00	0.00
0.08	0.01	0.00	0.00
0.16	0.04	0.00	0.00
0.24	0.12	0.00	0.00
0.32	0.20	0.00	0.00
0.40	0.30	0.00	0.00
0.48	0.41	0.00	0.00
0.56	0.51	0.00	0.00
0.64	0.63	0.00	0.00
0.72	0.80	0.00	0.00
0.80	0.98	0.02	0.02
0.88	1.24	0.07	0.05
0.96	1.68	0.21	0.14
1.04	2.33	0.51	0.30
1.12	2.66	0.70	0.19
1.20	2.90	0.85	0.15
1.28	3.06	0.95	0.10
1.36	3.20	1.04	0.09
1.44	3.31	1.11	0.07
1.52	3.42	1.18	0.07
1.60	3.52	1.25	0.07
1.68	3.62	1.32	0.07
1.76	3.70	1.38	0.06
1.84	3.75	1.42	0.04
1.92	3.77	1.43	0.01
2.00	3.78	1.44	0.01

3. Route the rainfall excess hyetograph through the watershed using the unit hydrograph developed in Example 4-7. 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 rainfall excess occurred. The rainfall excess hydrograph is obtained by summing the ordinates of each routed incremental hydrograph, as shown in Table 4-15.

4. Check that hydrograph volume is equal to the rainfall excess.

$$V = \frac{12(0.08)(3{,}600)(702)}{50\ (43{,}560)} = 1.48 \text{ inches (close enough)}$$

4.7 HYDROLOGIC CHANNEL ROUTING

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

4.7.1 MUSKINGUM METHOD

The Muskingum Method is applied with the following steps:

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

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

$$v = \frac{1}{B}\left(\frac{Q(Y + \Delta Y) - Q(Y)}{\Delta Y}\right) \qquad (4\text{-}20)$$

where:

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

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

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

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

B = Top width of water surface, in feet

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

$$\Delta L_{min} = \frac{Q}{BS_o v} \qquad (4\text{-}21)$$

where:

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

Q = Flow rate, in cfs

B = Top width of water surface, in feet

S_o = Slope of channel bottom, in feet/foot

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

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

$$K = \frac{\Delta L}{v} \qquad (4\text{-}22)$$

where:

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

ΔL = Channel routing segment length, in feet

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

5. Estimate the value of X using the equation:

$$X = 0.5\left(1 - \frac{Q}{BS_o v \Delta L}\right) \qquad (4\text{-}23)$$

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:

$$\frac{K}{3} \leq \Delta t \leq K \qquad (4\text{-}24)$$

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

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (4\text{-}25)$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \quad (4\text{-}26)$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \quad (4\text{-}27)$$

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:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \quad (4\text{-}28)$$

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 then performed by repetitively solving Equation 4-28, assigning the current value of O_2 to O_1, and determining a new value of O_2. This sequence of calculations continues until the entire inflow hydrograph is routed through the channel.

4.7.2 SCS TR-55 TABULAR METHOD

The SCS tabular method can be used to approximate the effects of hydrologic channel routing on flood hydrographs. Details for applying the procedure are contained in SCS TR-55 (1986). Limitations are summarized in Section 4.6.5. The 1986 version of TR-55 supersedes the 1975 version and should be used in place of the older publication.

Table 4-1
GUIDELINES FOR SELECTING HYDROLOGIC PROCEDURES

Hydrologic Method	Section of Manual	Peak Flow	Hydrograph
1. Rational Method[a]	4.5.2	Yes	No
2. SCS TR-55 Graphical	4.5.5	Yes	No
3. SCS TR-55 Tabular	4.6.5	Yes	Yes
4. TVA Regression Equations	4.5.3	Yes	No
5. TVA Graphs	4.5.4	Yes	No
6. Unit Hydrograph	4.6.3	Yes	Yes

Limits of Application

	Design Storm	Time of Concentration, t_c	Drainage Area, DA	Imperviousness	I_a/P
1. Rational Method[a]	t_c	5 min $\leq t_c \leq$ 30 min	$\leq$ 50 acres	0-100%	N/A
2. SCS TR-55 Graphical	24-hr Type II	0.1 hr $\leq t_c \leq$ 10 hr	b	40 $\leq$ CN $\leq$ 98	.1-.5
3. SCS TR-55 Tabular	24-hr Type II	0.1 hr $\leq t_c \leq$ 2 hr	c	40 $\leq$ CN $\leq$ 98	.1-.5
4. TVA Regression Equations	N/A	N/A	0.1 sq. mi. $\leq$ DA $\leq$ 5 sq. mi.	3-50%	N/A
5. TVA Graphs	N/A	N/A	d	d	N/A
6. Unit Hydrograph	Any	> 0	> 0	0-100%	N/A

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

[b]A single homogeneous subbasin is required. The procedure was developed using 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 computer analysis with a DA of 1 square mile.

[d]Applicable only to the stream reaches modeled as identified on graphs.

N/A = Not applicable.

GENERAL NOTE:

At the discretion of the Engineer of Public Works, design discharge values available from other projects or studies shall be used in the design or analysis of selected stormwater management facilities.

Table 4-2
MONTHLY AND ANNUAL PRECIPITATION AVERAGES FOR
HUNTSVILLE, ALABAMA, BETWEEN 1959 AND 1986

	Precipitation (inches)												
Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
1959	4.36	3.62	3.29	4.44	5.53	3.65	3.17	3.55	3.49	4.69	2.74	5.74	48.27
1960	6.18	2.58	8.31	2.09	3.35	3.31	3.49	5.51	4.50	4.11	3.71	3.51	50.65
1961	1.76	9.13	8.45	2.21	4.68	4.46	2.16	5.39	0.49	1.46	3.71	12.17	56.07
1962	8.53	8.38	6.26	4.57	1.55	4.68	2.17	0.72	3.78	3.13	5.47	3.33	52.57
1963	3.59	1.88	10.45	9.05	3.09	6.54	6.08	1.13	0.97	T	4.44	6.28	53.51
1964	5.34	4.00	10.75	12.55	2.13	4.28	7.42	2.54	4.49	3.25	4.00	3.45	64.20
1965	3.61	5.67	8.20	3.15	3.10	5.79	4.44	3.38	3.60	0.65	0.64	0.76	42.99
1966	4.29	4.93	2.16	4.04	6.23	2.90	3.28	6.30	4.53	3.36	1.72	3.17	46.91
1967[a]	1.83	4.02	1.63	3.90	6.48	4.20	14.81	6.24	2.87	3.89	4.12	11.06	65.05
1968	6.01	1.14	5.63	4.05	5.87	0.77	2.90	0.94	4.15	1.94	3.84	5.19	42.43
1969	7.38	6.97	3.00	6.79	6.85	6.15	4.36	3.38	4.39	2.49	2.47	9.48	63.71
1970	1.71	2.75	7.40	9.08	3.06	6.61	2.44	4.04	1.80	4.76	2.58	4.73	50.96
1971	5.25	9.57	6.64	3.29	3.59	1.34	4.70	2.97	5.99	0.77	1.82	9.89	55.82
1972	7.97	2.58	4.94	4.02	3.53	1.90	6.57	2.25	3.83	2.20	5.07	6.62	51.48
1973	6.33	4.84	14.77	3.86	4.38	2.24	9.44	0.93	1.78	3.41	6.79	8.44	67.21
1974	10.50	7.73	3.56	5.57	9.03	3.00	5.14	3.91	3.23	1.36	5.25	6.01	64.29
1975	5.38	4.96	12.17	3.17	6.48	4.61	8.32	3.57	5.11	12.06	3.59	3.55	72.97[b]
1976	3.94	3.36	6.84	1.84	6.60	7.30	4.35	1.27	7.40	4.38	2.61	3.47	53.36
1977	3.71	3.40	9.50	6.68	4.13	2.40	3.41	2.46	9.03	5.31	11.53	2.25	63.81
1978	3.86	0.59	4.81	2.89	6.30	3.87	5.64	3.45	2.83	0.79	3.63	7.68	46.34
1979	5.95	5.49	5.74	7.46	4.03	2.60	6.49	3.04	5.38	2.91	6.65	2.20	57.94
1980	5.50	1.79	17.00	5.04	9.08	2.09	1.87	3.55	9.78	2.46	5.83	0.91	64.90
1981	1.80	3.09	5.93	2.87	3.55	1.99	3.72	4.37	0.78	4.07	5.41	4.23	41.81[b]
1982	10.92	6.12	2.70	10.02	3.89	3.10	4.09	3.92	0.55	1.98	5.49	6.87	59.65
1983	2.26	5.17	5.02	9.95	11.88	5.87	0.79	1.35	2.17	2.59	8.60	11.74	67.39
1984	3.66	3.65	5.86	5.85	7.51	3.40	4.51	4.58	0.76	6.67	6.89	2.69	56.03
1985	4.65	4.18	3.48	3.01	6.56	4.72	6.49	7.19	3.26	7.01	1.86	2.65	55.06
1986	1.32	4.43	3.26	0.42	5.68	4.85	3.07	9.81	5.10	3.90	7.87	4.86	54.57
Record Mean	4.91	4.50	6.71[b]	5.07	5.29	3.88	4.83	3.63	3.79	3.41[b]	4.58	5.46	56.07

Reference: U.S. Department of Commerce (1986).

Note: T = Trace

[a]Station change from old municipal airport off U.S. Highway 231 to Huntsville/Madison County Airport off Alabama Highway 20.

[b]Extreme data points (maximum and minimum).

Table 4-3
RUNOFF COEFFICIENTS[a] FOR A DESIGN STORM RETURN PERIOD OF 10 YEARS OR LESS

Slope	Land Use	Sandy Soils Min.	Sandy Soils Max.	Clay Soils Min.	Clay Soils Max.
Flat (0-2%)	Woodlands	0.10	0.15	0.15	0.20
	Pasture, grass, and farmland[b]	0.15	0.20	0.20	0.25
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavement[c]	0.75	0.95	0.90	0.95
	SFR: ½-acre lots and larger	0.30	0.35	0.35	0.45
	Smaller lots	0.35	0.45	0.40	0.50
	Duplexes	0.35	0.45	0.40	0.50
	MFR: Apartments, townhouses, and condominiums	0.45	0.60	0.50	0.70
	Commercial and industrial	0.50	0.95	0.50	0.95
Rolling (2-7%)	Woodlands	0.15	0.20	0.20	0.25
	Pasture, grass, and farmland[b]	0.20	0.25	0.25	0.30
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavement[c]	0.80	0.95	0.90	0.95
	SFR: ½-acre lots and larger	0.35	0.50	0.40	0.55
	Smaller lots	0.40	0.55	0.45	0.60
	Duplexes	0.40	0.55	0.45	0.60
	MFR: Apartments, townhouses, and condominiums	0.50	0.70	0.60	0.80
	Commercial and industrial	0.50	0.95	0.60	0.95
Steep (7%+)	Woodlands	0.20	0.25	0.25	0.30
	Pasture, grass, and farmland[b]	0.25	0.35	0.30	0.40
	Rooftops and pavement	0.95	0.95	0.95	0.95
	Pervious pavement[c]	0.85	0.95	0.90	0.95
	SFR: ½-acre lots and larger	0.40	0.55	0.50	0.65
	Smaller lots	0.45	0.60	0.55	0.70
	Duplexes	0.45	0.60	0.55	0.70
	MFR: Apartments, townhouses, and condominiums	0.60	0.75	0.65	0.85
	Commercial and industrial	0.60	0.95	0.65	0.95

Reference: DeKalb County, Georgia (1976).

Note: SFR = Single-family residential
MFR = Multi-family residential

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

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

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

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

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

Reference: Wright-McLaughlin Engineers (1969).

Table 4-5
RUNOFF CURVE NUMBERS FOR URBAN AREAS[a]

Cover Description: Cover Type and Hydrologic Condition	Average Percent Impervious Area[b]	Curve Numbers for Hydrologic Soil Group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawn, parks, golf courses, cemeteries, etc.)[c]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (townhouses)	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 are determined using cover types similar to those in Table 4-5)					

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

[a]Average runoff condition, Antecedent Moisture Condition II, and $I_a = 0.2S$.

[b]The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

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

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

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Table 4-6
RUNOFF CURVE NUMBERS FOR RURAL AREAS[a]

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

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

[a]Average runoff condition, Antecedent Moisture Condition II, 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.

Table 4-7

OVERLAND FLOW MANNING'S n VALUES

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

Notes: These values were determined specifically for overland flow conditions and are not appropriate for conventional open channel flow calculations. See Chapter 5 for open channel flow procedures.

All values are from Engman (1983), unless noted otherwise.

[a]Woolhiser (1975).

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

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

Table 4-8
STREAM GAGES IN THE VICINITY OF HUNTSVILLE

Gaging Station	Number	Drainage Area (square miles)	Period of Record	Agency
Walker Branch near Plevna	03574796	0.44	1971-1975[a]	USGS
Morris Branch near Toney	03574840	1.43	1971-1975[a]	USGS
Straight Ditch at Huntsville	03574852	0.17	1971-1975[a]	USGS
Glover Cove Creek near Owens Cross Road	03575340	3.52	1971-1975[a]	USGS
Aldridge Creek at Huntsville	03575686	1.15	1971-1975[a]	USGS
Aldridge Creek near Lily Flagg	03575696	13.9	1971-1975[a]	USGS
Aldridge Creek near Farley (Mt. Gap Road)	03575700	14.1	1960-1964[b] 1986-present	USGS
Pinhook Creek at Clinton	03575890	22.5	1986-present	USGS
Indian Creek near Madison	03575830	49.0	1959-1966[b] 1966-1971[c] 1971-1975[a]	USGS USGS USGS
Five Points Ditch at Huntsville (Tributary to Pinhook Creek at River Mile 15.15)	03575880	0.62	1971-1975[a]	USGS
Pinhook Creek at Huntsville	03575890	22.5	1966-1968[b] 1971-1973[a]	TVA USGS
Pinehaven Ditch at Huntsville	03575910	0.16	1971-1975[a]	USGS
Broglan Branch at Holmes Avenue at Huntsville	03575930	8.87	1971-1975[a]	USGS
Broglan Branch at Clinton	03575933	9.51	1986-present	USGS
Huntsville Spring Branch at Johnson Road at Huntsville	03575950	41.8	1967-1968[b] 1971-1975[a] 1986-present	TVA USGS
McDonald Creek at Pailon Road	03575980	--	1986-present	USGS
Dallas Branch at Coleman Street	0357587728	2.79	1986-present	USGS

USGS = U.S. Geological Survey

TVA = Tennessee Valley Authority

[a]Flood hydrograph partial-record station.

[b]Continuous-record station.

[c]Crest-stage partial-record station.

Table 4-9
TVA REGRESSION EQUATIONS FOR SMALL, UNGAGED, URBAN WATERSHEDS IN HUNTSVILLE, ALABAMA[a]

Equation	Standard Error[b] of Estimate (%)
$Q_2 = 156\ A^{0.69}\ I^{0.32}$	44
$Q_{10} = 365\ A^{0.69}\ I^{0.23}$	44
$Q_{50} = 680\ A^{0.69}\ I^{0.15}$	--
$Q_{100} = 855\ A^{0.69}\ I^{0.13}$	44
$Q_{500} = 1{,}400\ A^{0.69}\ I^{0.09}$	--

where:

Q_T = Estimated discharge, in cfs, for the indicated recurrence interval, T

A = Tributary drainage area, in square miles

I = Watershed imperviousness, in percent

Reference: TVA (1986).

[a]Tributary drainage area ranged from 0.1 to 5 square miles and imperviousness values ranged from 3 to 50 percent.

[b]The 50-year regression equation was developed by interpolation and the 500-year regression equation was developed by extrapolation; as a result, the standard error was not computed for these relationships.

Table 4-10
TYPICAL IMPERVIOUSNESS FOR
DEVELOPMENT DENSITY IN HUNTSVILLE, ALABAMA[a]

Development Density	Typical Imperviousness (%)
Single-Family Residential	37
Multi-Family Residential	49
Light Industrial and Institutional	32
Medium Industrial (Research Park)	50
Heavy Industrial and Commercial	88

Reference: TVA (1986).

[a]Use actual development plan to estimate imperviousness when available.

Table 4-11
STREAM REACHES WITH AVAILABLE
MODELED FLOW FREQUENCY DATA

Stream	Reach (miles)	Figure
Pinhook Creek--Existing Channel Conditions	14.63-19.07	4-11
Pinhook Creek--Proposed Channel Conditions	14.63-19.07	4-12
East Fork Pinhook Creek	0-1.68	4-13
West Fork Pinhook Creek	0-2.54	4-14
Blue Spring Creek	0-1.90	4-15
Normal Branch	0-3.28	4-16
Dallas Branch	0-2.74	4-17
Dallas Branch Bypass	0-1.37	4-18
Fagan Creek	0-3.16	4-19
Dry Creek	0-2.60	4-20
Broglan Branch--Existing Channel Conditions	0-4.48	4-21
Broglan Branch--Proposed Channel Conditions	0-4.48	4-22
McDonald Creek	3.67-9.07	4-23
Tributary to McDonald Creek at Mile 6.14	0-1.00	4-24
Sherwood Branch	0-3.14	4-25
Tributary to Sherwood Branch at Mile 1.42	0-0.5	4-26
Huntsville Spring Branch--Existing Channel Conditions	9.75-14.63	4-27
Huntsville Spring Branch--Proposed Channel Conditions	9.75-14.63	4-28
Indian Creek	12.80-17.52	4-29
Aldridge Creek	0-12.12	4-30
Aldridge Creek Tributaries		
Unnamed Tributary at Mile 3.33	0-0.26	4-31
Mountain Gap at Mile 4.62	0-0.92	4-31
Gourdneck at Mile 3.44	0-0.64	4-31
Camelot at Mile 4.63	0-0.62	4-31
Sunset Cove at Mile 6.09	0-0.75	4-32
Esslinger Cove at Mile 7.33	0-1.01	4-32
Weatherly Cove at Mile 5.94	0-0.70	4-32
Drake Cove at Mile 8.72	0-1.36	4-32
Bailey Cove at Mile 9.23	0-1.01	4-33
Toney Hollow at Mile 10.35	0-0.61	4-33
Martin Hollow at Mile 9.87	0-1.11	4-33
Greenwycke Village at Mile 10.83	0-0.86	4-33

Reference: TVA (1986).

Note: Stream mileages are shown on the drainageway maps.

Table 4-12
I_a VALUES FOR RUNOFF CURVE NUMBERS

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

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

Table 4-13
SCS DIMENSIONLESS UNIT HYDROGRAPH
RATIOS AND MASS DATA

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

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

Table 4-14
BALANCED STORM HYETOGRAPH AND CUMULATIVE MASS CURVE COMPUTATION, EXAMPLE 4-8

Time, t (hours)	Intensity, i (inches/hour)	Rainfall Depth (inches)	Increm. Depth (inches)	Balanced Depth (inches)	Cumulative Depth (inches)	Rainfall Excess (inches)	Rainfall Hyetograph (inches)
0.00	9.50	0.00	0.00	0.00	0.00	0.00	0.00
0.08	8.15	0.65	0.65	0.01	0.01	0.00	0.00
0.16	6.80	1.09	0.44	0.03	0.04	0.00	0.00
0.24	5.92	1.42	0.33	0.08	0.12	0.00	0.00
0.32	5.25	1.68	0.26	0.09	0.20	0.00	0.00
0.40	4.80	1.92	0.24	0.10	0.30	0.00	0.00
0.48	4.38	2.10	0.18	0.11	0.41	0.00	0.00
0.56	4.04	2.26	0.16	0.10	0.51	0.00	0.00
0.64	3.80	2.43	0.17	0.12	0.63	0.00	0.00
0.72	3.57	2.57	0.14	0.17	0.80	0.00	0.00
0.80	3.36	2.69	0.12	0.18	0.98	0.02	0.02
0.88	3.18	2.80	0.11	0.26	1.24	0.07	0.05
0.96	3.02	2.90	0.10	0.44	1.68	0.21	0.14
1.04	2.89	3.01	0.11	0.65	2.33	0.51	0.30
1.12	2.78	3.11	0.11	0.33	2.66	0.70	0.19
1.20	2.68	3.22	0.10	0.24	2.90	0.85	0.15
1.28	2.59	3.32	0.10	0.16	3.06	0.95	0.10
1.36	2.51	3.41	0.10	0.14	3.20	1.04	0.09
1.44	2.43	3.50	0.09	0.11	3.31	1.11	0.07
1.52	2.36	3.59	0.09	0.11	3.42	1.18	0.07
1.60	2.29	3.66	0.08	0.10	3.52	1.25	0.07
1.68	2.21	3.71	0.05	0.10	3.62	1.32	0.07
1.76	2.13	3.74	0.03	0.09	3.70	1.38	0.06
1.84	2.05	3.76	0.02	0.05	3.75	1.42	0.04
1.92	1.96	3.76	0.01	0.02	3.77	1.43	0.01
2.00	1.89	3.78	0.01	0.01	3.78	1.44	0.01

Table 4-15
HYDROGRAPH COMPUTATION, EXAMPLE 4-8

Unit Hydrograph (cfs/inch)	0	10	31	65	92	99	92	77	55	39	28	20	15	11	8	5	4	3	2	1	1	1	0	0	0	0
Runoff Time[a] (minutes)	0	5	10	14	19	24	29	34	38	43	48	53	58	62	67	72	77	82	86	91	96	101	106	110	115	120
Incremental Rainfall Excess (inches)																										
0.02	0	0.2	0.6	1.2	1.6	1.8	1.6	1.4	1.0	0.7	0.5	0.4	0.3	0.2	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.05		0.0	0.5	1.6	3.3	4.7	5.0	4.7	3.9	2.8	2.0	1.4	1.0	0.8	0.6	0.4	0.3	0.2	0.2	0.1	0.1	0.1	0.1	0.0	0.0	0.0
0.14			0.0	1.4	4.3	9.1	12.9	13.9	12.9	10.8	7.7	5.5	3.9	2.8	2.1	1.5	1.1	0.7	0.6	0.4	0.3	0.1	0.1	0.1	0.0	0.0
0.30				0.0	3.0	9.4	19.7	27.9	30.0	27.9	23.3	16.7	11.8	8.5	6.1	4.5	3.3	2.4	1.5	1.2	0.9	0.6	0.3	0.3	0.3	0.0
0.19					0.0	1.9	5.8	12.1	17.2	18.5	17.2	14.4	10.3	7.3	5.2	3.7	2.8	2.1	1.5	0.9	0.7	0.6	0.4	0.2	0.2	0.2
0.15						0.0	1.5	4.5	9.4	13.4	14.4	13.4	11.2	8.0	5.7	4.1	2.9	2.2	1.6	1.2	0.7	0.6	0.4	0.3	0.1	0.1
0.10							0.0	1.0	3.1	6.6	9.3	10.0	9.3	7.8	5.6	3.9	2.8	2.0	1.5	1.1	0.8	0.5	0.4	0.3	0.2	0.1
0.09								0.0	0.9	2.8	5.9	8.3	8.9	8.3	6.9	5.0	3.5	2.5	1.8	1.4	1.0	0.7	0.5	0.4	0.3	0.2
0.07									0.0	0.7	2.3	4.8	6.8	7.3	6.8	5.7	4.0	2.9	2.1	1.5	1.1	0.8	0.6	0.4	0.3	0.2
0.07										0.0	0.7	2.2	4.7	6.6	7.1	6.6	5.5	4.0	2.8	2.0	1.4	1.1	0.8	0.6	0.4	0.3
0.07											0.0	0.7	2.2	4.6	6.5	7.0	6.5	5.4	3.9	2.7	2.0	1.4	1.1	0.8	0.6	0.4
0.07												0.0	0.7	2.1	4.5	6.3	6.8	6.3	5.3	3.8	2.7	1.9	1.4	1.0	0.8	0.5
0.06													0.0	0.6	1.9	4.0	5.7	6.1	5.7	4.8	3.4	2.4	1.7	1.2	0.9	0.7
0.04														0.0	0.4	1.1	2.3	3.3	3.5	3.3	2.7	2.0	1.4	1.0	0.7	0.5
0.01															0.0	0.1	0.4	0.9	1.3	1.4	1.3	1.1	0.8	0.6	0.4	0.3
0.01																0.0	0.1	0.2	0.5	0.7	0.7	0.7	0.6	0.4	0.3	0.2
Total Runoff (cfs)	0	0	1	4	12	27	47	65	78	84	83	78	71	65	59	54	48	41	33	26	19	14	10	7	5	4

Total = 935

[a]Time zero is when runoff first begins.

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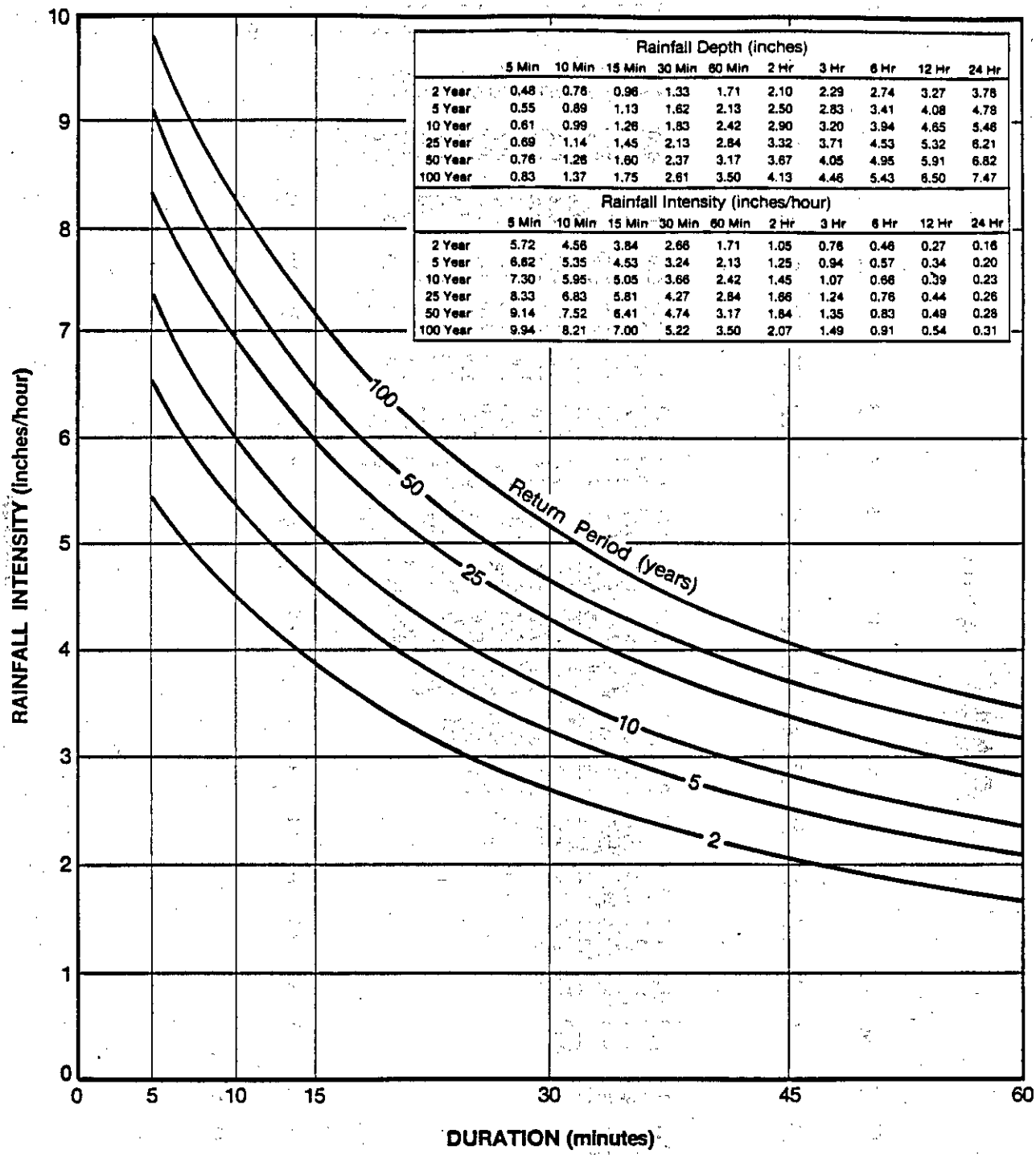
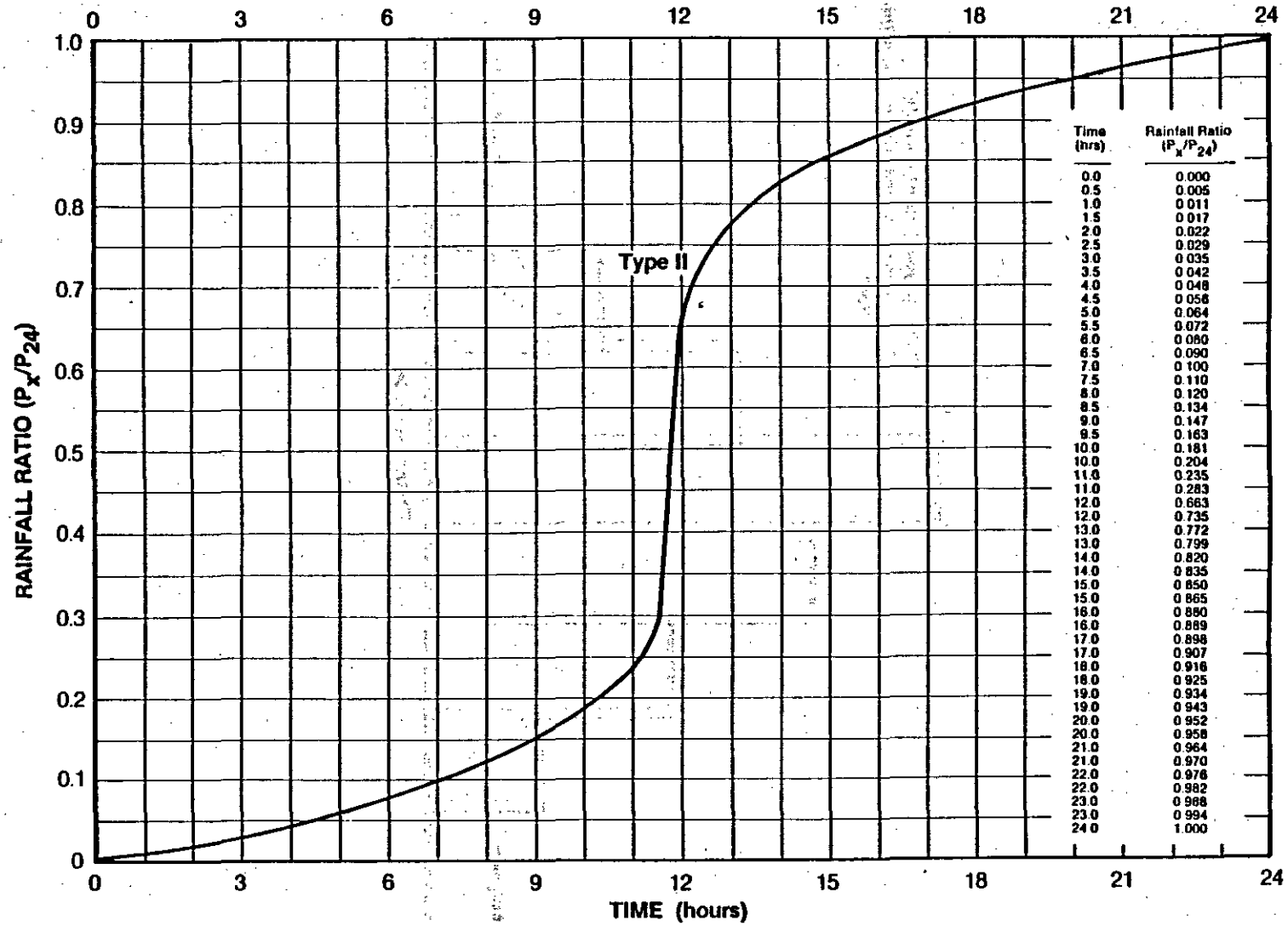


Rainfall Depth (inches)	5 Min	10 Min	15 Min	30 Min	60 Min	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
2 Year	0.48	0.76	0.96	1.33	1.71	2.10	2.29	2.74	3.27	3.78
5 Year	0.55	0.89	1.13	1.62	2.13	2.50	2.83	3.41	4.08	4.78
10 Year	0.61	0.99	1.26	1.83	2.42	2.90	3.20	3.94	4.65	5.46
25 Year	0.69	1.14	1.45	2.13	2.84	3.32	3.71	4.53	5.32	6.21
50 Year	0.76	1.26	1.60	2.37	3.17	3.67	4.05	4.95	5.91	6.82
100 Year	0.83	1.37	1.75	2.61	3.50	4.13	4.46	5.43	6.50	7.47

Rainfall Intensity (inches/hour)	5 Min	10 Min	15 Min	30 Min	60 Min	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
2 Year	5.72	4.56	3.84	2.66	1.71	1.05	0.76	0.46	0.27	0.16
5 Year	6.62	5.35	4.53	3.24	2.13	1.25	0.94	0.57	0.34	0.20
10 Year	7.30	5.95	5.05	3.66	2.42	1.45	1.07	0.66	0.39	0.23
25 Year	8.33	6.83	5.81	4.27	2.84	1.66	1.24	0.76	0.44	0.26
50 Year	9.14	7.52	6.41	4.74	3.17	1.84	1.35	0.83	0.49	0.28
100 Year	9.94	8.21	7.00	5.22	3.50	2.07	1.49	0.91	0.54	0.31

FIGURE 4-1
Intensity-Duration-Frequency Curves and Depth-Duration-Frequency Data for Huntsville



Time (hrs)	Rainfall Ratio (P_x/P_{24})
0.0	0.000
0.5	0.006
1.0	0.011
1.5	0.017
2.0	0.022
2.5	0.029
3.0	0.035
3.5	0.042
4.0	0.048
4.5	0.056
5.0	0.064
5.5	0.072
6.0	0.080
6.5	0.090
7.0	0.100
7.5	0.110
8.0	0.120
8.5	0.134
9.0	0.147
9.5	0.163
10.0	0.181
10.0	0.204
11.0	0.235
11.0	0.283
12.0	0.663
12.0	0.735
13.0	0.772
13.0	0.799
14.0	0.820
14.0	0.835
15.0	0.850
15.0	0.865
16.0	0.880
16.0	0.889
17.0	0.898
17.0	0.907
18.0	0.916
18.0	0.925
19.0	0.934
19.0	0.943
20.0	0.952
20.0	0.958
21.0	0.964
21.0	0.970
22.0	0.976
22.0	0.982
23.0	0.988
23.0	0.994
24.0	1.000

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

FIGURE 4-2
SCS Type II Cumulative Rainfall Distribution

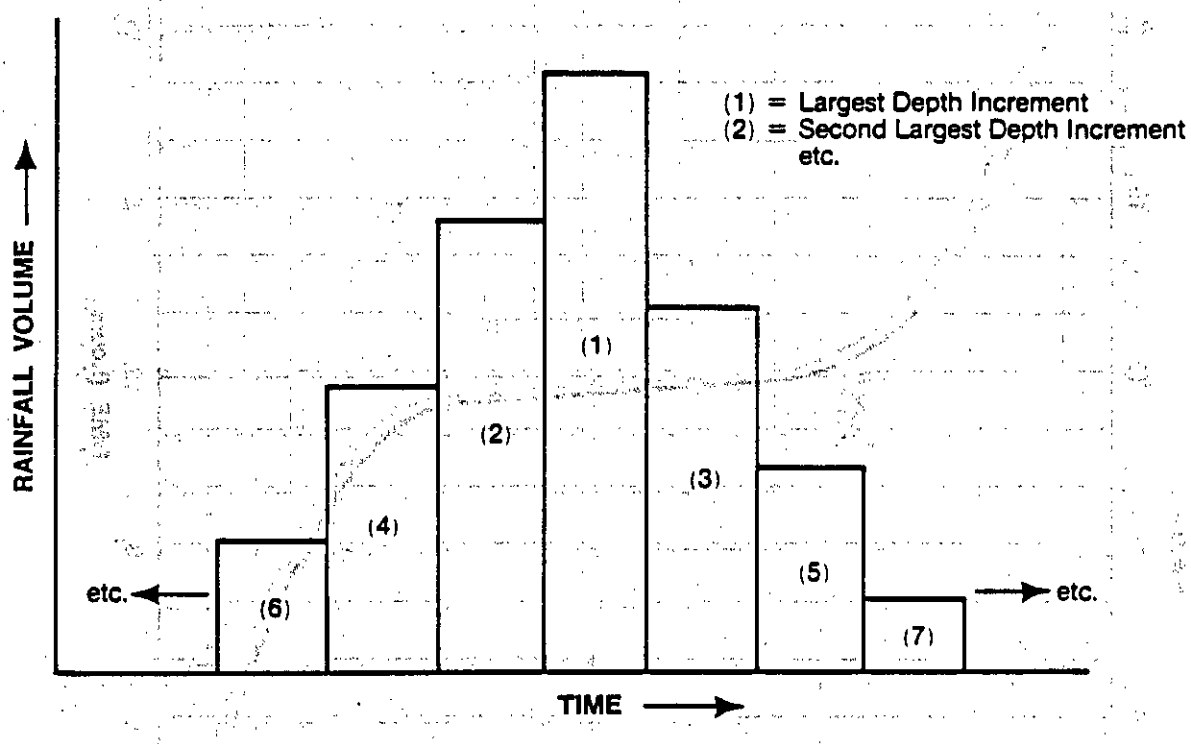
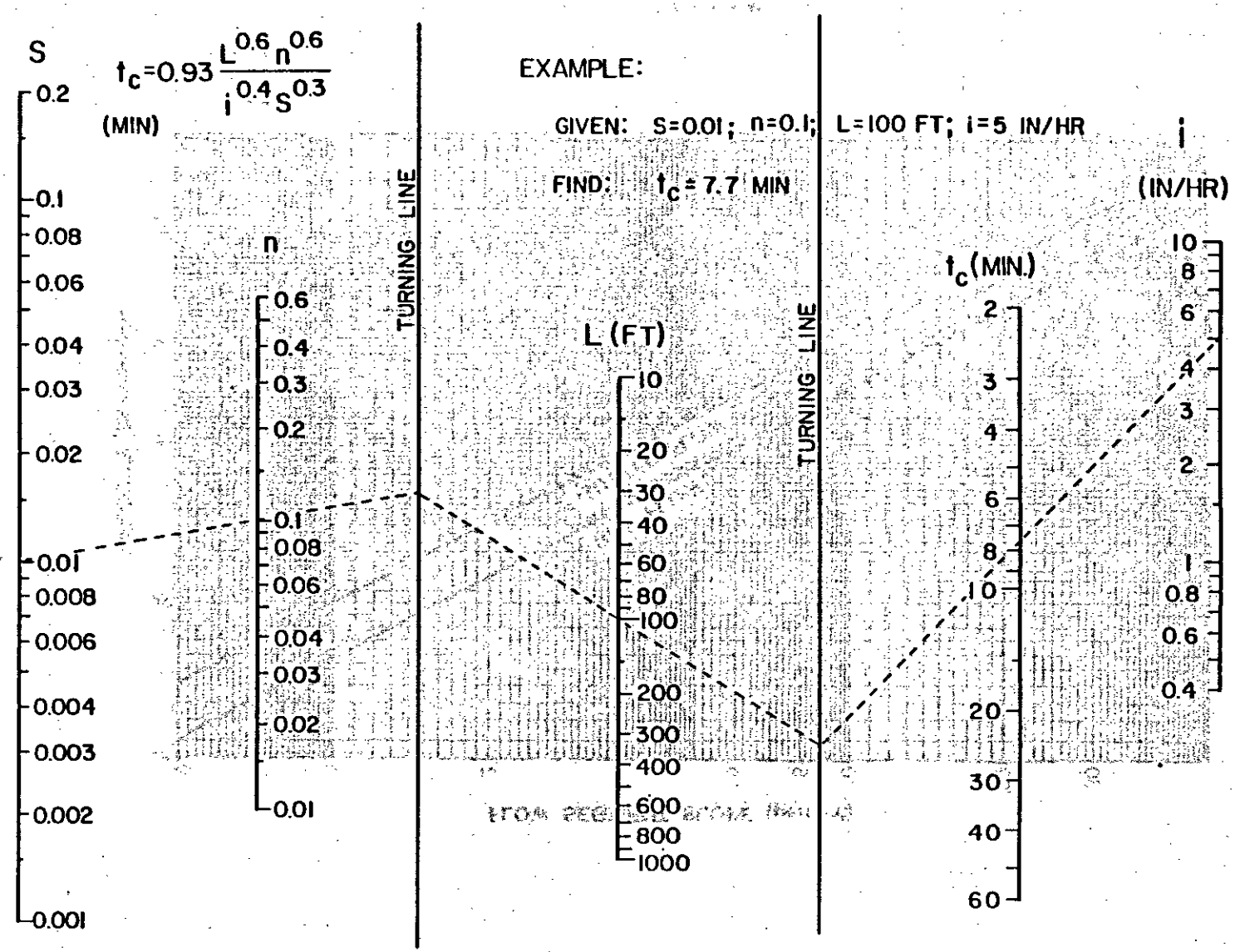
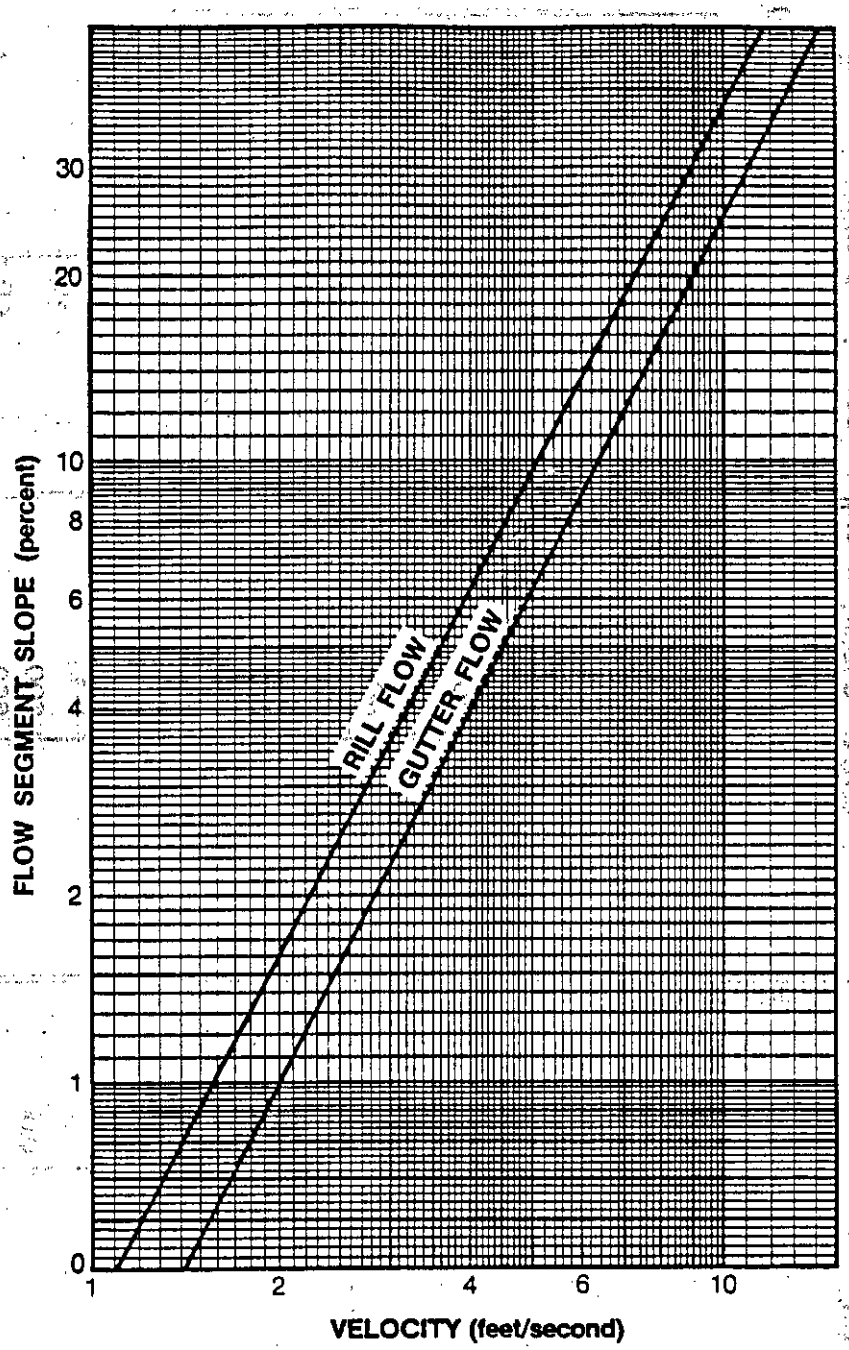


FIGURE 4-3
Balanced Storm Approach for Developing a Design Storm Hyetograph



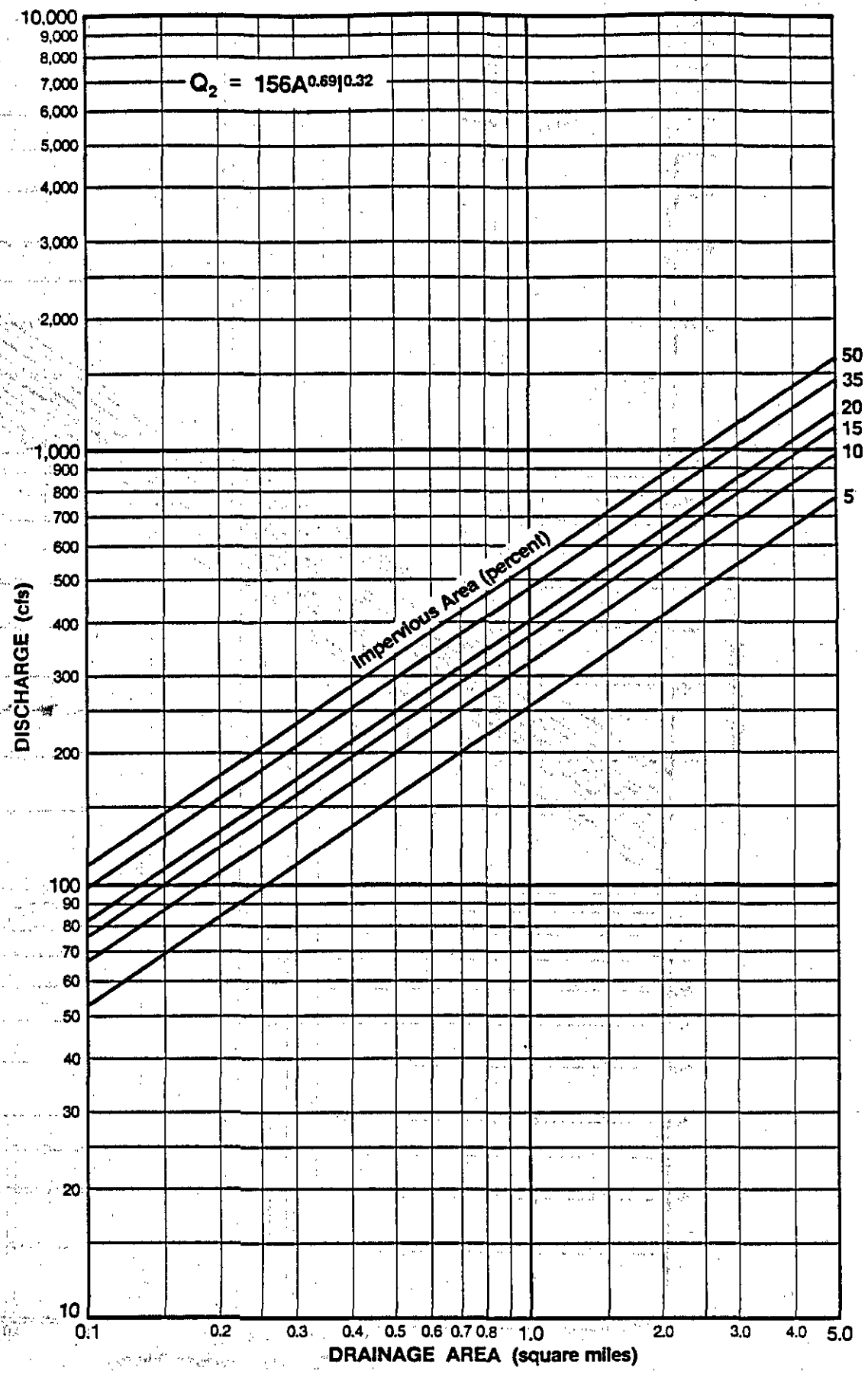
Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 4-4
Kinematic Wave Nomograph for Estimating Overland Flow Travel Time



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

FIGURE 4-5
Average Velocities for Estimating Travel Time for Shallow Channel Flow



Reference: TVA (1986).

FIGURE 4-6
Graphical Solution to the 2-Year Return Frequency Peak Discharge Regression Equation

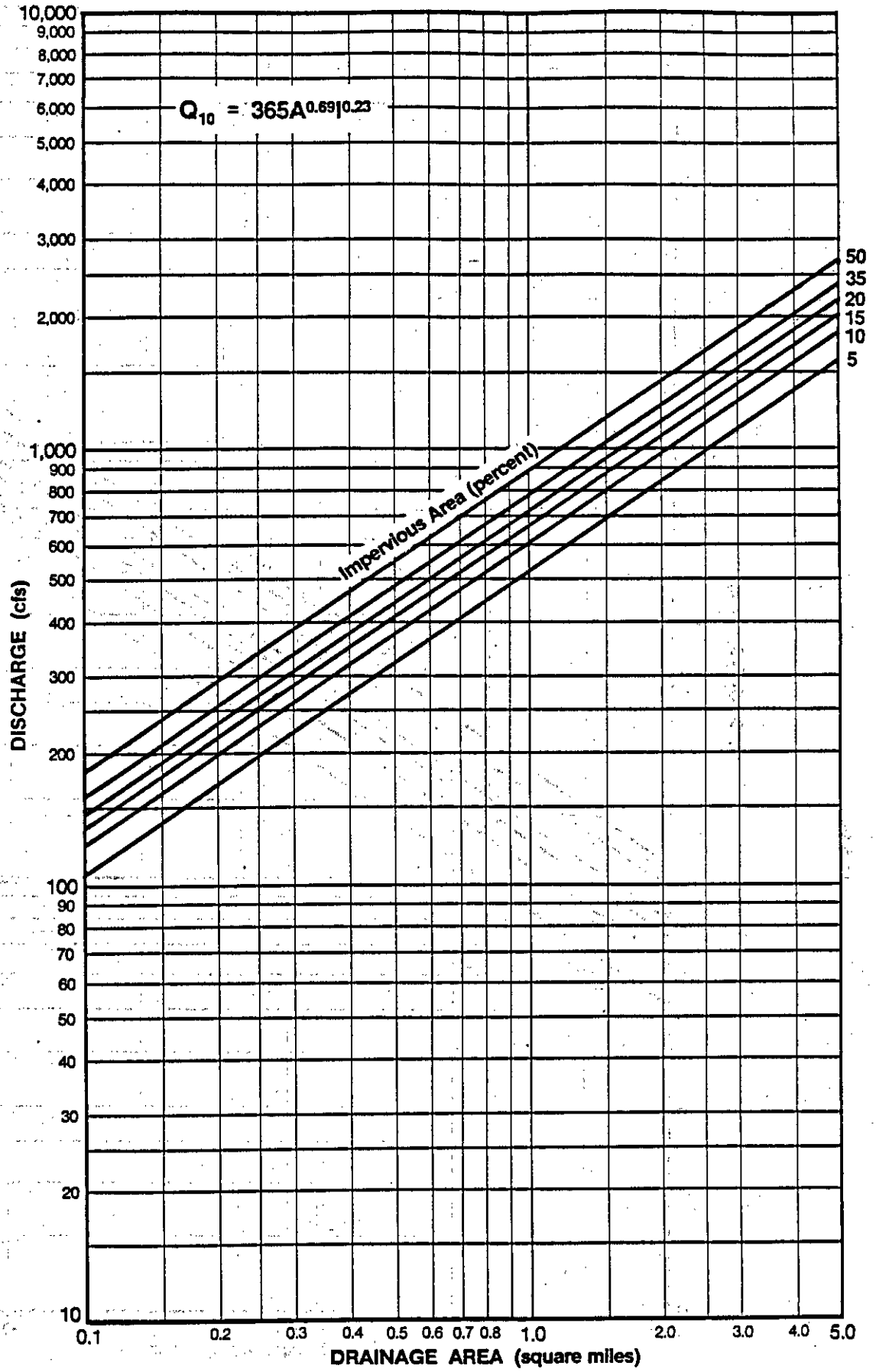
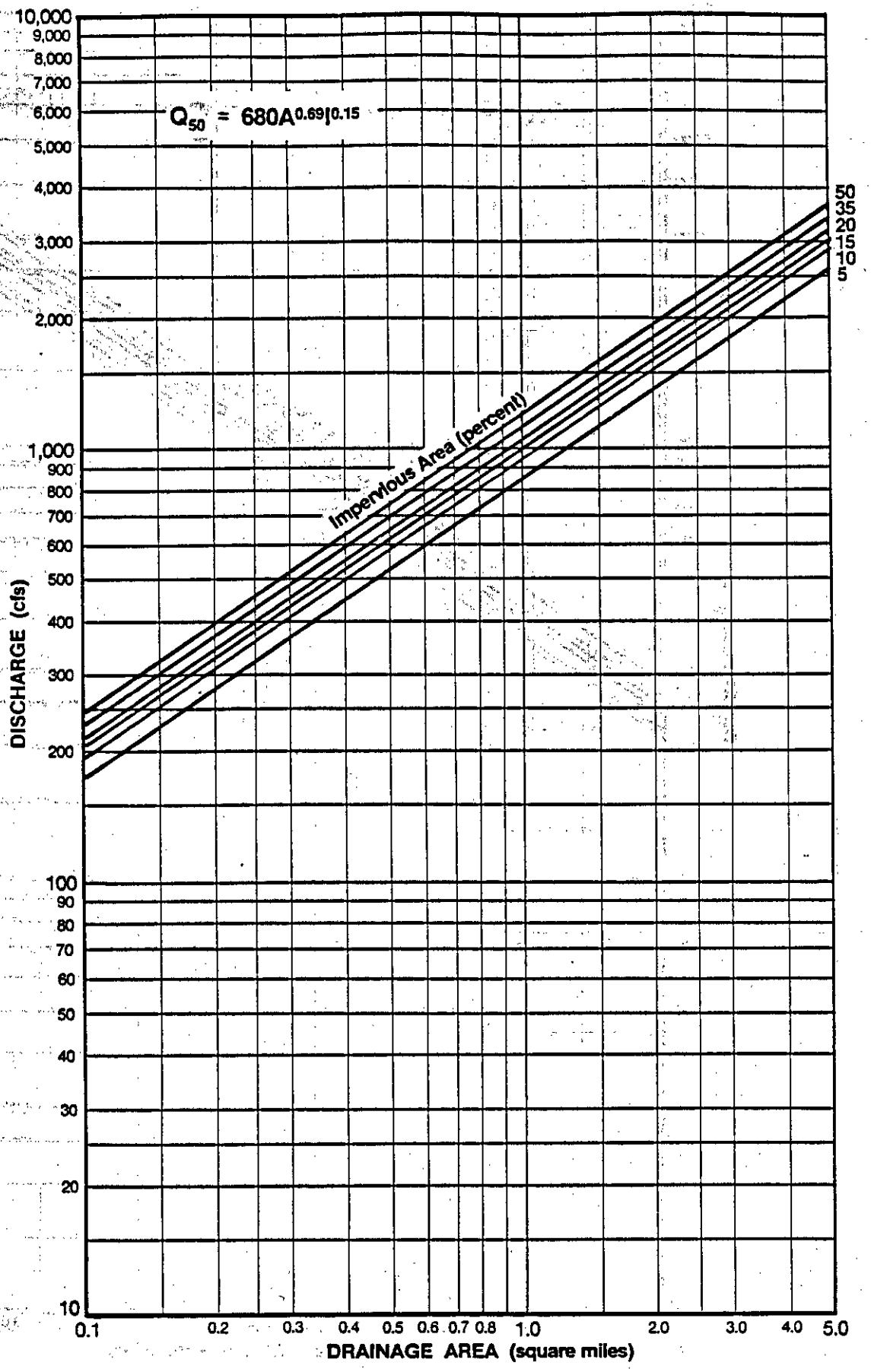


FIGURE 4-7
Graphical Solution to the 10-Year Return Frequency Peak Discharge Regression Equation

Reference: TVA (1986).



Reference: TVA (1986).

FIGURE 4-8
Graphical Solution to the 50-Year Return Frequency Peak Discharge Regression Equation

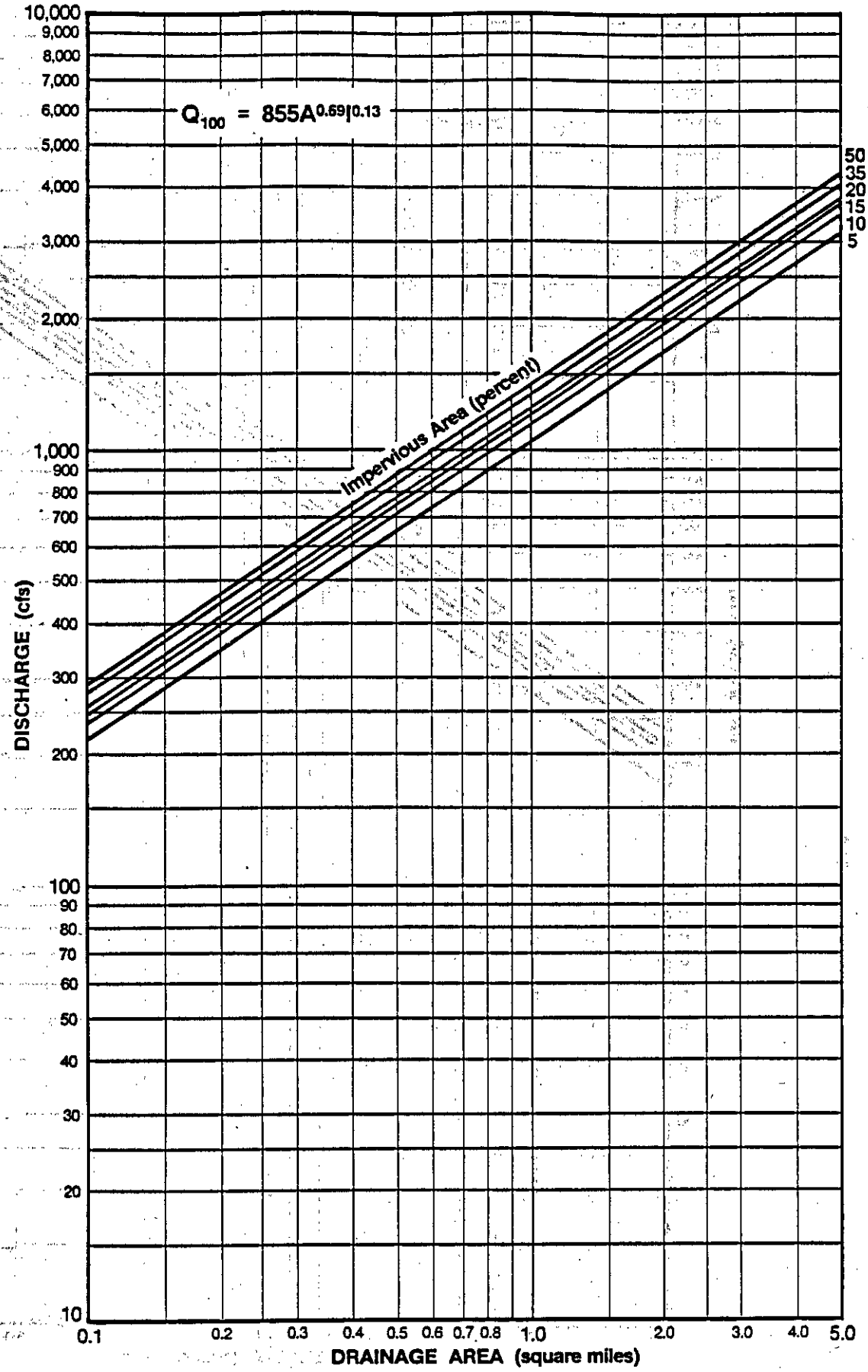


FIGURE 4-9
Graphical Solution to the 100-Year Return Frequency Peak Discharge Regression Equation

Reference: TVA (1986).

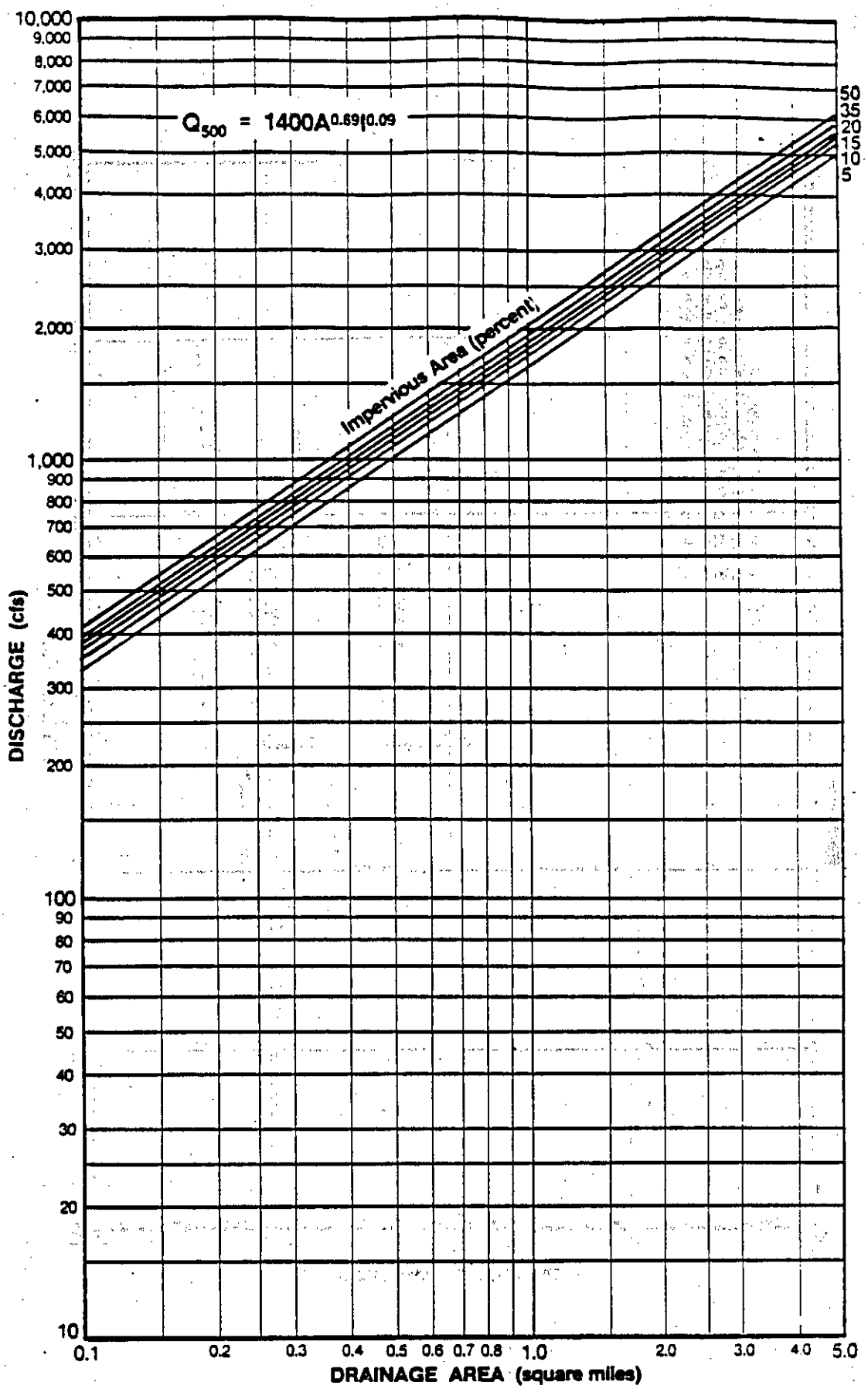
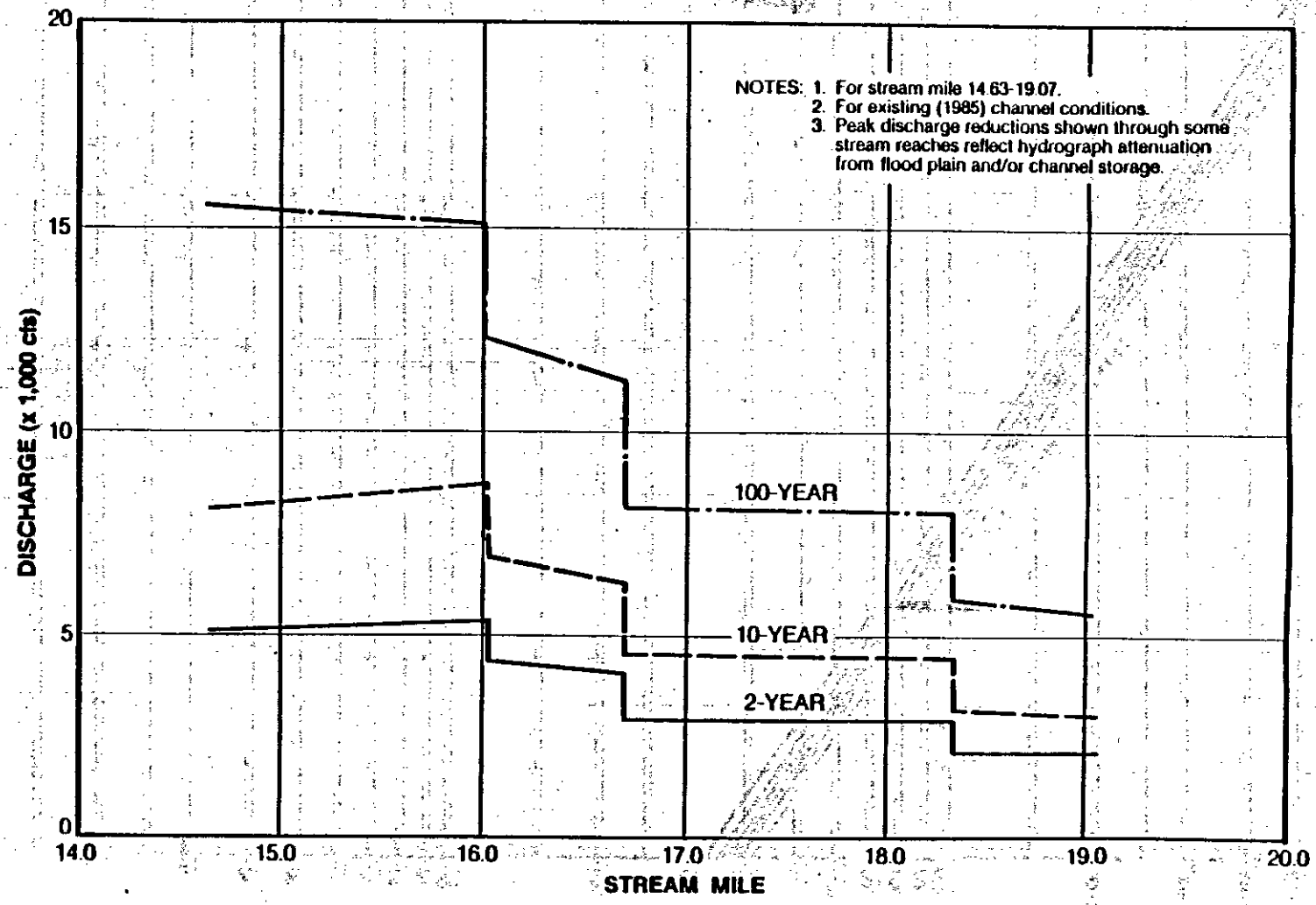


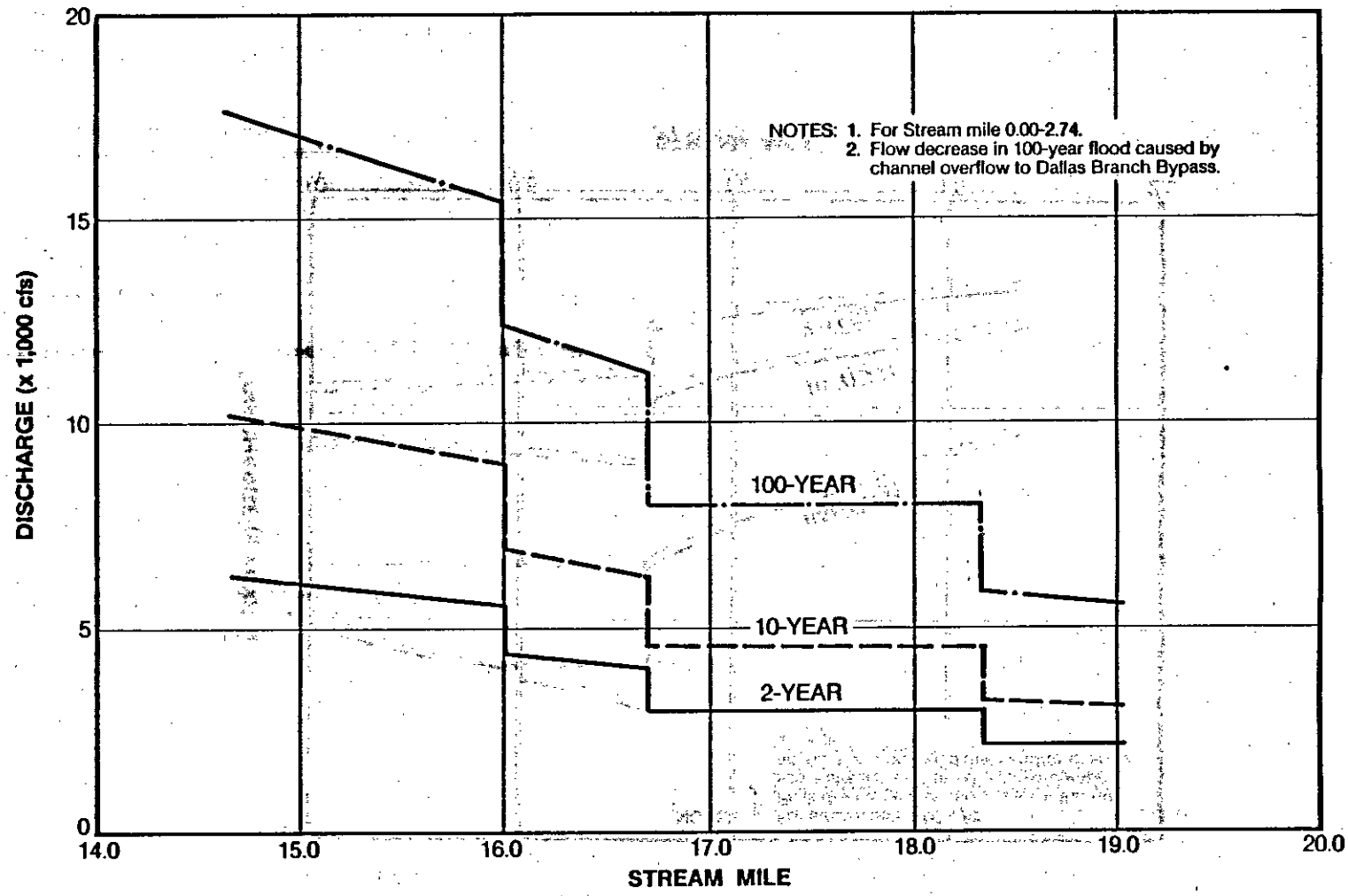
FIGURE 4-10
Graphical Solution to the 500-Year Return Frequency Peak Discharge Regression Equation

Reference: TVA (1986).



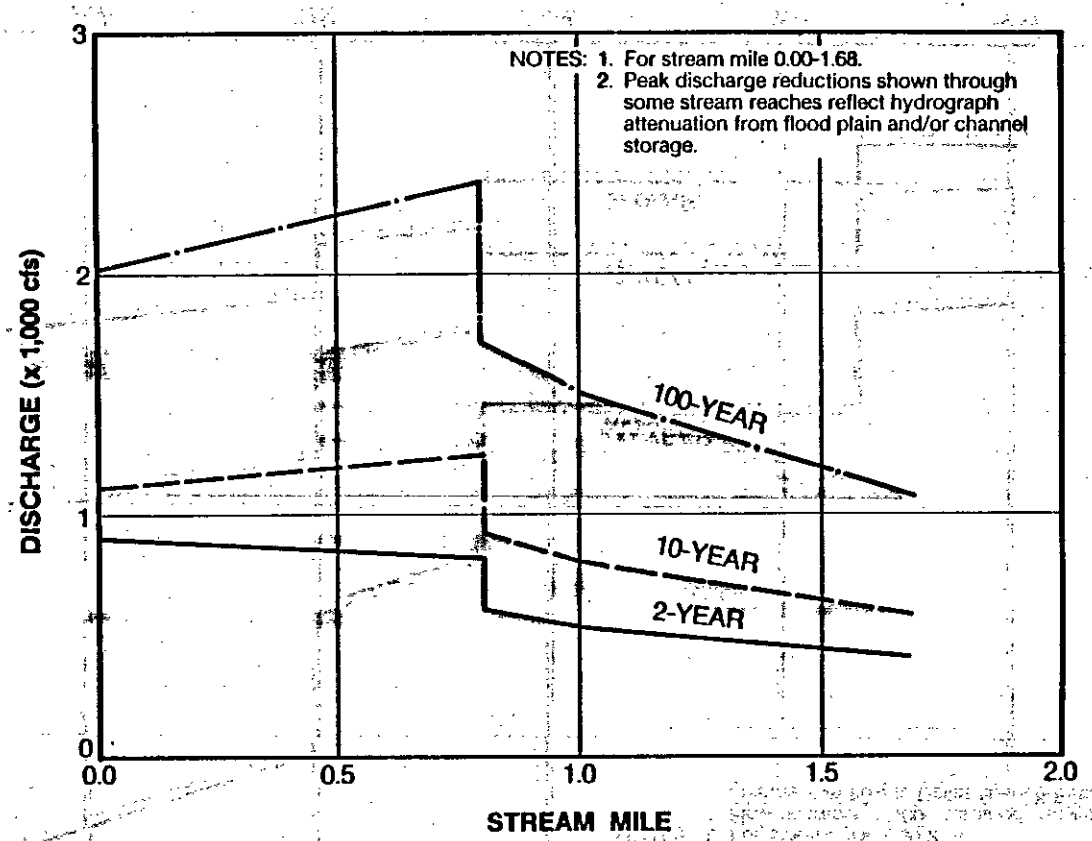
Reference TVA (1986)

FIGURE 4-11
Flow Frequency Estimates for Pinhook Creek, Existing Channel



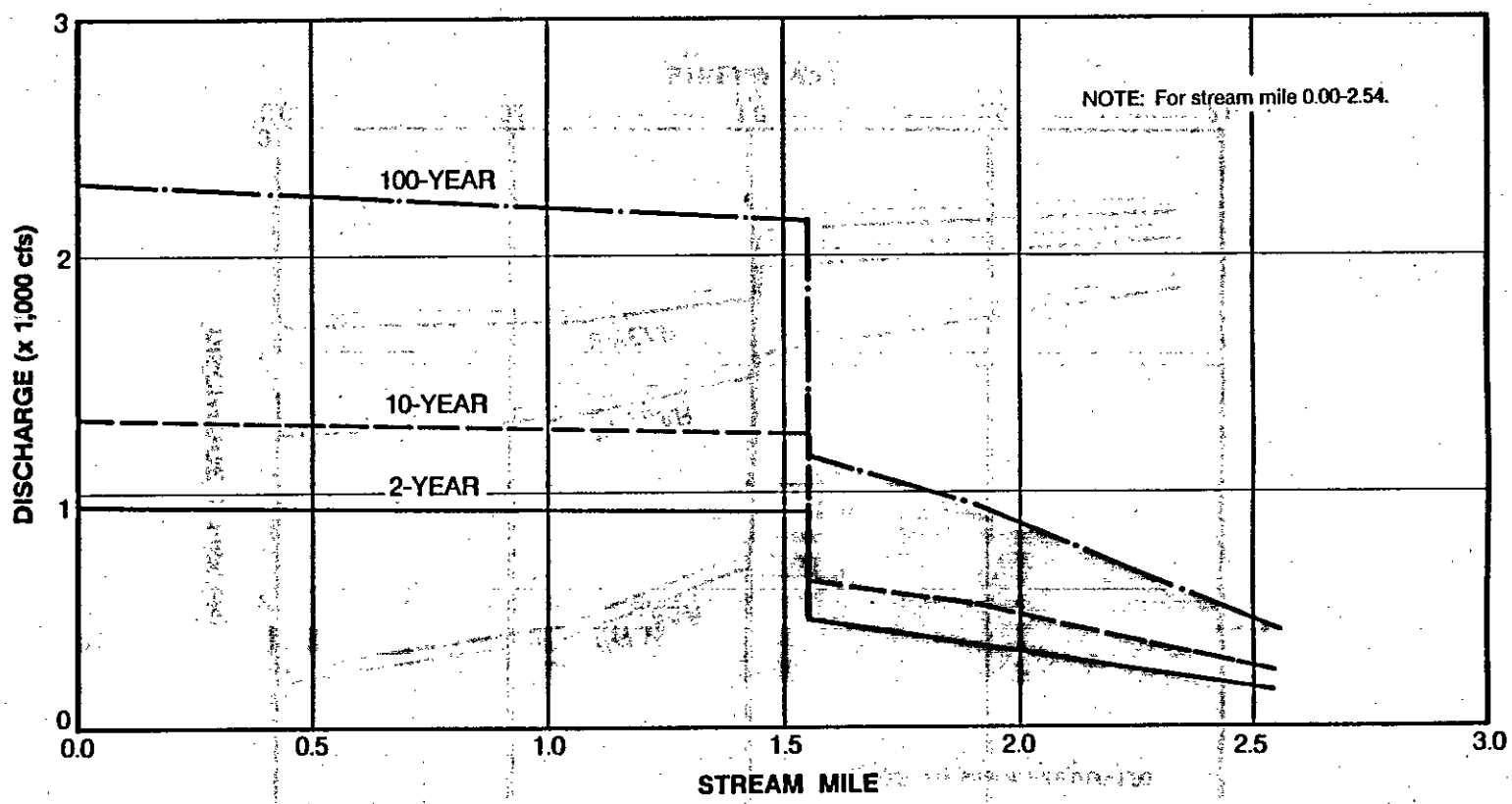
Reference: TVA (1986).

FIGURE 4-12
Flow Frequency Estimates for Pinhook Creek, Proposed Channel



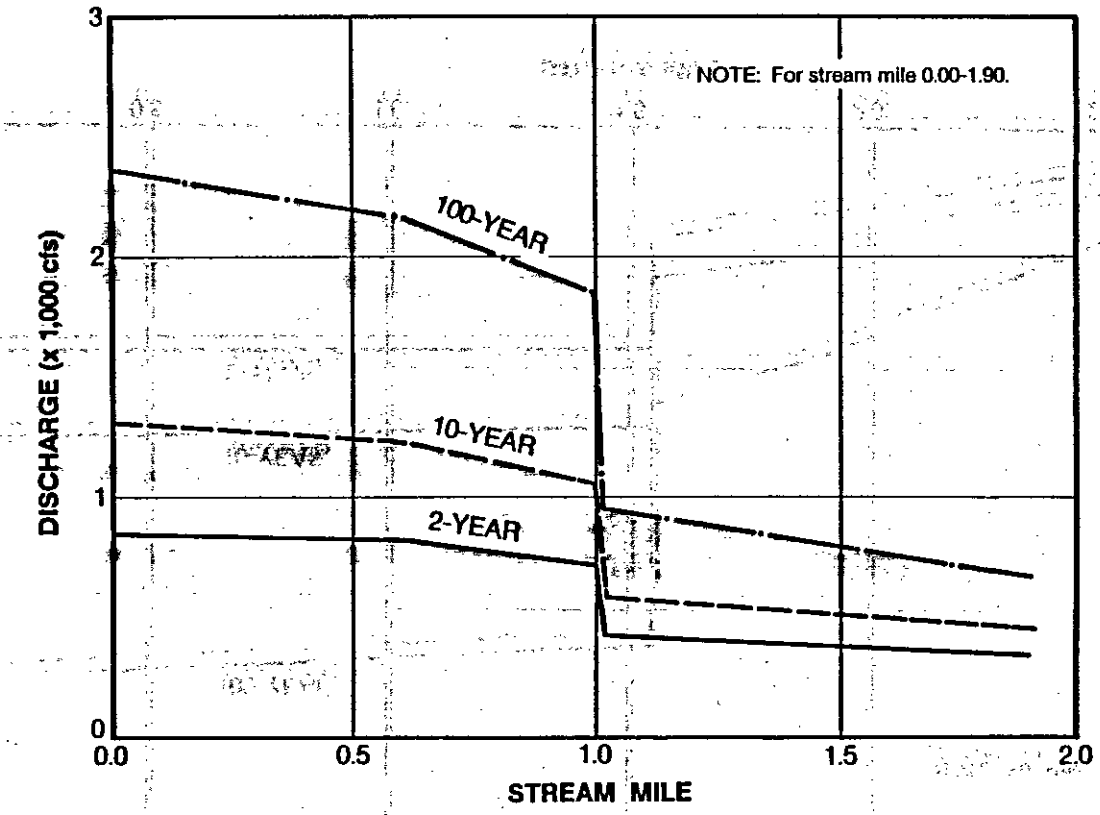
Reference: TVA (1986).

FIGURE 4-13
Flow Frequency Estimates for East Fork Pinhook Creek



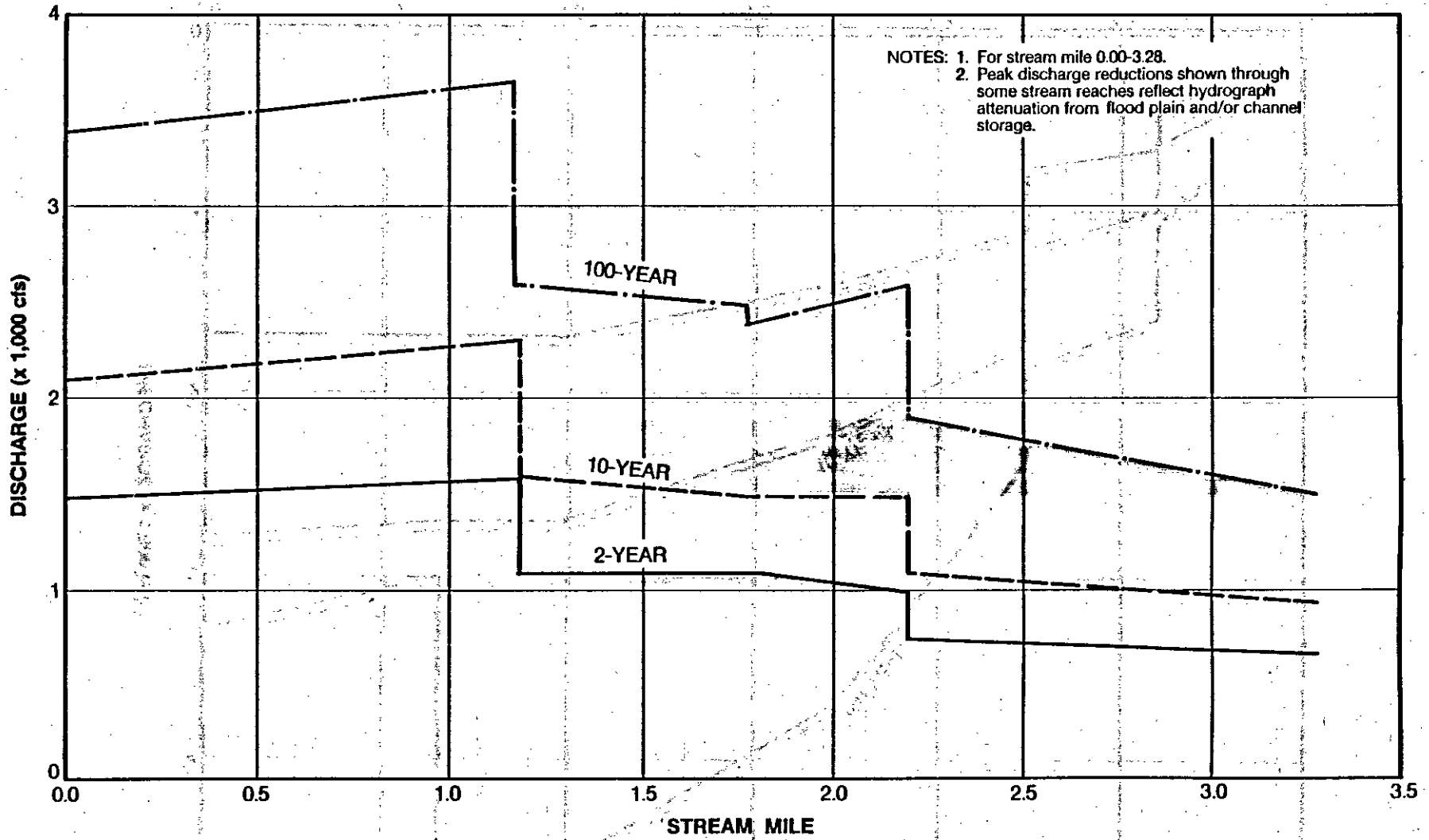
Reference: TVA (1986).

FIGURE 4-14
Flow Frequency Estimates for West Fork Pinhook Creek



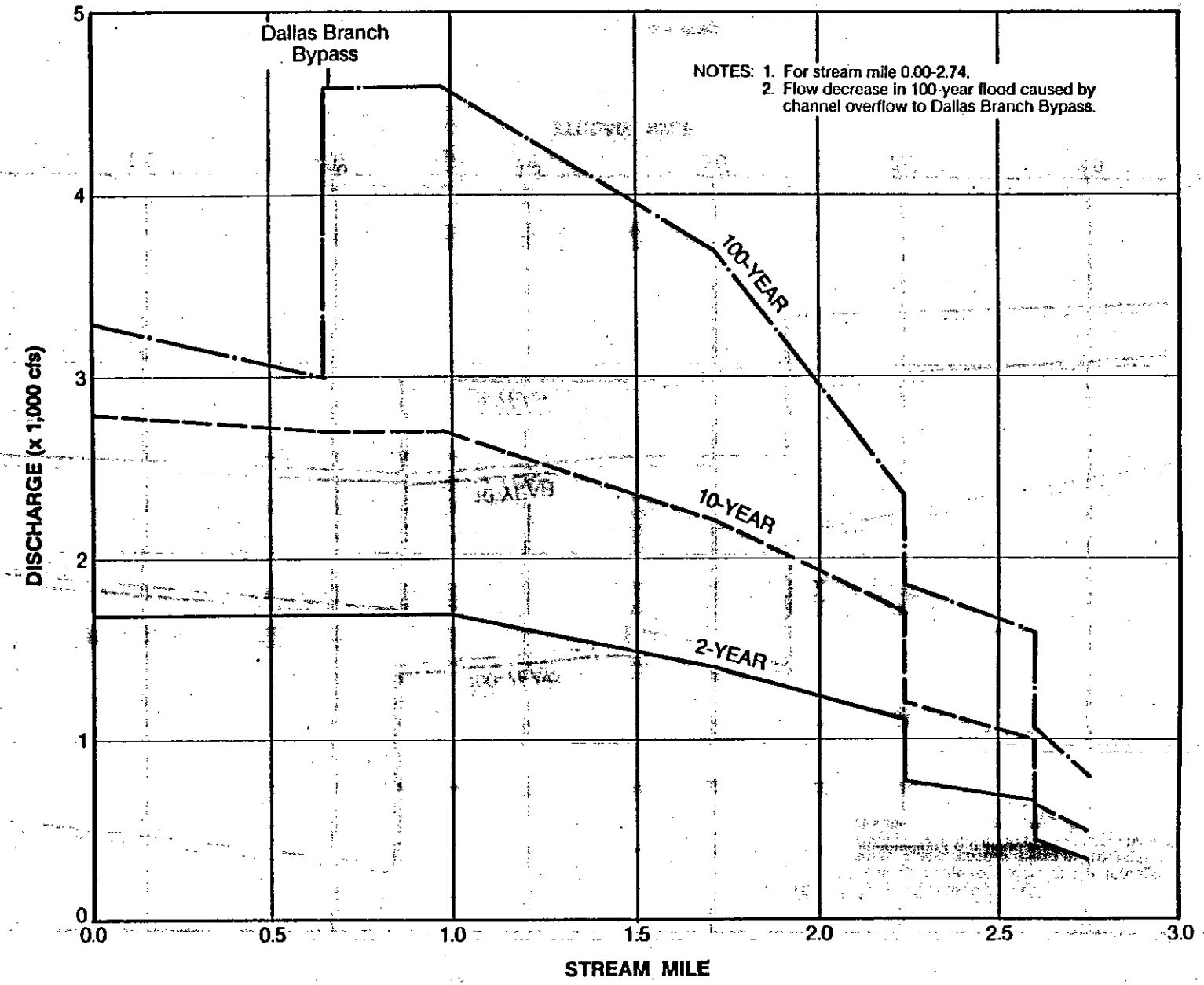
Reference: TVA (1986).

FIGURE 4-15
Flow Frequency Estimates for Blue Spring Creek



Reference: TVA (1986).

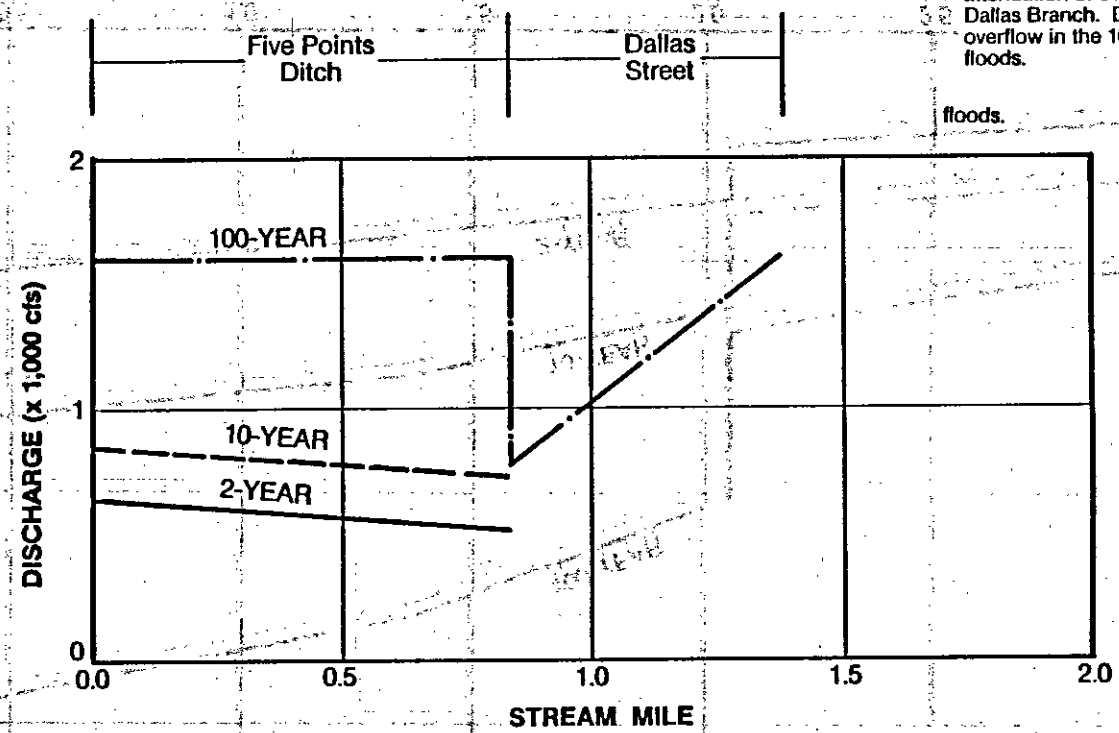
FIGURE 4-16
Flow Frequency Estimates for Normal Branch



Reference: TVA (1986).

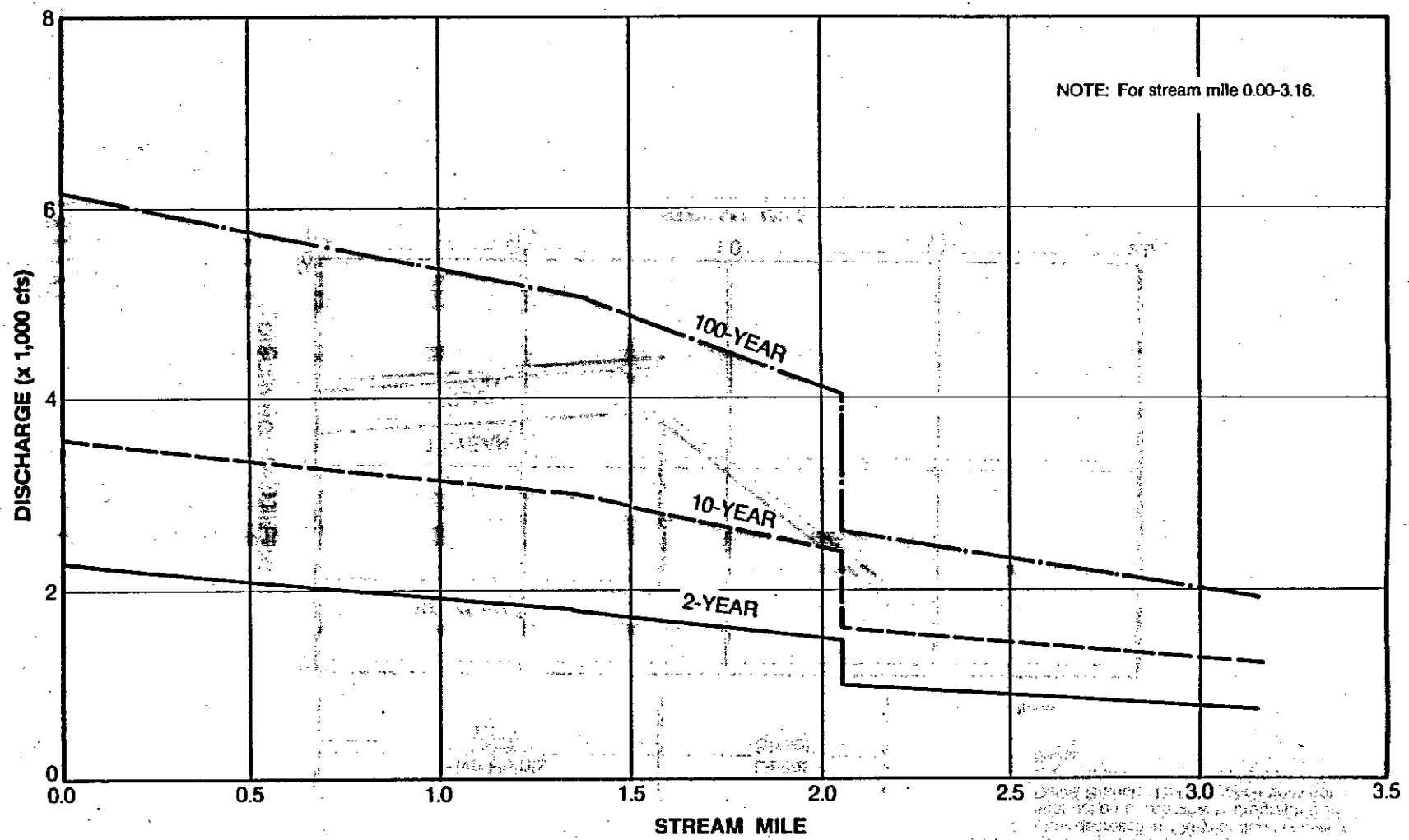
FIGURE 4-17
Flow Frequency Estimates for Dallas Branch

NOTES: 1. For stream mile 0.00-1.37.
2. Flow decrease in 100-year flood caused by attenuation of overflow hydrograph from Dallas Branch. Dallas Branch does not overflow in the 10-year and 2-year floods.



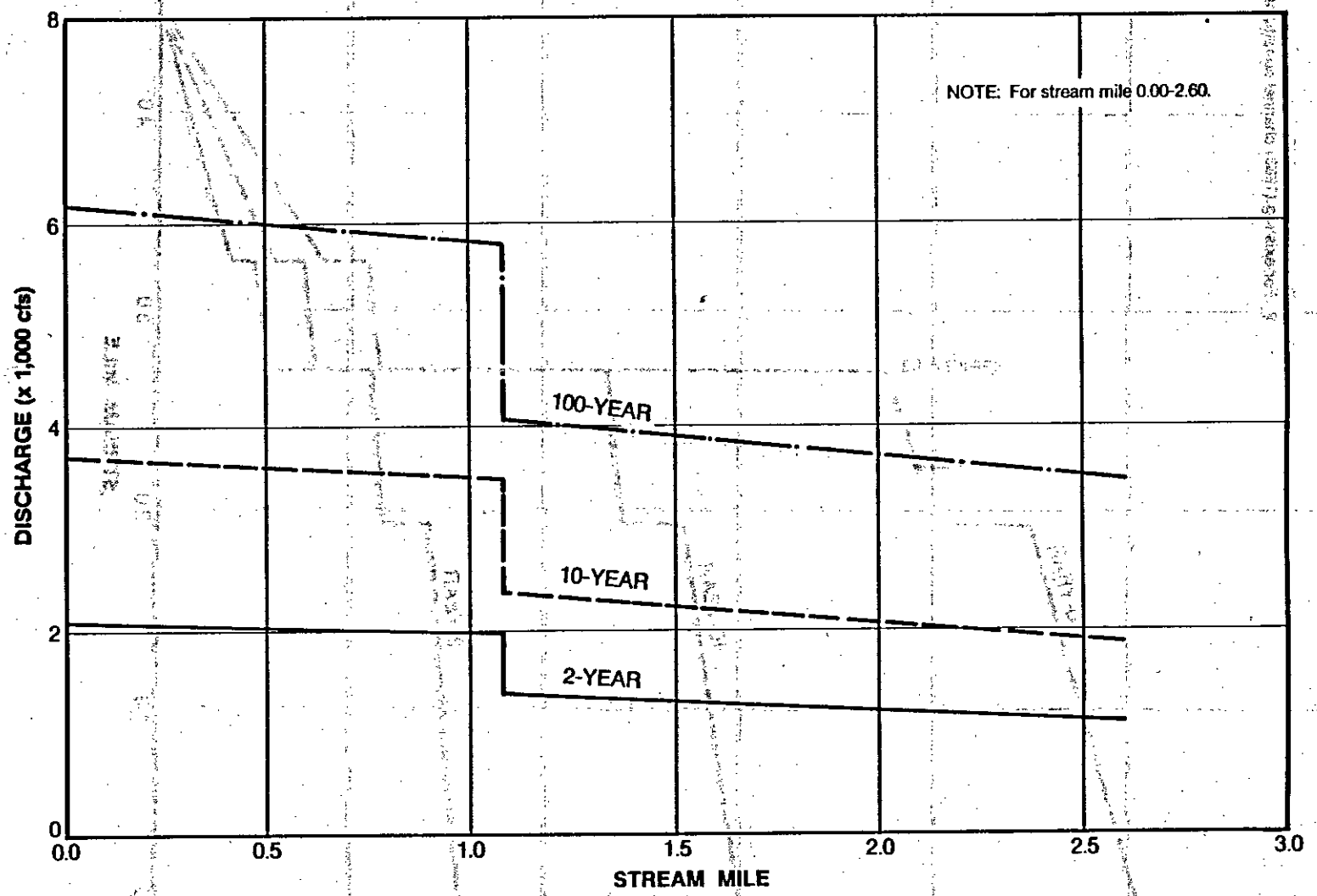
Reference: TVA (1986).

FIGURE 4-18
Flow Frequency Estimates for Dallas Branch Bypass



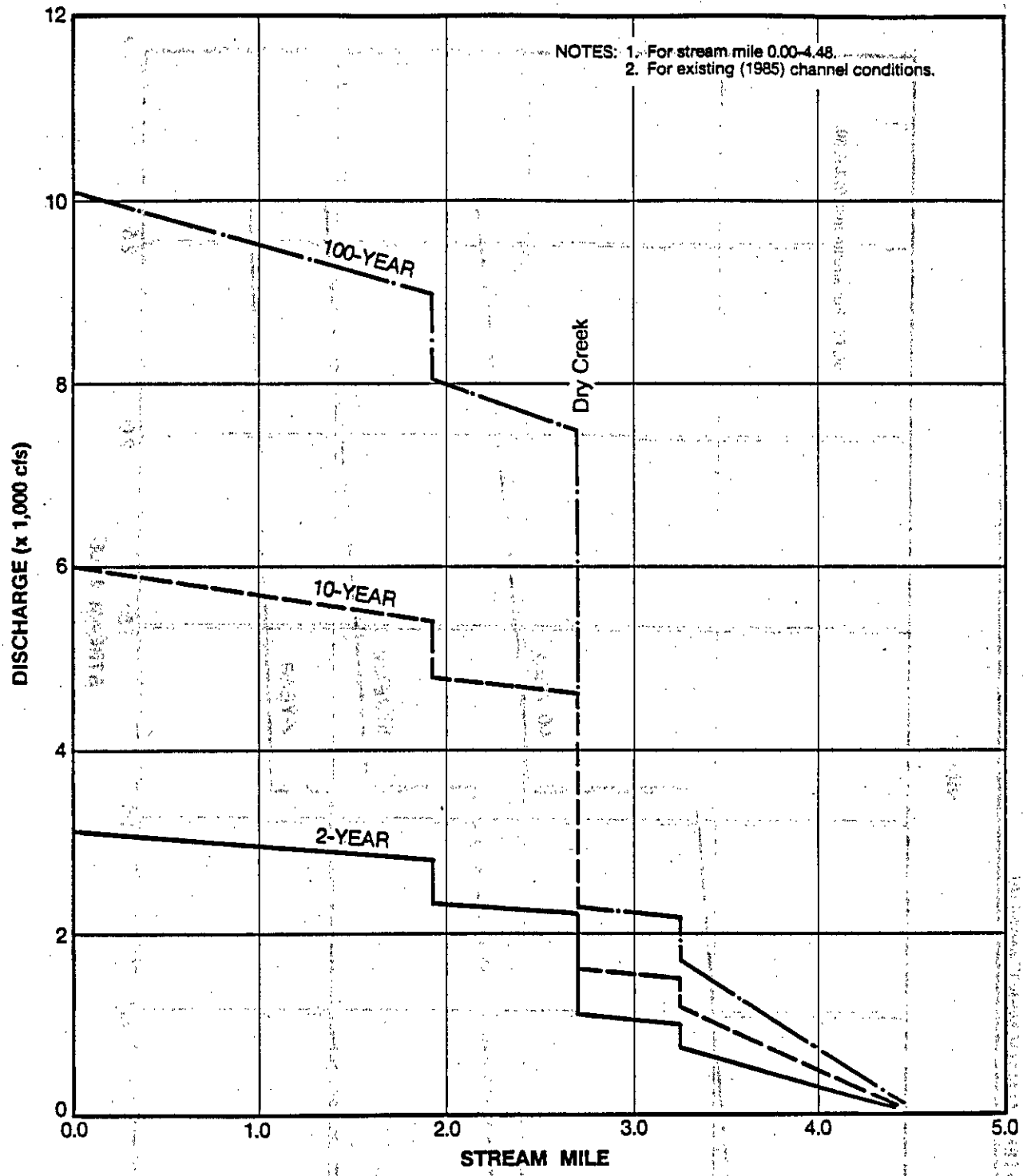
Reference: TVA (1986).

FIGURE 4-19
Flow Frequency Estimates for Fagan Creek



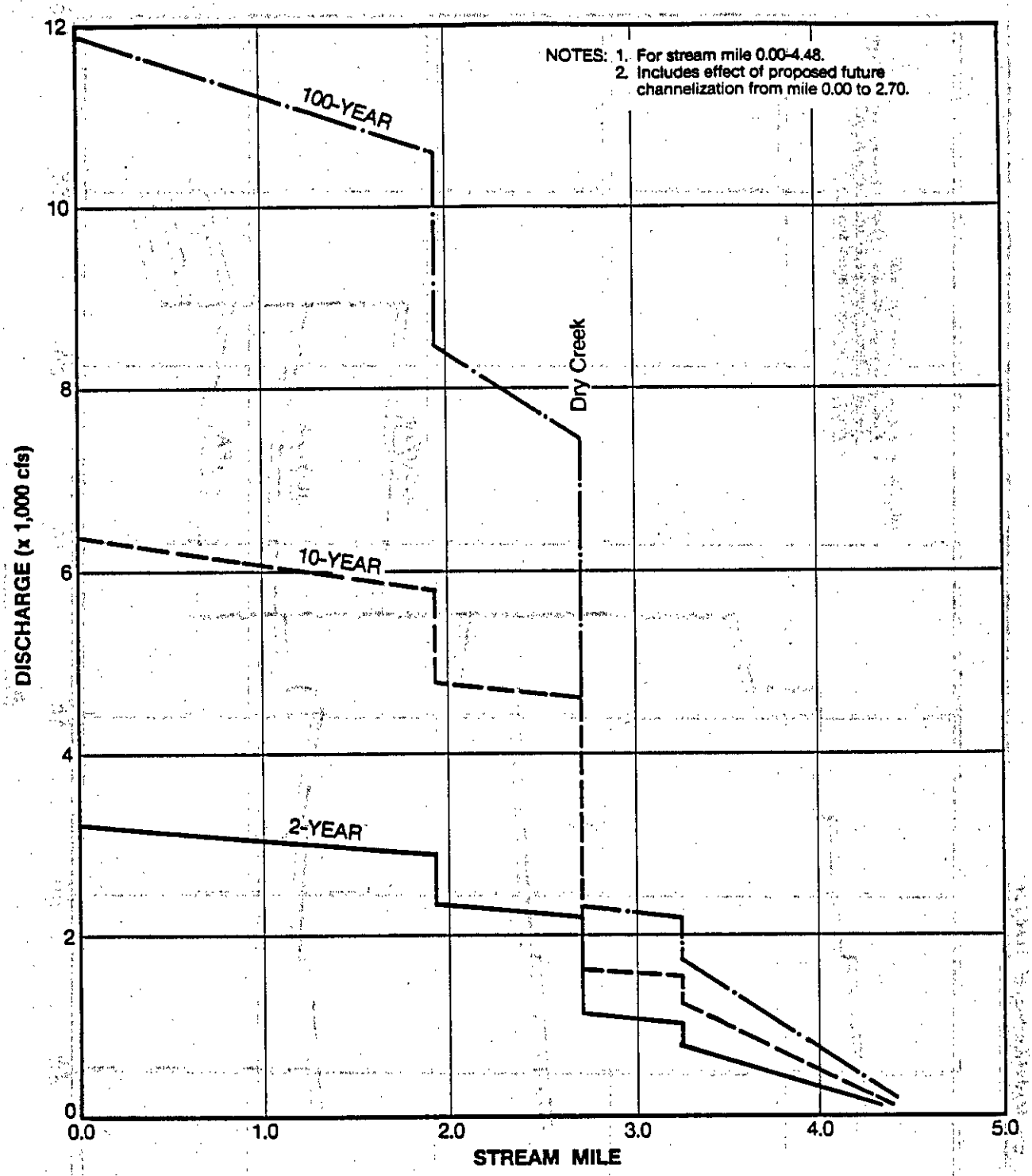
Reference: TVA (1986).

FIGURE 4-20
Flow Frequency Estimates for Dry Creek



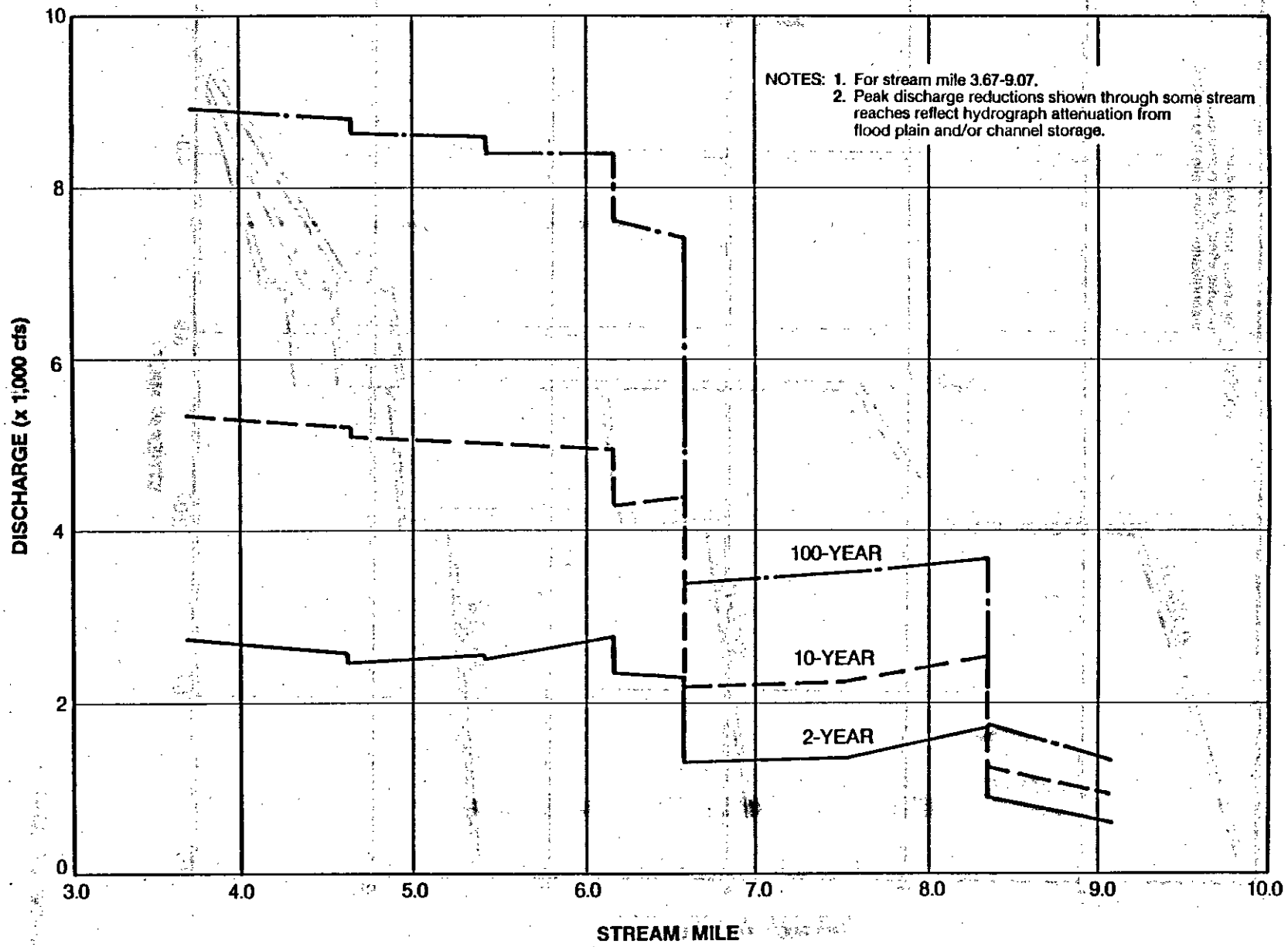
Reference: TVA (1986).

FIGURE 4-21
Flow Frequency Estimates for Broglan Branch, Existing Channel



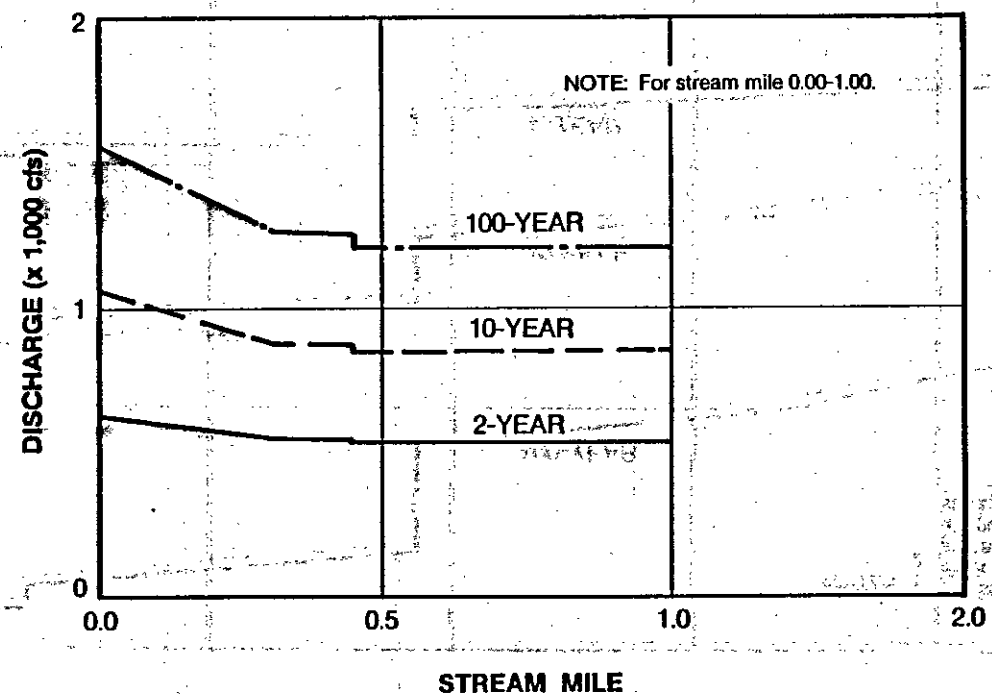
Reference: TVA (1986).

FIGURE 4-22
Flow Frequency Estimates for Broglan Branch, Proposed Channel



Reference: TVA (1986).

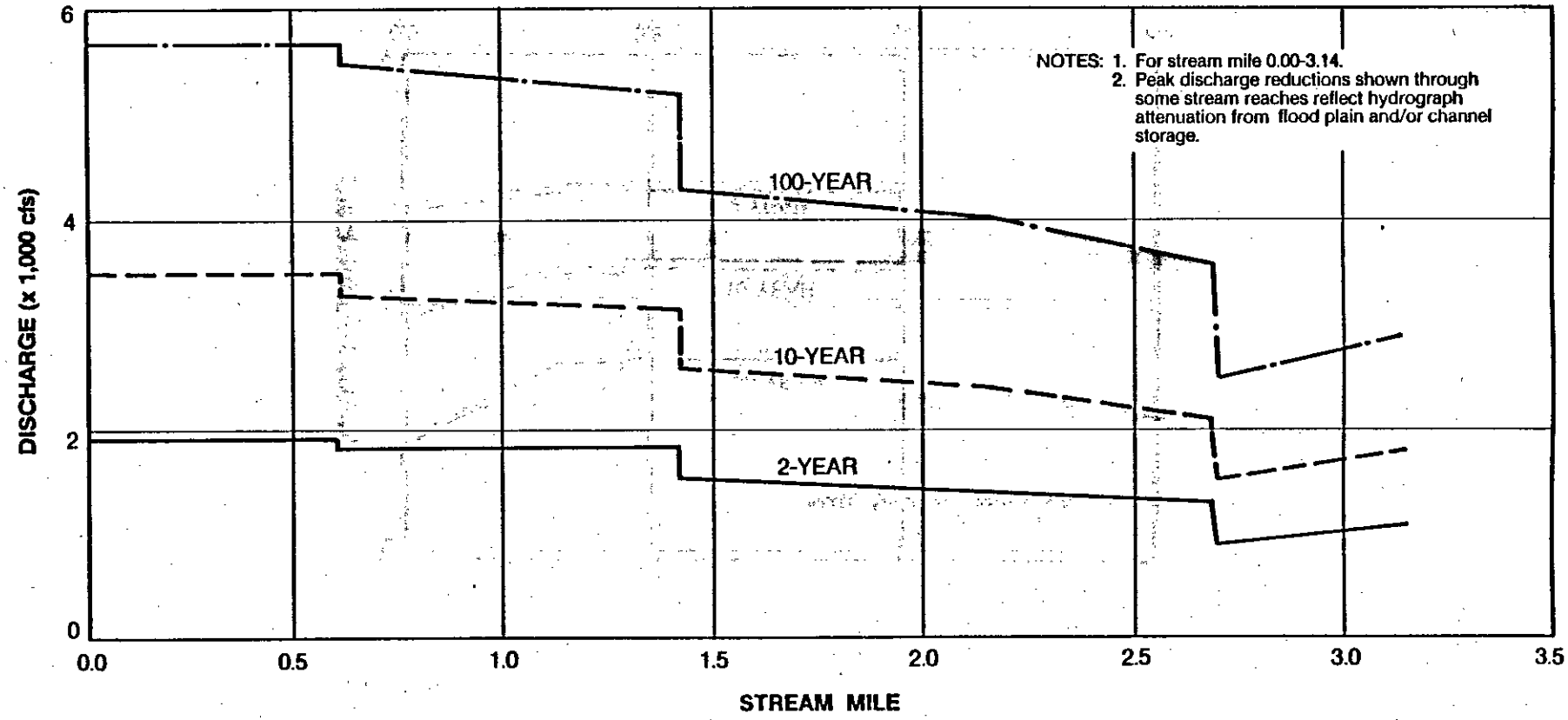
FIGURE 4-23
Flow Frequency Estimates for McDonald Creek



Reference: TVA (1986).

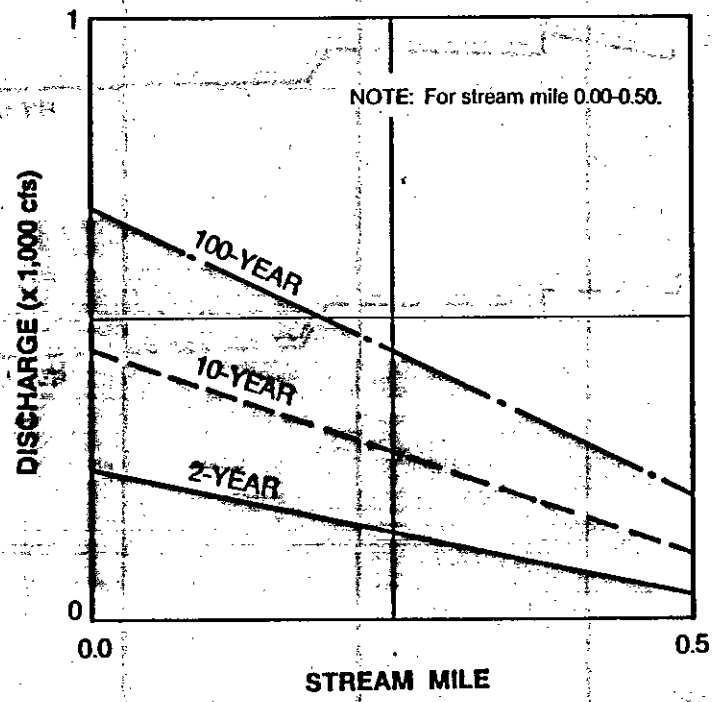
FIGURE 4-24

Flow Frequency Estimates for Tributary to McDonald Creek at Mile 6.14



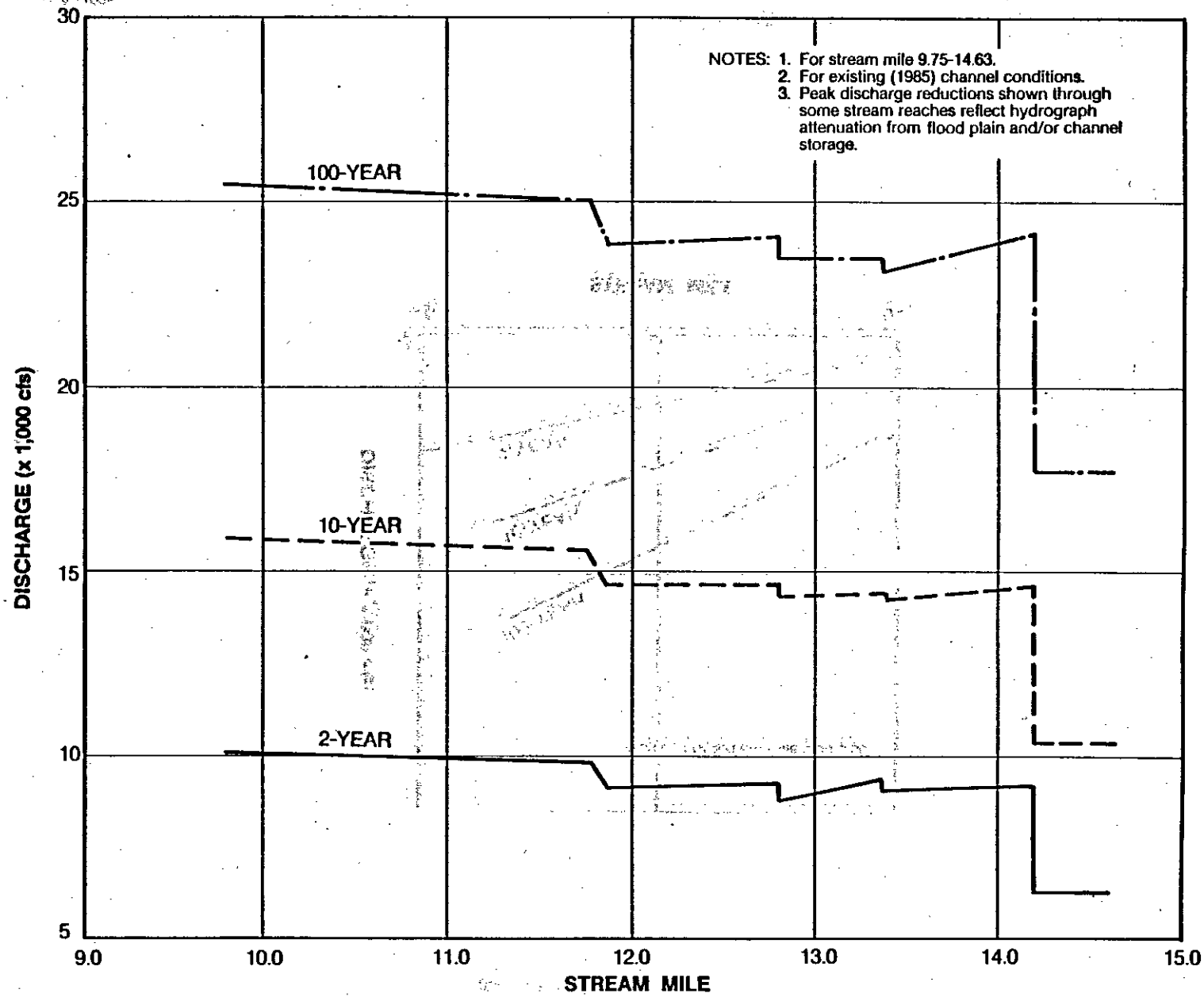
Reference: TVA (1986).

FIGURE 4-25
Flow Frequency Estimates for Sherwood Branch



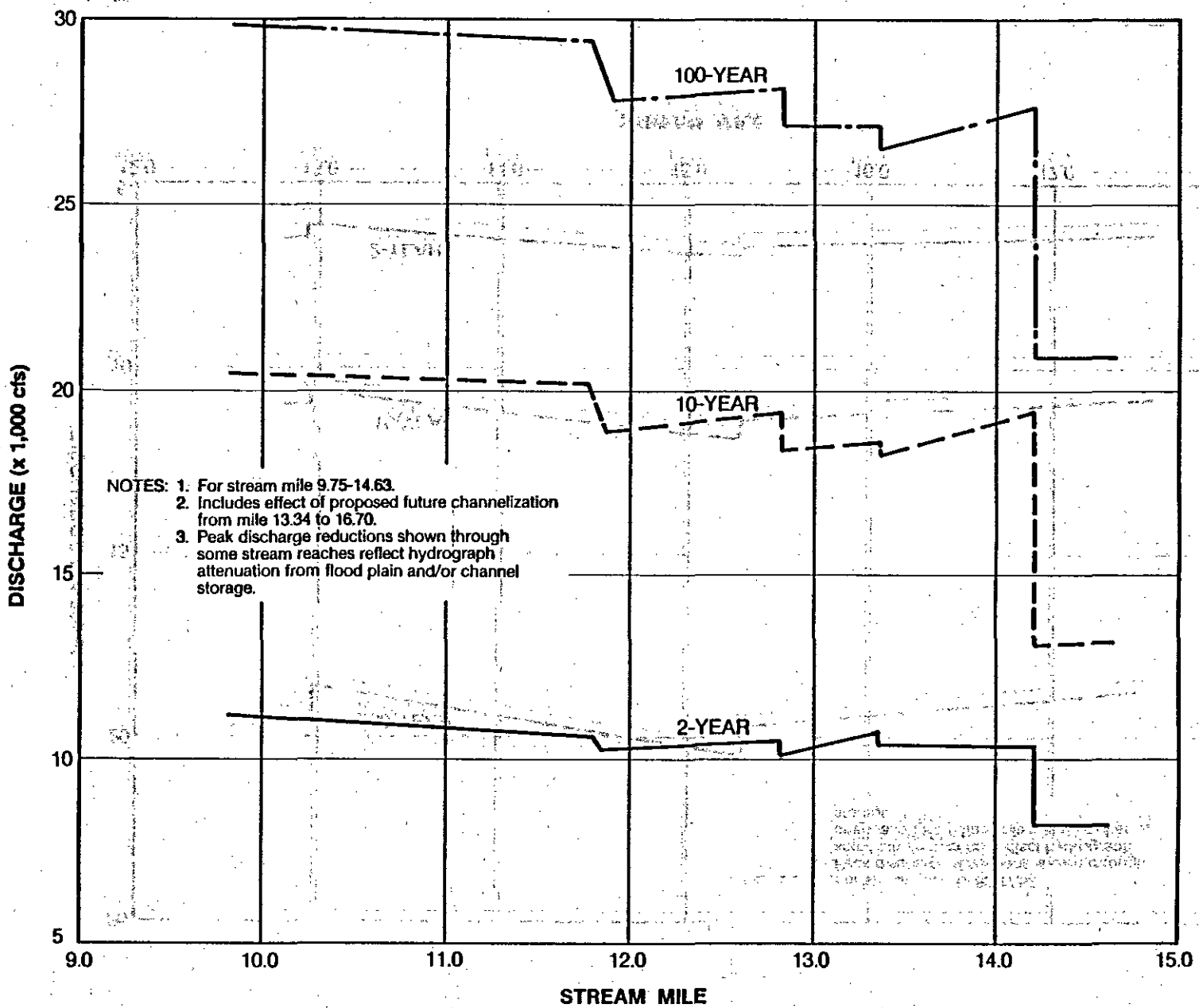
Reference: TVA (1986).

FIGURE 4-26
Flow Frequency Estimates for Tributary to Sherwood Branch at Mile 1.42



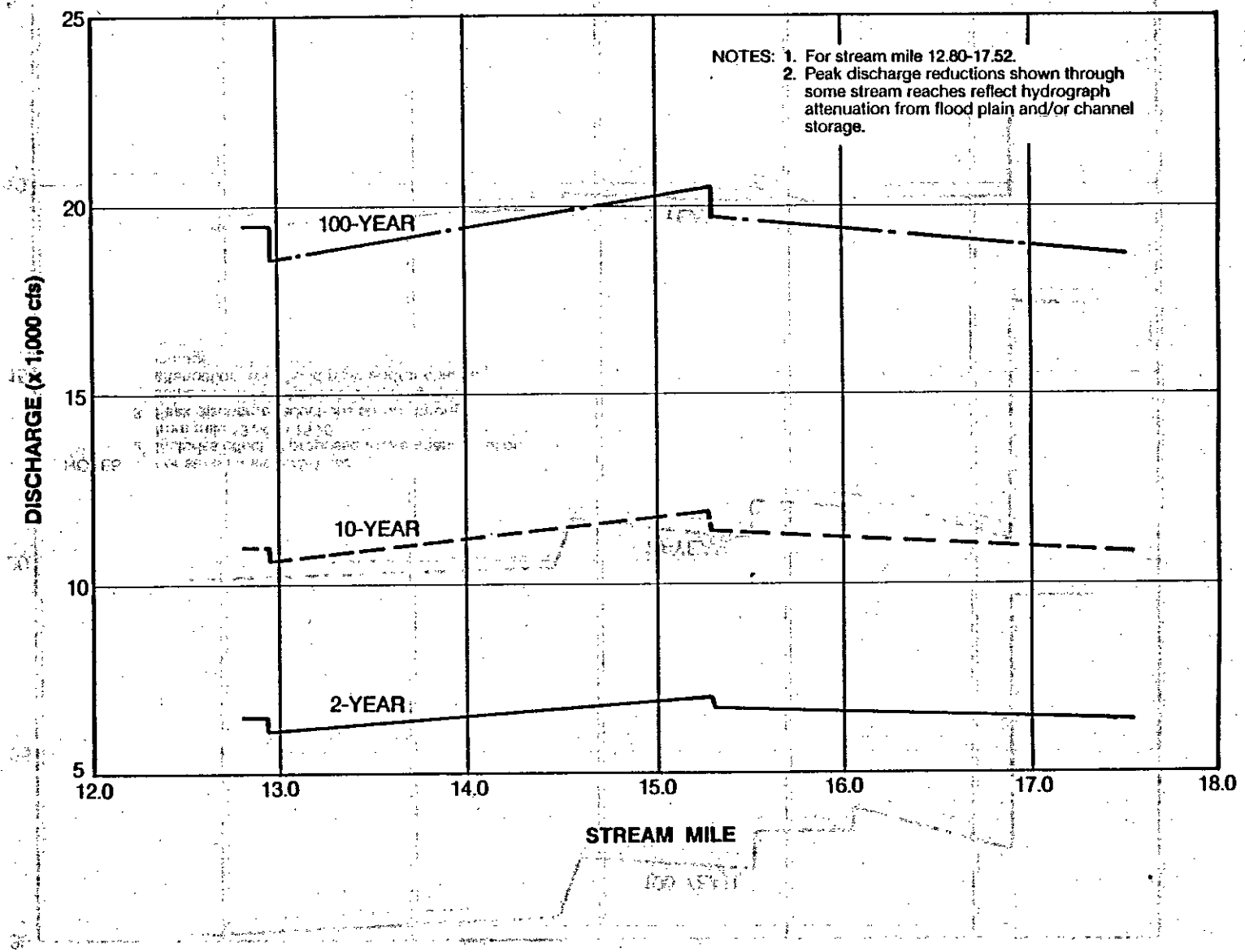
Reference: TVA (1986).

FIGURE 4-27
Flow Frequency Estimates for Huntsville Spring Branch, Existing Channel



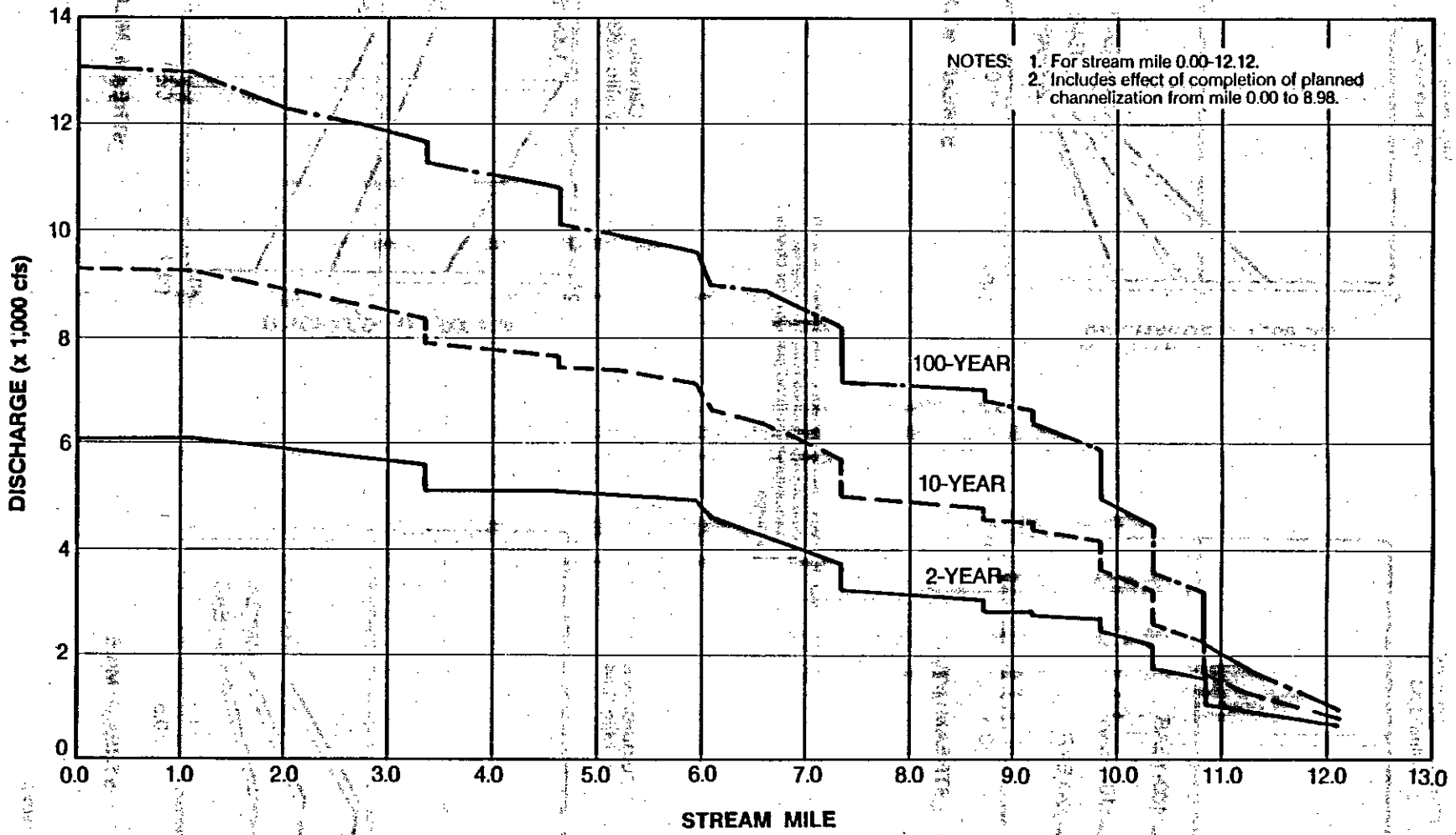
Reference: TVA (1986).

FIGURE 4-28
Flow Frequency Estimates for Huntsville Spring Branch, Proposed Channel



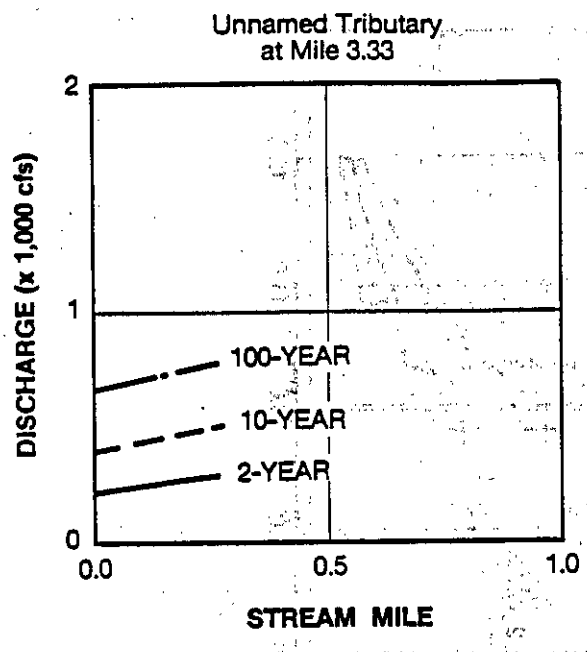
Reference: TVA (1986).

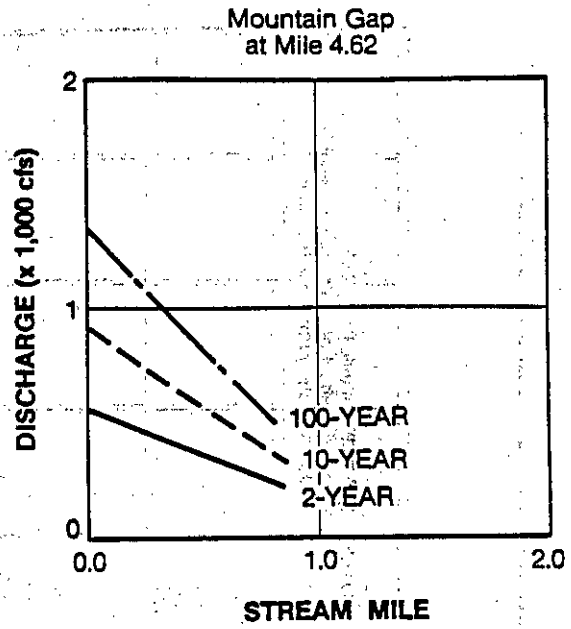
FIGURE 4-29
Flow Frequency Estimates for Indian Creek



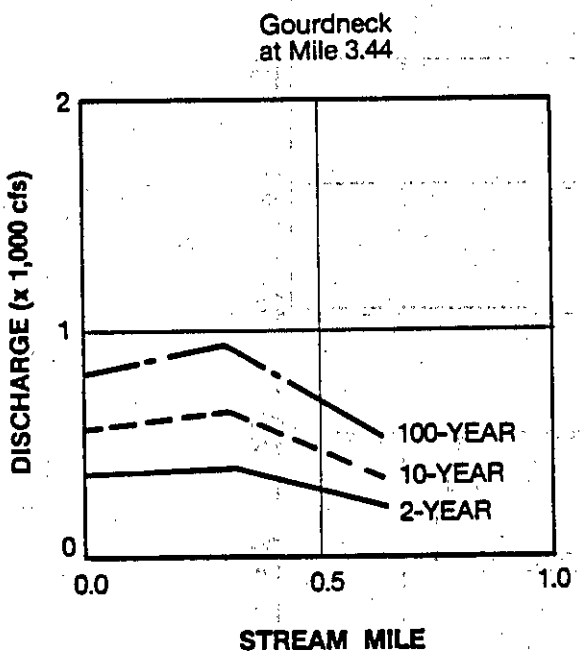
Reference: TVA (1986).

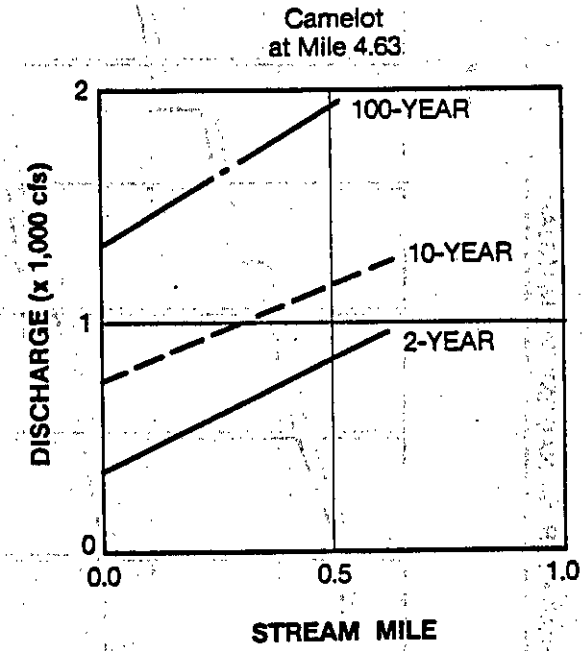
FIGURE 4-30
Flow Frequency Estimates for Aldridge Creek





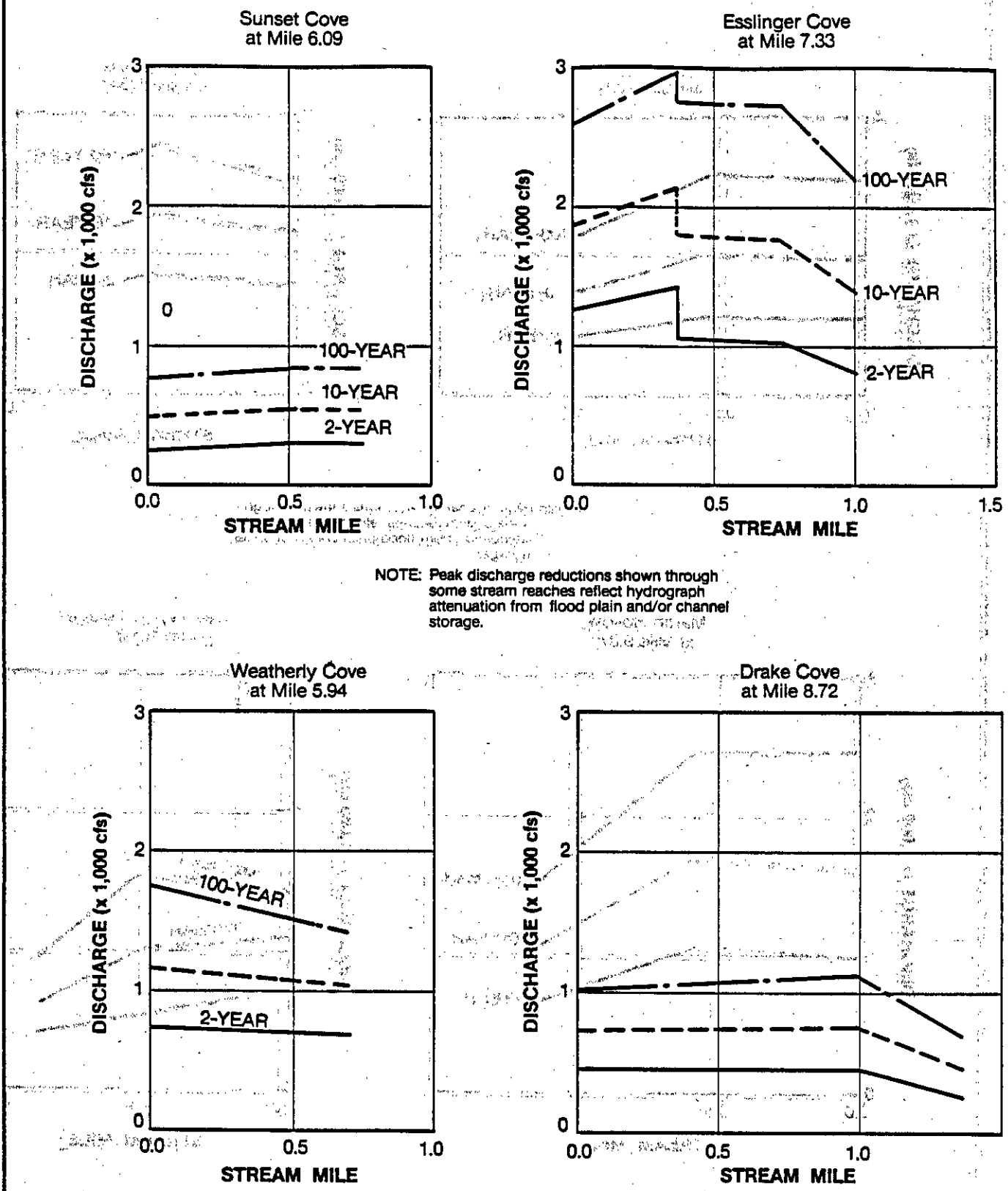
NOTE: Peak discharge reductions shown through some stream reaches reflect hydrograph attenuation from flood plain and/or channel storage.





Reference: TVA (1986).

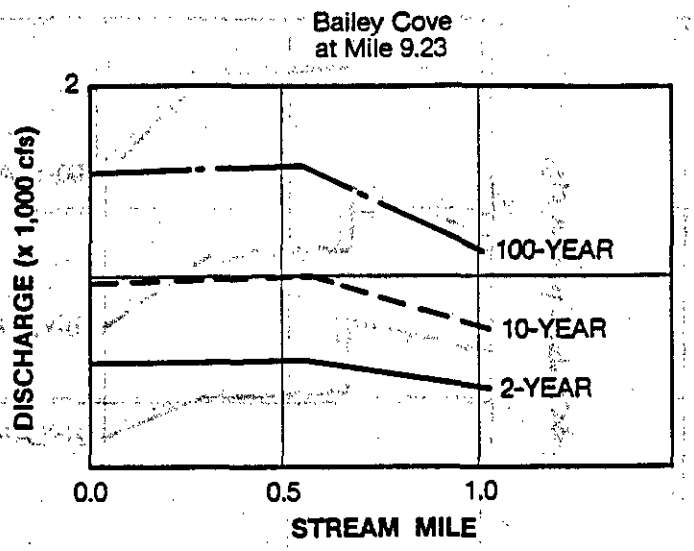
FIGURE 4-31
Flow Frequency Estimates for Aldridge Creek Tributaries

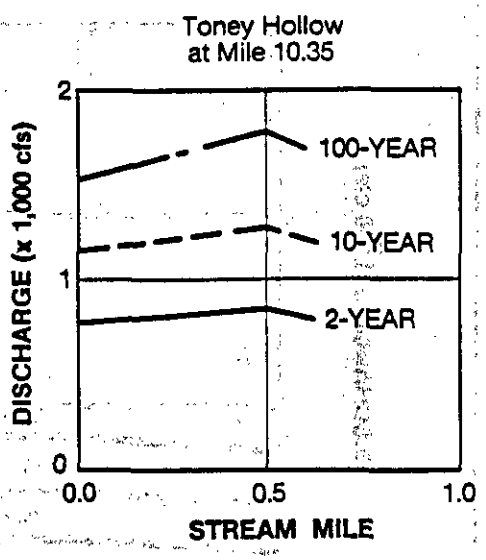


NOTE: Peak discharge reductions shown through some stream reaches reflect hydrograph attenuation from flood plain and/or channel storage.

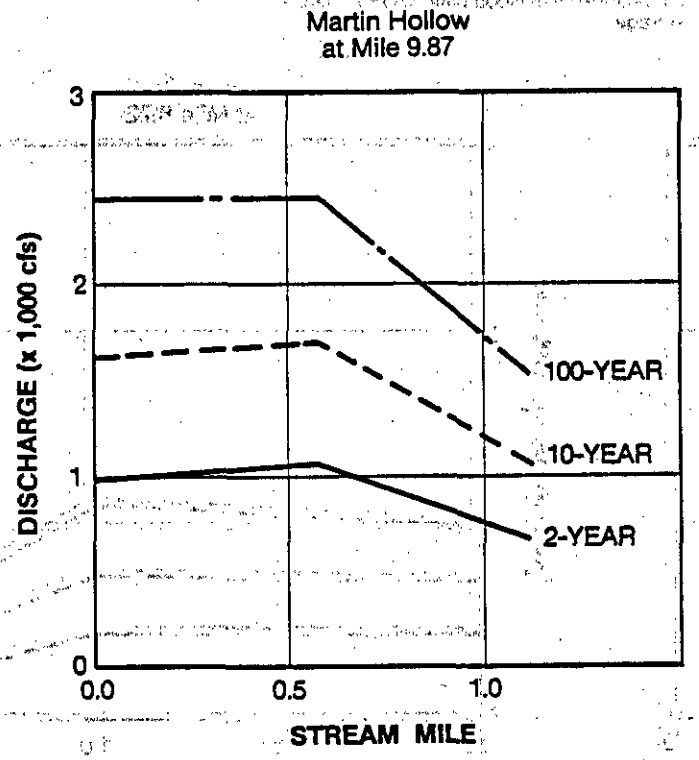
Reference: TVA (1986).

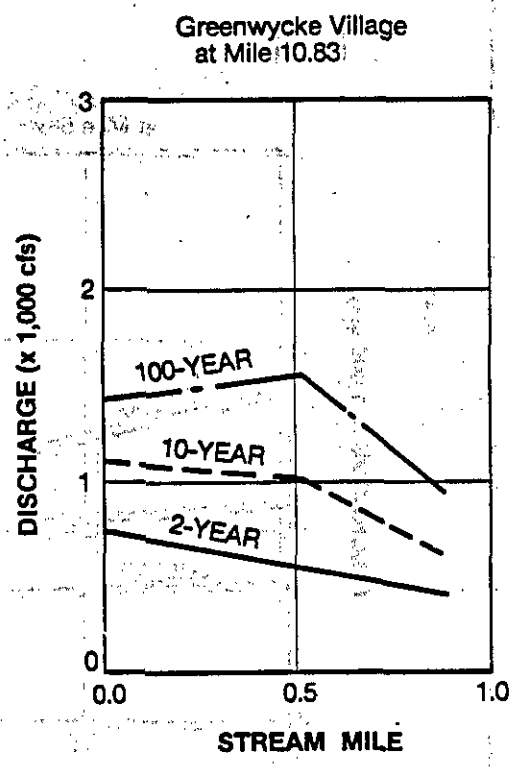
FIGURE 4-32
Flow Frequency Estimates for Aldridge Creek Tributaries





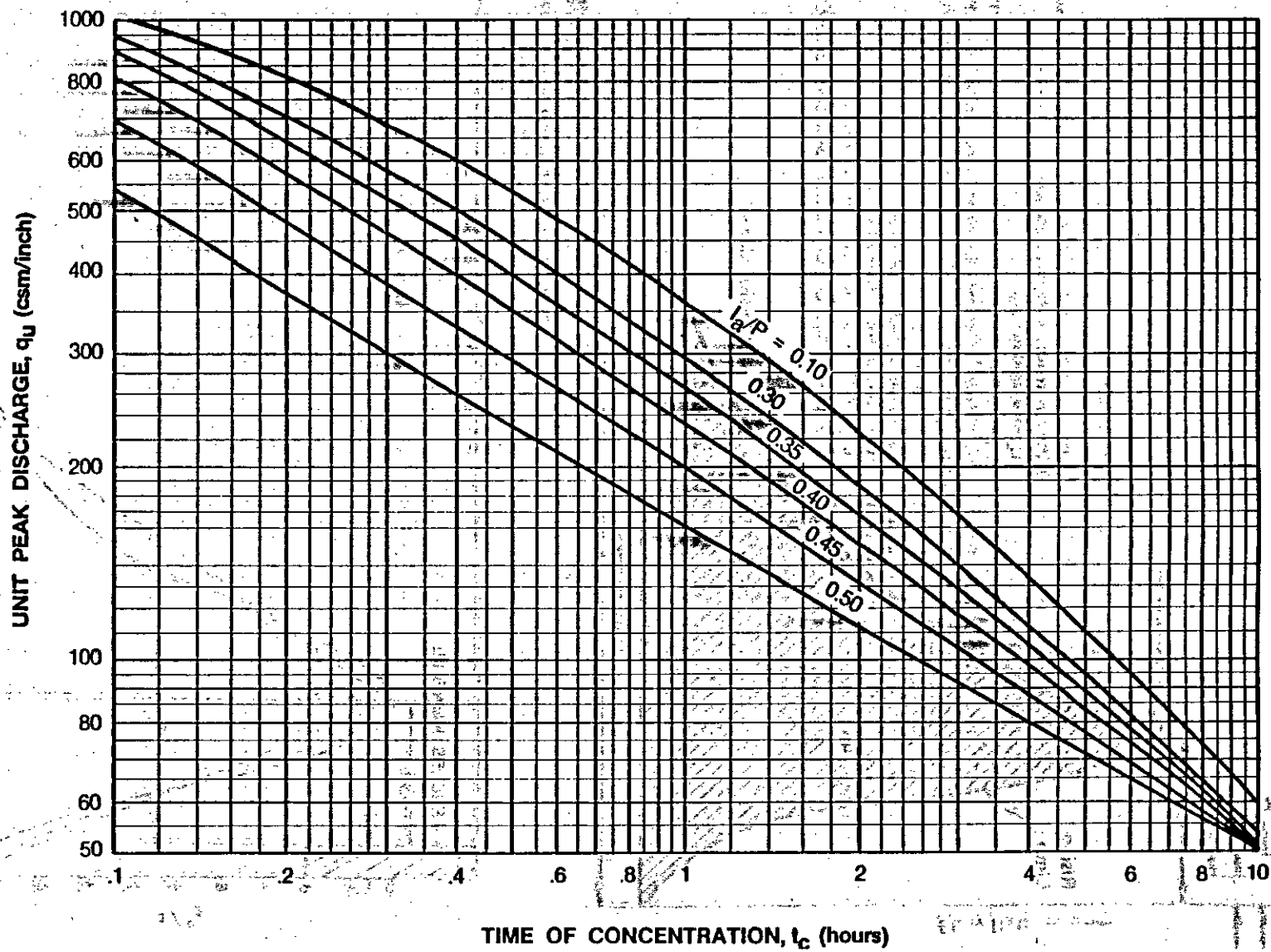
NOTE: Peak discharge reductions shown through some stream reaches reflect hydrograph attenuation from flood plain and/or channel storage.





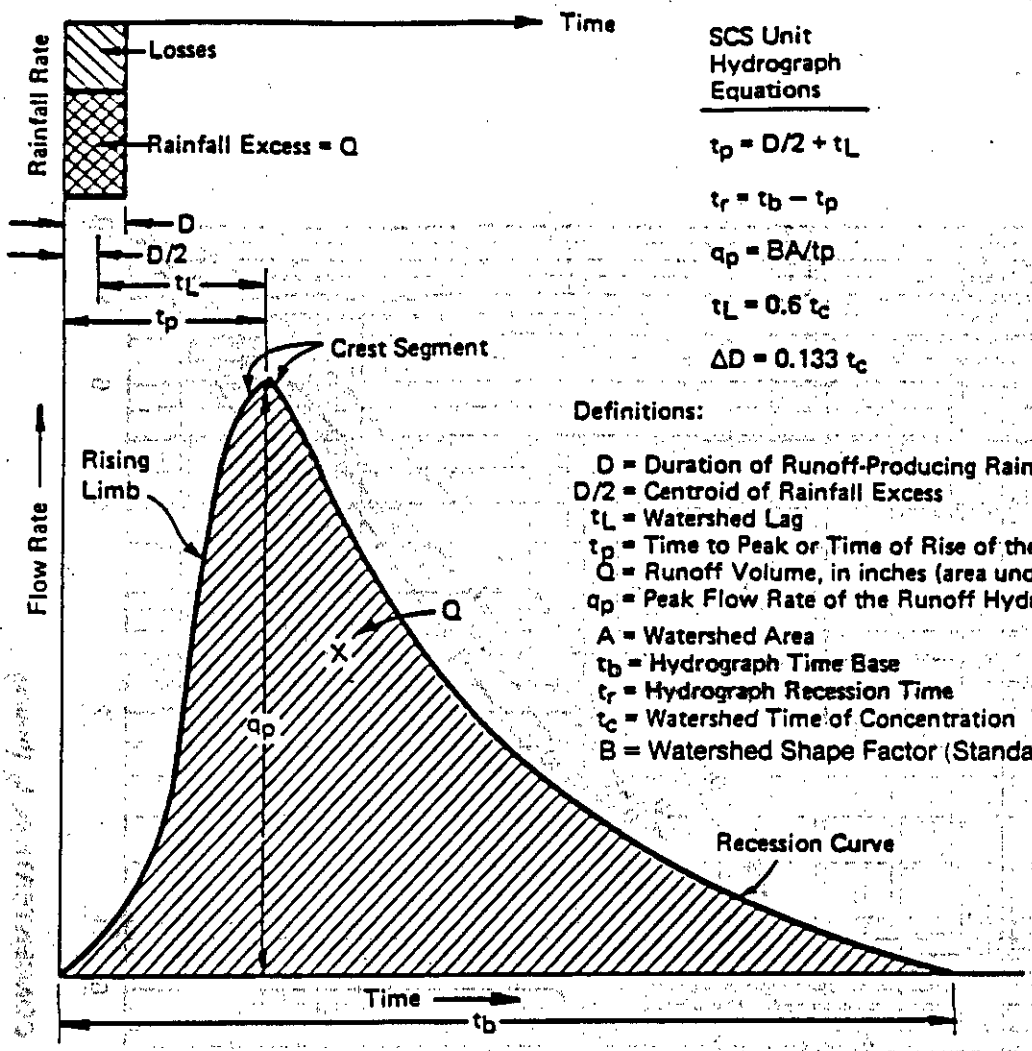
Reference: TVA (1986).

FIGURE 4-33
Flow Frequency Estimates for Aldridge Creek Tributaries



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

FIGURE 4-34
Unit Peak Discharge (q_u) for SCS Type II Rainfall Distribution



SCS Unit Hydrograph Equations

$t_p = D/2 + t_L$

$t_r = t_b - t_p$

$q_p = BA/t_p$

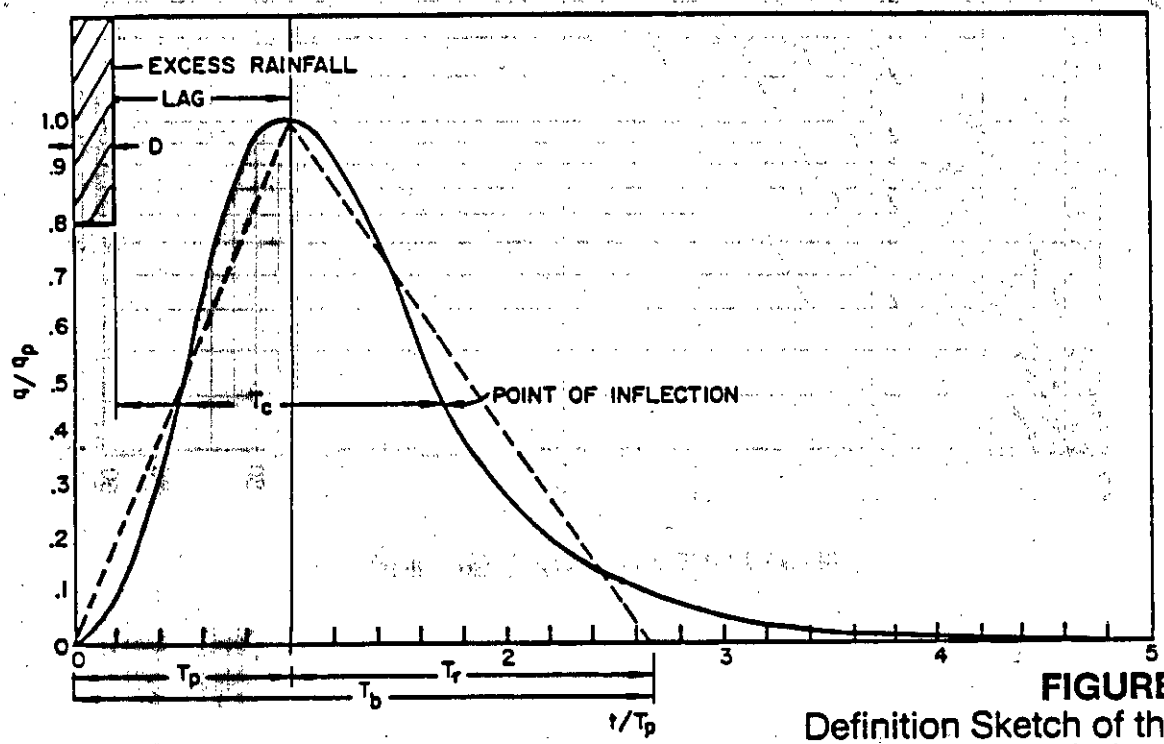
$t_L = 0.6\ t_c$

$\Delta D = 0.133\ t_c$

Definitions:

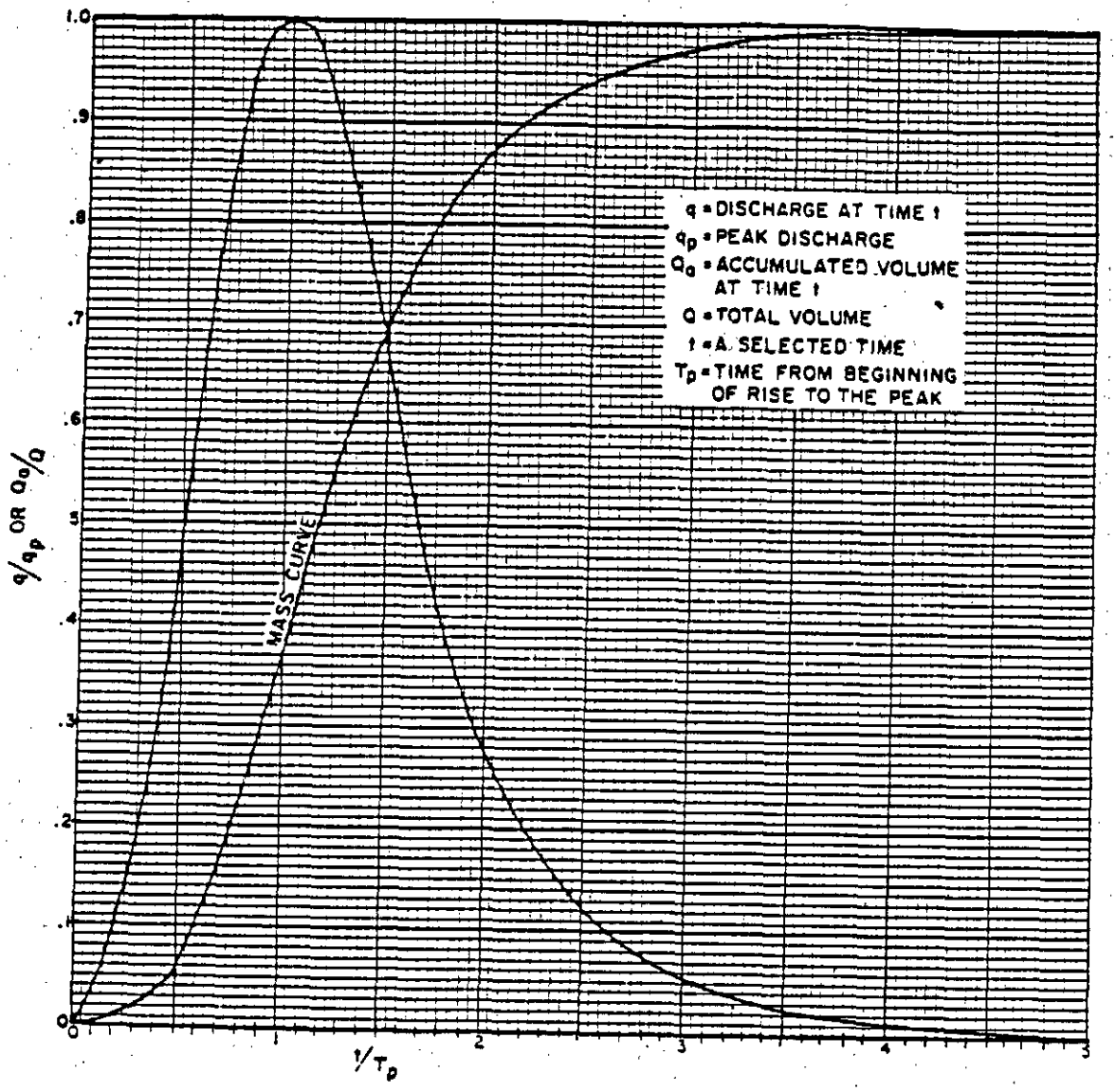
D = Duration of Runoff-Producing Rainfall in Hours
D/2 = Centroid of Rainfall Excess
t_L = Watershed Lag
t_p = Time to Peak or Time of Rise of the Runoff Hydrograph
Q = Runoff Volume, in inches (area under the curve)
q_p = Peak Flow Rate of the Runoff Hydrrograph in cfs
A = Watershed Area
t_b = Hydrograph Time Base
t_r = Hydrograph Recession Time
t_c = Watershed Time of Concentration
B = Watershed Shape Factor (Standard SCS B = 484)

FIGURE 4-35
General Hydrograph Terminology



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

FIGURE 4-36
Definition Sketch of the SCS Dimensionless Unit Hydrographs



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

FIGURE 4-37
SCS Dimensionless Unit Hydrograph and Mass Curve

CHAPTER 5
Open Channel Hydraulics

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Chapter 5
OPEN CHANNEL HYDRAULICS

5.1 LININGS

Open channel linings are divided into the following four main classifications:

1. Vegetative (grass with mulch and sod)
2. Flexible (rock riprap and geotextile or interlocking concrete grids)
3. Rigid (concrete or grouted riprap)
4. Temporary (synthetic or natural materials to protect soil until permanent stabilization is established)

5.1.1 VEGETATIVE

A healthy vegetative lining can stabilize the body of the channel, consolidate the soil mass of the bed, check erosion on the channel surface, and control the movement of soil particles along the channel bottom. Conditions under which a vegetative lining may not be acceptable, however, include but are not limited to:

1. Flow conditions in excess of the velocity limitations presented in Section 5.2.2
2. Standing or continuous flowing water
3. Lack of the regular maintenance necessary to prevent domination by taller vegetation
4. Lack of nutrients and inadequate topsoil
5. Excessive shade

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of a healthy growth of grass. Soil testing should be performed and the

results evaluated by an agronomist to determine soil treatment requirements for parameters such as pH, nitrogen, phosphorus, potassium, and other factors (see Table 10-1). In many cases, temporary erosion control measures are required to provide time for the seeding to establish a viable vegetative lining (see Chapter 10). Sodding should be staggered, to avoid seams in the direction of flow.

5.1.2 FLEXIBLE

Flexible linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. Rock riprap including rubble is the most common type of flexible lining. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocities. The use of flexible lining may be restricted where right-of-way is limited, because the higher roughness values create larger cross sections.

5.1.3 RIGID

Rigid linings are generally constructed in places with limited right-of-way, where a smooth lining offers a higher capacity for a given cross-sectional area. 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. Filter fabric is required to prevent soil loss through pavement cracks.

5.1.4 TEMPORARY

Synthetic (e.g., geotextile) or natural (e.g., straw mulch) materials are required to protect exposed soil until permanent stabilization is established. The layout for temporary runoff control measures should be consistent with the layout of permanent facilities.

5.2 DESIGN CRITERIA

5.2.1 RETURN PERIOD

The design storm return period for open channel systems shall be 25 years.

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.

5.2.2 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 5-1. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 5-2. Vegetative lining calculations are presented in Section 5.3.5 and riprap procedures are presented in Section 5.3.6.

5.2.3 MANNING'S n VALUES

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and riprap linings are presented in Table 5-3. Recommended values for vegetative linings should be determined using Figure 5-1, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 5-4). Figure 5-1 is used iteratively as described in Section 5.3.5.

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

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \qquad (5\text{-}1)$$

where:

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

n_0 = Coefficient associated with channel lining material

n_1 = Coefficient associated with the degree of channel irregularity

n_2 = Coefficient associated with variations of the channel cross section

n_3 = Coefficient associated with the relative effect of channel obstructions

n_4 = Coefficient associated with channel vegetation

m_5 = Coefficient associated with the degree of channel meandering

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

5.2.4 SIDE SLOPES

Maximum open channel side slopes shall not be steeper than the following values, unless slope stability calculations, approved by the Engineer of Public Works, indicate that steeper slopes are stable:

Lining Material	Maximum Side Slope
Concrete, Grouted Riprap	1:1
Ungrouted Riprap, Rock Lined, Solid Sod in Non-Residential Subdivision	2:1
Vegetative in Non-Residential Subdivision	3:1
Vegetative in Residential Subdivision, Includes Solid Sod	4:1

5.2.5 RIPRAP LINING

Guidelines for specifying riprap gradation, thickness, and filter material are as follows:

1. Gradation: The size distribution curve should be smooth; the following range of particle sizes is acceptable:

$$\frac{D_{100}}{D_{50}} \text{ or } \frac{D_{50}}{D_{20}} \leq 3.0 \text{ and } \geq 1.5$$

where:

D_{100} = 100% of the material is smaller than this size

D_{50} = 50% of the material is smaller than this size, or mean particle size

D_{20} = 20% of the material is smaller than this size

2. Thickness: The thickness of riprap lining shall be greater than or equal to the diameter of the largest rock in the gradation, or 1.5 times the mean rock diameter, whichever is greater.

3. Filter material: Filter material underlying rock riprap can be either a granular filter layer or a geotextile fabric. Major concerns include soil retention, water permeability, and clogging. In general, the permeability of the filter material should be greater than the permeability of the native soil.

5.3 UNIFORM FLOW CALCULATIONS

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

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

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \quad (5\text{-}3)$$

$$S = \left(\frac{Qn}{1.49 A R^{2/3}}\right)^2 \quad (5\text{-}4)$$

where:

v = Average channel velocity, in feet/second

Q = Discharge rate for design conditions,

n = Manning's roughness coefficient (see Section 5.2.3)

A = Cross-sectional area, in square feet

R = Hydraulic radius A/P, in feet

P = Wetted perimeter, in feet

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

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

5.3.1 GEOMETRIC RELATIONSHIPS

Mathematical expressions are presented in Figure 5-2 for calculating the following parameters:

1. Area
2. Wetted perimeter
3. Hydraulic radius
4. Channel top width
5. Critical depth section factor

The selected cross sections include the following types:

1. Trapezoidal
2. Rectangular
3. Triangular
4. Parabolic
5. Circular

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 (U.S. Geological Survey, 1976a,b).

5.3.2 DIRECT SOLUTIONS

When the hydraulic radius, cross-sectional area, slope, and roughness coefficient of an open channel are known, discharge can be calculated directly from Equation 5-3. Design aids for the direct solution of Manning's Equation are presented in the form of tables, charts, and nomographs.

A constant parameter, which depends on only the geometry and channel roughness, can be used to simplify the direct solution of Manning's Equation as follows:

$$Q = C_1 S^{1/2} \tag{5-5}$$

$$S = \left(Q/C_1\right)^2 \tag{5-6}$$

where:

Q = Discharge for design conditions, in cfs

$$C_1 = \frac{1.49}{n} A R^{2/3} \tag{5-7}$$

A = Cross-sectional area, in square feet

R = Hydraulic radius, A/P, in feet

P = Wetted perimeter, in feet

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

Tables

Tables of C_1 values are provided for concrete pipe with the following cross sections:

Circular	Table 5-6
Elliptical	Table 5-7
Arch	Table 5-8
Precast Box	Table 5-9

Using the C value obtained from the appropriate table, the capacity or slope can be obtained directly by solving Equation 5- or 5-6.

Charts

Full flow capacity charts for the following pipe shapes and roughness coefficients are duplicated from the American Concrete Pipe Association (1980) as the figures listed:

Arch, n = 0.011, Figure 5-3
Arch, n = 0.012, Figure 5-4
Arch, n = 0.013, Figure 5-5
Box, n = 0.012, sizes 3'x 2' to 8'x 8', Figure 5-6
Box, n = 0.012, sizes 9'x 5' to 12'x 12', Figure 5-7
Box, n = 0.013, sizes 3'x 2' to 8'x 8', Figure 5-8
Box, n = 0.013, sizes 9'x 5' to 12'x 12', Figure 5-9

These charts provide a direct graphical solution for the capacity solution to Manning's Equation (Equation 5-3) when all but one variable in the equation are known.

Nomographs

Direct solutions to Manning's Equation can be obtained using the nomographs presented in Figures 5-10 and 5-11. Part A of Figure 5-10 provides a general solution for the velocity form of Manning's Equation (Equation 5-2), while Part B provides capacity and velocity solutions for circular pipe. Figure 5-11 provides a nomograph solution for trapezoidal channels.

General. The following steps are used for the general solution nomograph in Part A of Figure 5-10:

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.

Circular. The following steps are used for the circular pipe nomograph in Part B of Figure 5-10:

1. Determine input data, including slope in feet/foot, Manning's n value, and pipe diameter in inches or feet.

2. Connect a line from the slope scale, Point 1, to the Manning's n scale, Point 2, and note the point of intersection on the turning line, Point 3.

3. Connect a line from the pipe diameter, Point 5, to the point of intersection obtained in Step 2, Point 3.

4. Extend the line from Step 3 to the discharge and velocity scales to read the discharge at Point 4 and the velocity at Point 6.

Trapezoidal. The trapezoidal channel nomograph solution to Manning's Equation in Figure 5-11 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

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

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

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

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

d. 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.

5.3.3 TRIAL AND ERROR SOLUTIONS

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

$$AR^{2/3} = \frac{Qn}{1.49\ S^{1/2}} \qquad (5\text{-}8)$$

where:

A = Cross-sectional area, in square feet

R = Hydraulic radius (A/P), in feet

P = Wetted perimeter, in feet

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 5.2.3)

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

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of Equation 5-8 is satisfied such that the design flow is conveyed within the banks of the desired channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figures 5-12 through 5-15, which are described below.

Trapezoidal Channels

For trapezoidal channels, Figure 5-12 can be used to make capacity calculations as follows:

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:

$$K_T = \frac{Qn}{b^{8/3} S^{1/2}} \qquad (5\text{-}9)$$

where:

K_T = Trapezoidal open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 5.2.3)

b = Bottom width, in feet

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

3. Enter the x-axis of Figure 5-12 with the value of K_T calculated in Step 2 and draw a line vertically to the curve that corresponds 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.

Circular Pipe

For partial flow in a circular pipe, Figure 5-13 can be used for capacity calculations as follows:

1. Determine input data, including design discharge, Q, Manning's n value, pipe diameter, D, and channel slope, S.

2. Calculate the circular pipe conveyance factor using the equation:

$$K_p = \frac{Qn}{D^{8/3} S^{1/2}} \qquad (5\text{-}10)$$

where:

K_p = Circular pipe open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient

D = Pipe diameter, in feet

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

3. Enter the x-axis of Figure 5-13 with the value of K_p calculated in Step 2 and draw a line vertically to the curve.

4. From the point of intersection obtained in Step 3, draw 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.

An alternative procedure for evaluating partial flow at any depth in circular pipe is to adjust the full pipe capacity using Figure 5-14.

Pipe Arch

The capacity of pipe arch at any depth can be estimated using Figure 5-15 to adjust the full pipe capacity. Full pipe capacity for pipe arch can be estimated using data in Table 5-8 or capacity charts in Figures 5-3, 5-4, or 5-5.

5.3.4 CRITICAL FLOW CALCULATIONS

A design flow at or near critical depth (±10 percent) should be avoided, because such flow conditions are not stable. The general equation for determining critical depth is expressed as:

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad (5\text{-}11)$$

where:

Q = Discharge rate for design conditions, in cfs

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

A = Cross-sectional area, in square feet

T = Top width of water surface, in feet

It is important to note that critical depth depends only on the discharge rate and channel geometry. A trial and error

procedure is required to solve Equation 5-11. Equations (as presented in Table 5-10) or section factors (as presented in Figure 5-2) can be used to simplify trial and error critical depth calculations. The following equation from Chow (1959) is used to determine critical depth using an appropriate critical flow section factor, Z:

$$Z = Q/\sqrt{g} \qquad (5\text{-}12)$$

where:

Z = Critical flow section factor (see Table 5-10)

Q = Discharge rate for design conditions, in cfs

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

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

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

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

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

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

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

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

$$Fr = v/(gA/T)^{0.5} \qquad (5\text{-}13)$$

where:

Fr = Froude number, dimensionless

v = velocity of flow, in feet/second

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

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

T = Top width of flow, in feet

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

5.3.5 VEGETATIVE DESIGN

A two-part procedure, adapted from Chow (1959) and presented below, is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determing channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 5-4. 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 5-4. If temporary lining is to be used during construction, vegetative retardance Class E should be used for 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 5-2 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 5-1. Use retardance Class D for permanent vegetation and E for temporary construction.

4. Calculate the hydraulic radius using the equation:

$$R = \frac{(vR)}{v_m} \qquad (5\text{-}14)$$

where:

R = Hydraulic radius of flow, in feet

vR = Value obtained from Figure 5-1 in Step 2

v_m = Maximum velocity from Step 2

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

$$vR = \frac{1.49\ R^{5/3}\ S^{1/2}}{n} \qquad (5\text{-}15)$$

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 5-1 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

7. Use the final Manning's n value from Step 6 to select channel dimensions that give a hydraulic radius close to the final value from Step 6. For trapezoidal channels, the flow depth can be estimated using Figures 5-11 or 5-12. The depth of flow for other channel shapes can be evaluated using the trial and error procedure in Section 5.3.3.

8. If bends are considered, calculate the length of downstream protection, L_p, for the bend using Figure 5-16. 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 5-2 for equations).

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

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

4. Use Figure 5-1 to find a Manning's n value for retardance Class C based on the vR value from Step 3.

5. Use Manning's Equation (Equation 5-2) or Figure 5-10 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

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

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

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

$$\Delta d = \frac{v^2 T}{g R_c} \qquad (5\text{-}16)$$

where:

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

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

T = Top width of flow, in feet

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

R_c = Mean radius of the bend, in feet

Add freeboard consistent with the calculated Δd.

5.3.6 RIPRAP DESIGN

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 5.3.3). Manning's n values for riprap can be calculated from the equation:

$$n = 0.0395 (d_{50})^{1/6} \qquad (5\text{-}17)$$

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 5-17 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 5-18.

4. Determine the required minimum d_{30} value from Figure 5-19, which is based on the equation:

$$d_{30}/D = 0.193\ Fr^{2.5} \qquad (5\text{-}18)$$

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 5-13), dimensionless

v = Mean velocity above the stone in feet/second

g = Acceleration of gravity, 32.2 feet/second²

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:

$$W = 0.5236\ \gamma_s\ d^3 \qquad (5\text{-}19)$$

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 inc
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 5-20 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

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

5.3.7 EXAMPLE PROBLEMS

Example 5-1. Direct Solution of Manning's Equation

Use Manning's Equation to find the full flow capacity, Q, of a 48-inch inside diameter, circular concrete pipe storm sewer with an n value of 0.012 and slope of 0.006 foot/foot.

1. Solve using Part B of Figure 5-10.

   a. Connect a line between the slope scale at 0.006 and the roughness scale at 0.012 and note the intersection point on the turning line.

   b. Connect a line between that intersection point and the pipe diameter scale at 48 inches and read the pipe capacity of 120 cfs from the discharge scale.

   c. Proceed to the velocity scale and read a value of 9.6 feet/second.

2. Solve using Table 5-6.

a. Look under the column n = 0.012 for a 48-inch diameter pipe and find C_1 = 1556.

b. For S = 0.006, calculate $S^{1/2}$ = 0.07746.

c. For Q, use Equation 5-5 to find (1556)(0.07746) = 121 cfs.

d. Use Table 5-6 to find A = 12.566 square feet. Since Q = Av, v = 121/12.566 = 9.6 feet/second.

Example 5-2. Grassed Channel Design Stability

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

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

2. Assume an n value of 0.035 and find the value of vR from Figure 5-1.

   vR = 5.4

3. Use Equation 5-14 to calculate the value of R.

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

4. Use Equation 5-15 to calculate the value of vR.

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

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 5-1)	R (Equation 5-14)	vR (Equation 5-15)
0.035	5.4	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

6. Use Figure 5-11 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

   Qn = (50)(0.040) = 2.0

   S = 0.015

   Try b = 8 feet, d = (8)(0.14) = 1.12 feet
   Since R = 0.844 foot and a value of 0.80 foot is required, b must be increased.

   Try b = 10 feet, d = (10)(0.098) = 0.98 foot
   Since R = 0.796 foot, channel geometry is adequate.

   Select: b = 10 feet
   z = 3
   d = 1 foot
   v = 3.9 feet/second (Equation 5-2)
   Fr = 0.76 (Equation 5-13)
   Flow is subcritical

   Design capacity calculations for this channel are presented in Example 5-3.

Example 5-3. Grassed Channel Design Capacity

Use a 10-foot bottom width for the trapezoidal channel sized in Example 5-2 and find the depth of flow for retardance Class C.

1. Assume a depth of 1.0 foot and calculate the following (see Figure 5-2):

   A = (b + zd) d

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

$A = 13.0$ square feet

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

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

$R = 0.796$ foot

2. Find the velocity.

$v = 50/13.0$

$v = 3.85$ feet/second

3. Find the value of vR.

$vR = (3.85)(0.796) = 3.06$

4. Using the vR product from Step 3, find Manning's n from Figure 5-1 for retardance Class C.

$n = 0.047$

5. Use Figure 5-11 or Equation 5-2 to find the velocity for $S = 0.015$, $R = 0.796$, and $n = 0.047$.

$v = 3.34$ feet/second

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

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 5-1)	Velocity (Eq. 5-2)
1	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.0475	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

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

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

   Q = 50 cfs

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

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

   n (stability) = 0.040, d = 1.0 foot, v = 3.9 feet/second, Froude number = 0.76 (Equation 5-13)

   n (capacity) = 0.048, d = 1.1 feet, v = 3.45 feet/second, Froude number = 0.64 (Equation 5-13)

Example 5-4. Grassed Channel Vegetation Comparison

A trapezoidal channel is required to carry 97 cfs at a bottom slope of 0.0275 foot/foot. Compare channel dimensions required for a grass mixture and bermudagrass.

1. Design stability calculations for a grass mixture are summarized below:

   v_m = 4 feet/second (from Table 5-2)

   Retardance Class = D

Assumed n Value	vR (Fig. 5-1)	R (Eq. 5-14)	vR (Eq. 5-15)
0.040	3.2	0.80	4.26
0.041	2.85	0.71	3.43
0.042	2.6	0.65	2.87
0.043	2.4	0.60	2.45

   Select n = 0.043 for stability criteria.

2. To obtain a hydraulic depth of 0.6 foot, a 39-foot bottom width is required. Depth calculations are summarized as follows:

Assumed Depth (ft)	Area (ft^2)	R (ft)	$AR^{2/3}$	$\frac{Qn}{1.49S^{0.5}}$	v (ft/sec)
0.40	16.08	0.387	8.54	16.88	3.05
0.50	20.25	0.480	12.42	16.88	3.52
0.60	24.48	0.572	16.87	16.88	3.96

Find normal depth = 0.60 foot, v = 3.96 feet/second

3. Design capacity calculations for the grass mixture are summarized as follows:

Retardance Class = C

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (cfs)	vR	n (Fig. 5-1)	v (ft/sec)
0.60	24.48	0.572	3.96	2.27	0.054	3.15
0.65	26.62	0.617	3.64	2.25	0.054	3.32
0.70	28.77	0.662	3.37	2.23	0.054	3.48
0.69	28.34	0.653	3.42	2.24	0.054	3.45

Select n = 0.054 and d = 0.69 foot.

4. Design stability calculations for bermudagrass are summarized below:

v_m = 6 feet/second (from Table 5-2)

Retardance Class = D

Assumed n Value	vR (Fig 5-1)	R (Eq 5-14)	vR (Eq 5-15)
0.035	5.4	0.90	5.92
0.036	4.8	0.80	4.73

Select n = 0.036 for stability criteria.

5. To obtain a hydraulic depth of 0.8 foot, a 15-foot bottom width is required. Depth calculations are as follows:

Assumed Depth (ft)	Area (ft^2)	R (ft)	$AR^{2/3}$	$1.49S^{0.5}$	v (ft/sec)
0.8	13.92	0.694	10.91	14.13	5.38
0.9	15.93	0.770	13.38	14.13	5.77
0.93	16.54	0.792	14.17	14.13	5.88

Find normal depth = 0.93 foot, v = 5.88 feet/second.

6. Design capacity calculations for bermudagrass using retardance Class C are summarized as follows:

Assumed Depth (ft)	Area (ft^2)	R (ft)	Velocity Q/A (cfs)	vR	n (Fig. 5-1)	v (ft/sec)
0.98	17.58	0.829	5.52	4.58	0.040	5.45
1.0	18.00	0.844	5.39	4.55	0.040	5.52
1.02	18.42	0.859	5.27	4.52	0.042	5.32

Select n = 0.042 and d = 1.02 feet.

7. Vegetative linings are compared as follows:

	Grass Mixture	Bermudagrass
Bottom width	39	15
Side slope	3	3
n (stability)	0.043	0.036
d (stability)	0.60	0.93
v (stability)	3.96	5.87
Fr (stability)	0.919	1.15
n (capacity)	0.054	0.042
d (capacity)	0.69	1.02
d (w/freeboard)	0.84	1.22
v (capacity)	3.44	5.29
Fr (capacity)	0.75	1.00
Top width (w/freeboard)	44.0	22.3

Example 5-5. 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 5-17, $C_b = 1.55$. The adjusted effective velocity is $(8)(1.55) = 12.4$ feet per second.

3. Determine the correction coefficient for the specific weight of 170 pounds from Figure 5-18 as 0.98. The adjusted effective velocity is $(12.4)(0.98) = 12.15$ feet per second.

4. Determine minimum d_{30} from Figure 5-19 or Equation 5-18 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 5-20, this gradation would be acceptable if the blanket thickness were increased from the original d_{100} thickness by 35 percent (a ratio of 1.35 on the horizontal axis).

7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 5-16.

5.4 GRADUALLY VARIED FLOW

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

Many computer programs are available for computation of backwater curves. The most general and widely used program, HEC-2, was developed by the U.S. Army Corps of Engineers (1982) and is 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, also presented by Chow (1959), is recommended, although it is a more tedious and iterative process. The use of HEC-2 is recommended for standard step calculations.

5.5 RAPIDLY VARIED FLOW

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

5.5.1 HYDRAULIC JUMP

A hydraulic jump can occur when flow passes rapidly from supercritical to subcritical depth and can be designed to dissipate highly erosive velocities. 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 will result. Control 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.

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/sewers and for spillways using the jump characteristics of rectangular sections.

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

$$d_2 = -\frac{d_1}{2} + \left(\frac{d_1^2}{4} + \frac{2v_1^2 d_1}{g}\right)^{1/2} \qquad (5\text{-}20)$$

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 5-20 is presented in Figure 5-21. Additional details on evaluating hydraulic jumps can be found in publications by French (1985) USDOT, FHWA (HEC-14, 1983), Chow (1959), and Peterska (1978).

5.5.2 BRIDGES

Water surface profile calculations in the vicinity of bridges are generally performed using a computer program such as HEC-2 or WSPRO (USDOT, FHWA, HY-7, 1986). The procedure selected should be consistent with results from previous studies, if available. For example, if changes to regulatory flood elevations and floodways are to be evaluated, the program and data files for the original study should be used.

Table 5-1
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 5-2
Rigid[a]	10

[a]Higher velocities may be acceptable for rigid linings if energy dissipation is provided (see USDOT, FHWA, HEC-14, 1983).

Table 5-2
MAXIMUM VELOCITIES FOR VEGETATIVE CHANNEL LININGS

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

Reference: USDA, TP-61 (1947).

[a]Based on erosive soils.

Table 5-3
RECOMMENDED MANNING'S n VALUES FOR ARTIFICIAL CHANGES

Lining Category	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	0.020	0.020
	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	6-inch (15-cm) D_{50}	0.104	0.069	0.035
	12-inch (30-cm) D_{50}	NA	0.078	0.040

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

Table 5-4
CLASSIFICATION OF VEGETAL 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
	Bermuda grass	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") (28 cm)
C	Crabgrass	Fair stand, uncut (10 to 48") (25 to 120 cm)
	Bermuda grass	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	Bermuda grass	Good stand, cut to 2.5-inch height (6 cm)
	Common lespedeza	Excellent stand, uncut (average 4.5") (11 cm)
	Buffalo grass	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 serices	After cutting to 2-inch height (5 cm) Very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5-inch height (4 cm)
	Bermuda grass	Burned stubble

Reference: USDA, TP-61 (1947).

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

Table 5-5
COEFFICIENTS FOR COMPUTING MANNING'S n VALUES
FOR NATURAL OR EXCAVATED CHANNELS USING COWAN'S EQUATION[a]

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

[a]Cowan's Equation is presented as Equation 5-1.

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

D Pipe Diameter (inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = \frac{1.486}{n} \times A \times R^{2/3}$			
			n=0.010	n=0.011	n=0.012	n=0.013
8	0.349	0.167	15.8	14.3	13.1	12.1
10	0.545	0.208	28.4	25.8	23.6	21.8
12	0.785	0.250	46.4	42.1	38.6	35.7
15	1.227	0.312	84.1	76.5	70.1	64.7
18	1.767	0.375	137	124	114	105
21	2.405	0.437	206	187	172	158
24	3.142	0.500	294	267	245	226
27	3.976	0.562	402	366	335	310
30	4.909	0.625	533	485	444	410
33	5.940	0.688	686	624	574	530
36	7.069	0.750	867	788	722	666
42	9.621	0.875	1308	1189	1090	1006
48	12.566	1.000	1867	1698	1556	1436
54	15.904	1.125	2557	2325	2131	1967
60	19.635	1.250	3385	3077	2821	2604
66	23.758	1.375	4364	3967	3636	3357
72	28.274	1.500	5504	5004	4587	4234
78	33.183	1.625	6815	6195	5679	5242
84	38.485	1.750	8304	7549	6920	6388
90	44.170	1.875	9985	9078	8321	7681
96	50.266	2.000	11850	10780	9878	9119
102	56.745	2.125	13940	12670	11620	10720
108	63.617	2.250	16230	14760	13530	12490
114	70.882	2.375	18750	17040	15620	14420
120	78.540	2.500	21500	19540	17920	16540
126	86.590	2.625	24480	22260	20400	18830
132	95.033	2.750	27720	25200	23100	21330
138	103.870	2.875	31210	28370	26010	24010
144	113.100	3.000	34960	31780	29130	26890

Reference: American Concrete Pipe Association (1980).

TABLE 5-6
Full Flow C_1 Values for Circular Concrete Pipe

Pipe Size R x S (HE) S x R (VE) (Inches)	Approximate Equivalent Circular Diameter (Inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = \frac{1.486}{n} \times A \times R^{2/3}$			
				n = 0.010	n = 0.011	n = 0.012	n = 0.013
14 x 23	18	1.8	0.367	138	125	116	108
19 x 30	24	3.3	0.490	301	274	252	232
22 x 34	27	4.1	0.546	405	368	339	313
24 x 38	30	5.1	0.613	547	497	456	421
27 x 42	33	6.3	0.686	728	662	607	560
29 x 45	36	7.4	0.736	891	810	746	686
32 x 49	39	8.8	0.812	1140	1036	948	875
34 x 53	42	10.2	0.875	1386	1260	1156	1067
38 x 60	48	12.9	0.969	1878	1707	1565	1445
43 x 68	54	16.6	1.106	2635	2395	2196	2027
48 x 76	60	20.5	1.229	3491	3174	2910	2686
53 x 83	66	24.8	1.352	4503	4094	3753	3464
58 x 91	72	29.5	1.475	5680	5164	4734	4370
63 x 98	78	34.6	1.598	7027	6388	5856	5406
68 x 106	84	40.1	1.721	8560	7790	7140	6590
72 x 113	90	46.1	1.845	10300	9365	8584	7925
77 x 121	96	52.4	1.967	12220	11110	10190	9403
82 x 128	102	59.2	2.091	14380	13070	11980	11060
87 x 136	108	66.4	2.215	16770	15240	13970	12900
92 x 143	114	74.0	2.340	19380	17620	16150	14910
97 x 151	120	82.0	2.461	22190	20180	18490	17070
106 x 166	132	99.2	2.707	28630	26020	23860	22020
116 x 180	144	118.6	2.968	36400	33100	30340	28000

Reference: American Concrete Pipe Association (1980).

TABLE 5-7

Full Flow C_1 Values for Elliptical Concrete Pipe

Pipe Size R x S (Inches)	Approximate Equivalent Circular Diameter (Inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = \frac{1.486}{n} \times A \times R^{2/3}$			
				n = 0.010	n = 0.011	n = 0.012	n = 0.013
11 x 18	15	1.1	0.25	65	59	54	50
13½ x 22	18	1.6	0.30	110	100	91	84
15½ x 26	21	2.2	0.36	165	150	137	127
18 x 28½	24	2.8	0.45	243	221	203	187
22½ x 36¼	30	4.4	0.56	441	401	368	339
26⅝ x 43¾	36	6.4	0.68	736	669	613	566
31 5/16 x 51⅛	42	8.8	0.80	1125	1023	938	866
36 x 58½	48	11.4	0.90	1579	1435	1315	1214
40 x 65	54	14.3	1.01	2140	1945	1783	1646
45 x 73	60	17.7	1.13	2851	2592	2376	2193
54 x 88	72	25.6	1.35	4641	4219	3867	3569
62 x 102	84	34.6	1.57	6941	6310	5784	5339
72 x 115	90	44.5	1.77	9668	8789	8056	7436
77¼ x 122	96	51.7	1.92	11850	10770	9872	9112
87⅛ x 138	108	66.0	2.17	16430	14940	13690	12640
96⅞ x 154	120	81.8	2.42	21975	19977	18312	16904
106½ x 168¾	132	99.1	2.65	28292	25720	23577	21763

Reference: American Concrete Pipe Association (1980).

TABLE 5-8

Full Flow C_1 Values for Concrete Pipe Arch

Box Size Span x Rise (Feet)	A Area (Square Feet)	R Hydraulic Radius (Feet)	C = 1.486/n(A x R²/³)		Box Size Span x Rise (Feet)	A Area (Square Feet)	R Hydraulic Radius (Feet)	C = 1.486/n(A x R²/³)	
			n = 0.012	n = 0.013				n = 0.012	n = 0.013
3 X 2	5.78	0.63	524	484	9 X 5	43.88	1.67	7060	7070
3 X 3	8.78	0.78	923	852	9 X 6	52.88	1.87	9950	9180
4 X 2	7.65	0.69	743	686	9 X 7	61.88	2.05	12400	11400
4 X 3	11.65	0.90	1340	1240	9 X 8	70.88	2.20	14800	13700
4 X 4	15.65	1.04	1990	1840	9 X 9	79.88	2.33	17400	16100
5 X 3	14.50	0.98	1770	1630	10 X 5	48.61	1.73	8690	8020
5 X 4	19.50	1.16	2660	2460	10 X 6	58.61	1.95	11300	10462
5 X 5	24.50	1.30	3620	3340	10 X 7	68.61	2.14	14100	13000
6 X 3	17.32	1.04	2200	2030	10 X 8	78.61	2.31	17000	15700
6 X 4	23.32	1.25	3350	3100	10 X 9	88.61	2.46	20000	18500
6 X 5	29.32	1.42	4590	4240	10 X 10	98.61	2.59	23000	21300
6 X 6	35.32	1.56	5880	5430	11 X 4	42.32	1.52	6930	6390
7 X 4	27.11	1.33	4050	3740	11 X 6	64.32	2.02	12730	11700
7 X 5	34.11	1.52	5590	5160	11 X 8	86.32	2.41	19200	17700
7 X 6	41.11	1.68	7200	6650	11 X 10	108.32	2.72	26100	24100
7 X 7	48.11	1.82	8880	8200	11 X 11	119.32	2.85	29700	27400
8 X 4	31.11	1.39	4790	4420	12 X 4	46.00	1.55	7630	7050
8 X 5	39.11	1.60	6630	6120	12 X 6	70.00	2.08	14100	13000
8 X 6	47.11	1.78	8760	7920	12 X 8	94.00	2.50	21400	19800
8 X 7	55.11	1.94	10600	9790	12 X 10	118.00	2.83	29300	27000
8 X 8	63.11	2.07	12700	11700	12 X 12	142.00	3.11	37500	34600

Reference: American Concrete Pipe Association (1980).

TABLE 5-9

Full Flow C_1 Values for Precast Concrete Box Sections

Table 5-10
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]	$d_c = \left(\frac{Q^2}{g\,b^2}\right)^{1/3}$	N/A
2. Trapezoidal[c]	$d_c = 0.81\left(\frac{Q^2}{g\,z^{0.75}\,b^{1.25}}\right)^{0.27} - \frac{b}{30z}$	$0.1 < 0.5522\,\frac{Q}{b^{2.5}} < 0.4$ For $0.5522\,\frac{Q}{b^{2.5}} < 0.1$, use rectangular channel equation
3. Triangular[c]	$d_c = \left(\frac{2\,Q^2}{g\,z^2}\right)^{1/5}$	N/A
4. Circular[d]	$d_c = 0.325\left(\frac{Q}{D}\right)^{2/3} + 0.083D$	$0.3 < \frac{d_c}{D} < 0.9$
5. General[e]	$\frac{A^3}{T} = \frac{Q^2}{g}$	N/A

where:

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

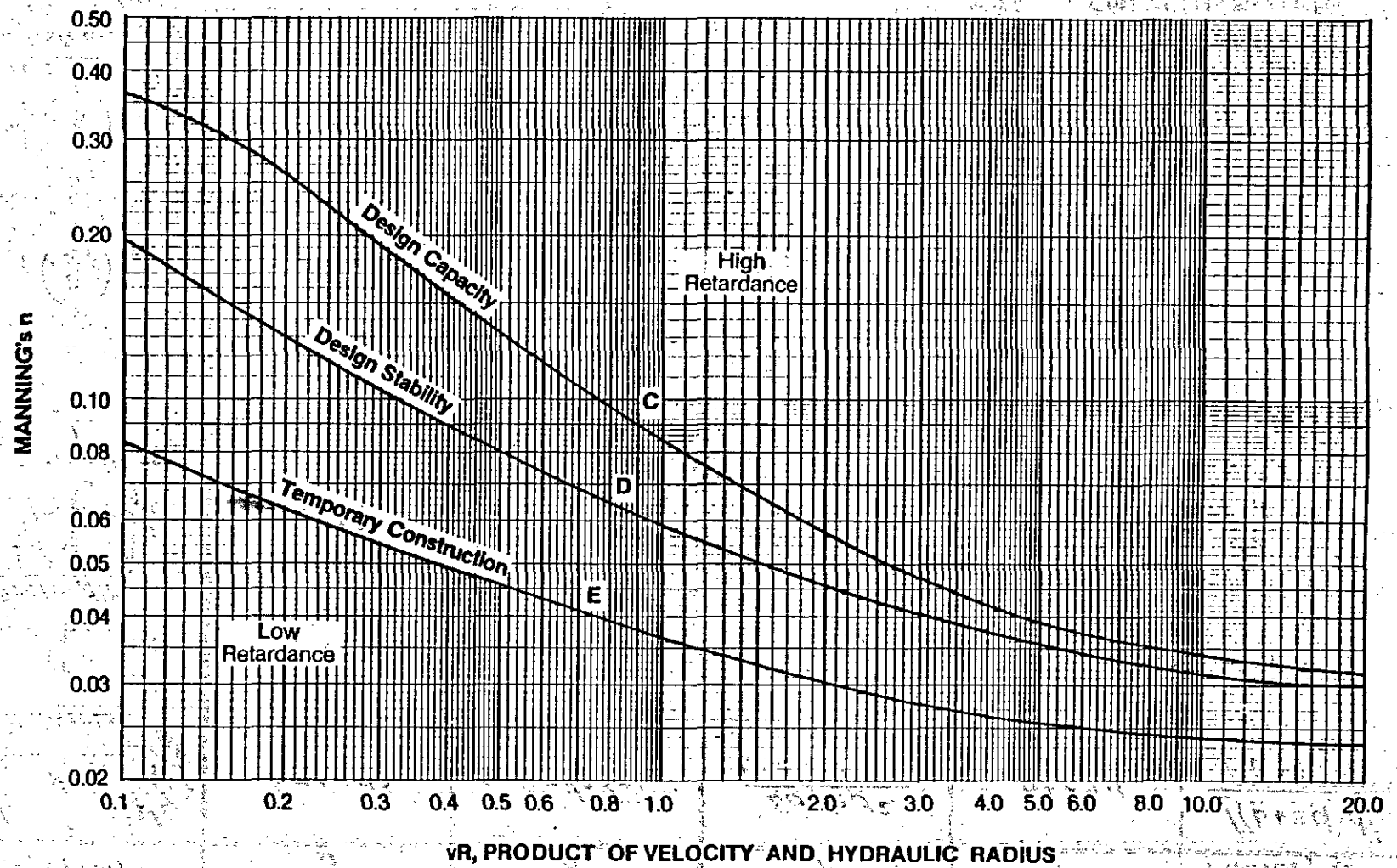
[a]See Figure 5-2 for channel sketches.

[b]Assumes uniform flow with the kinetic energy coefficient equal to 1.0.

[c]Reference: French (1985).

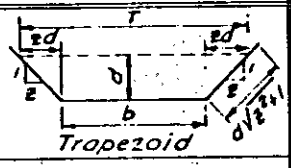
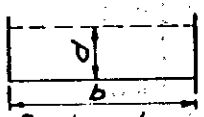
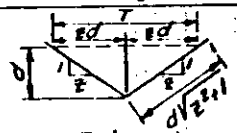
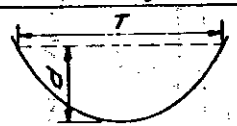
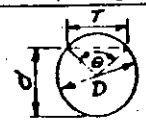
[d]Reference: USDOT, FHWA, HDS-4 (1965).

[e]Reference: Brater and King (1976).



Reference: USDA, TP-61 (1947).

FIGURE 5-1
Manning's n Values for Vegetated Channels

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

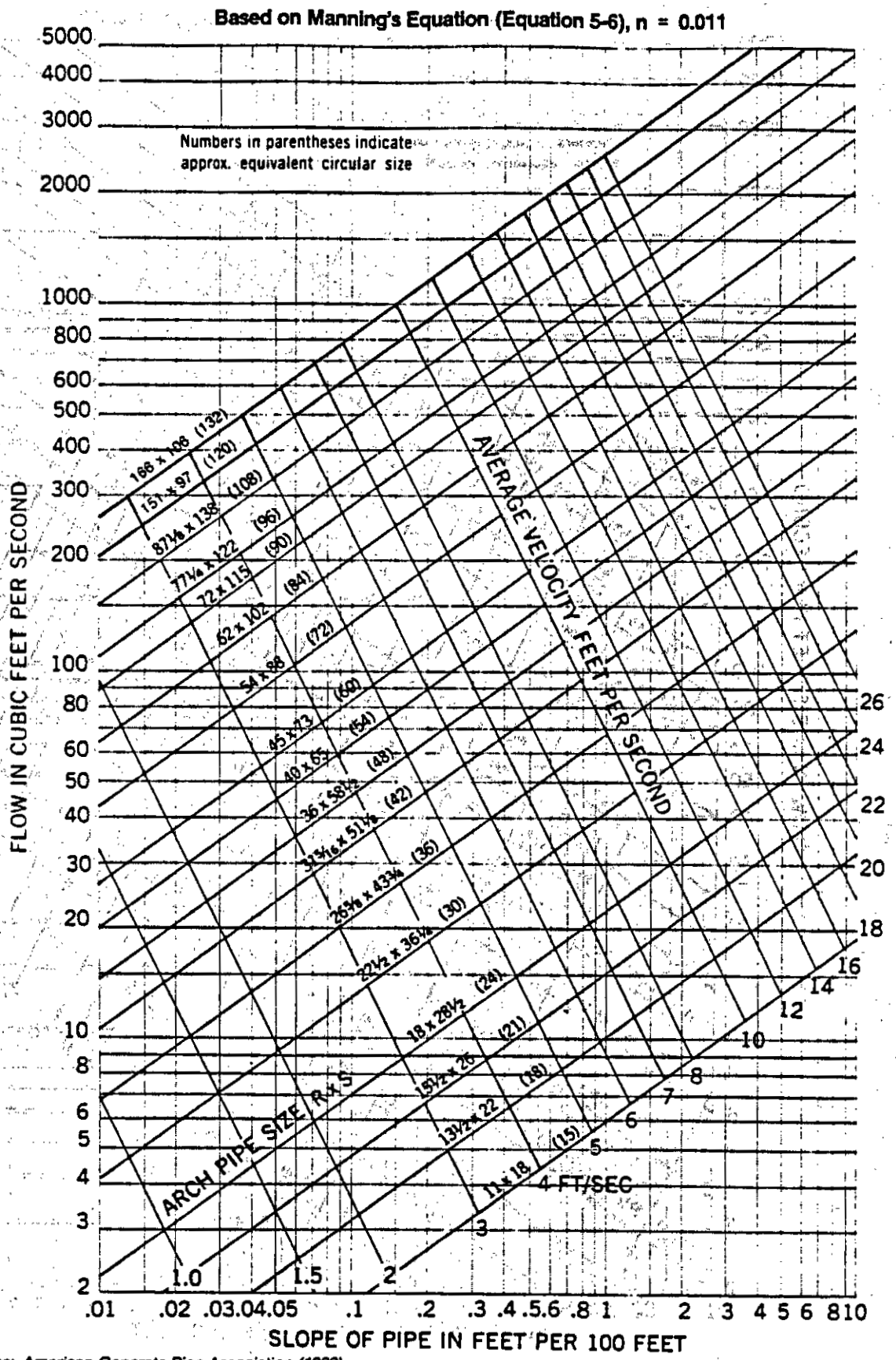
[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

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

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

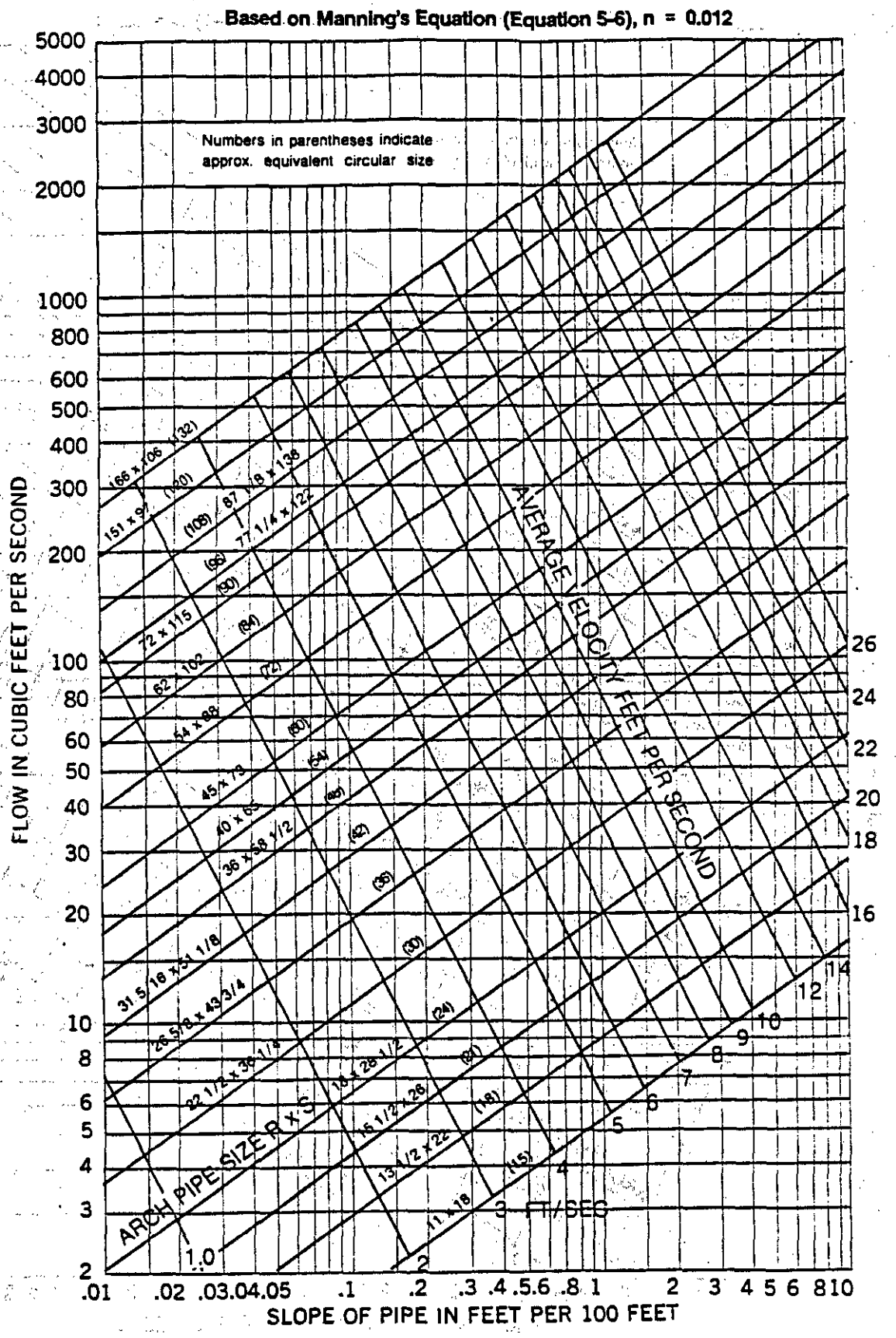
Reference: USDA, SCS, NEH-5 (1956).

FIGURE 5-2
Open Channel Geometric Relationships for Various Cross Sections



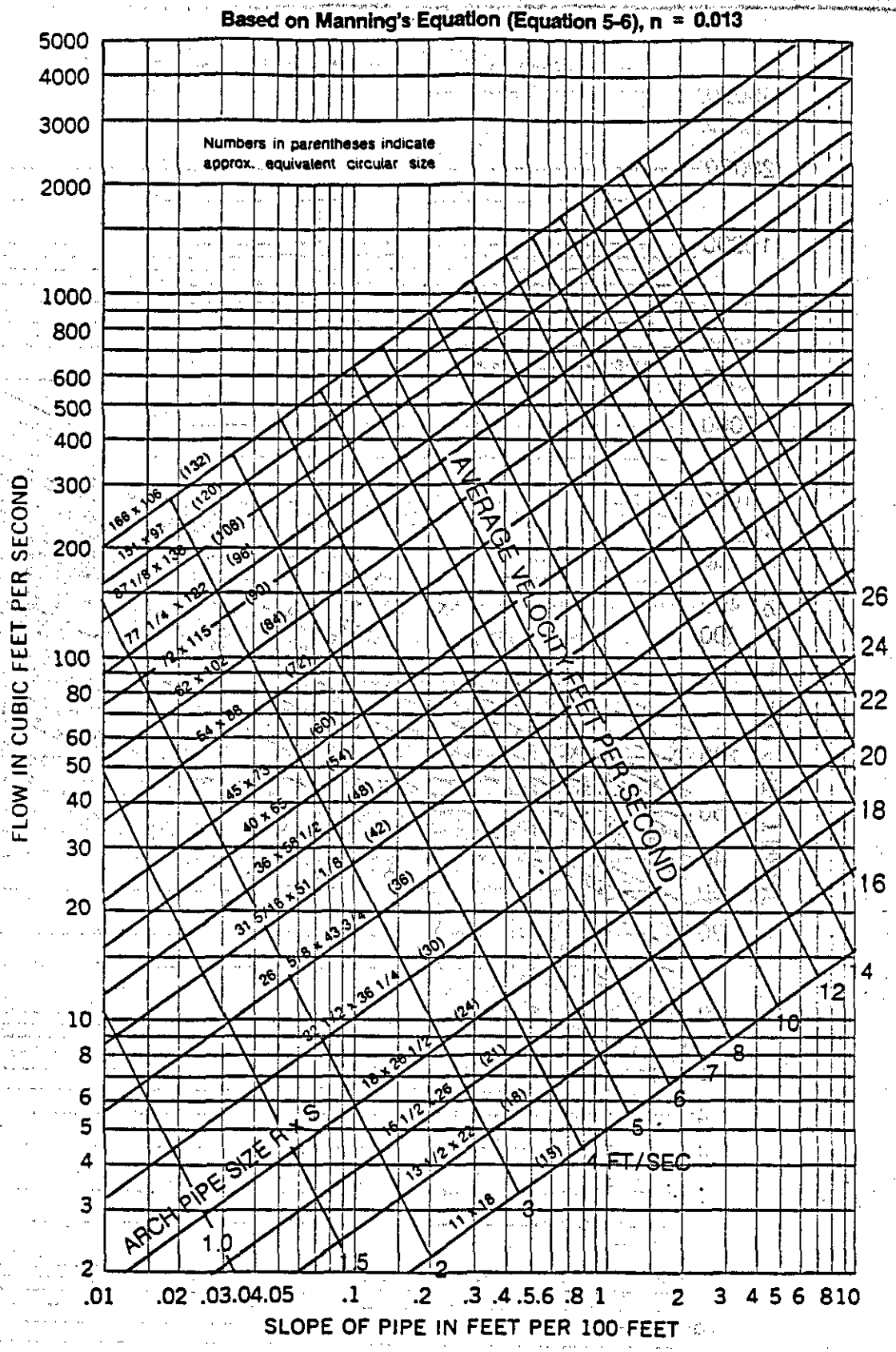
Reference: American Concrete Pipe Association (1980).

FIGURE 5-3
Capacity Chart for Pipe Arch Flowing Full, n = 0.011



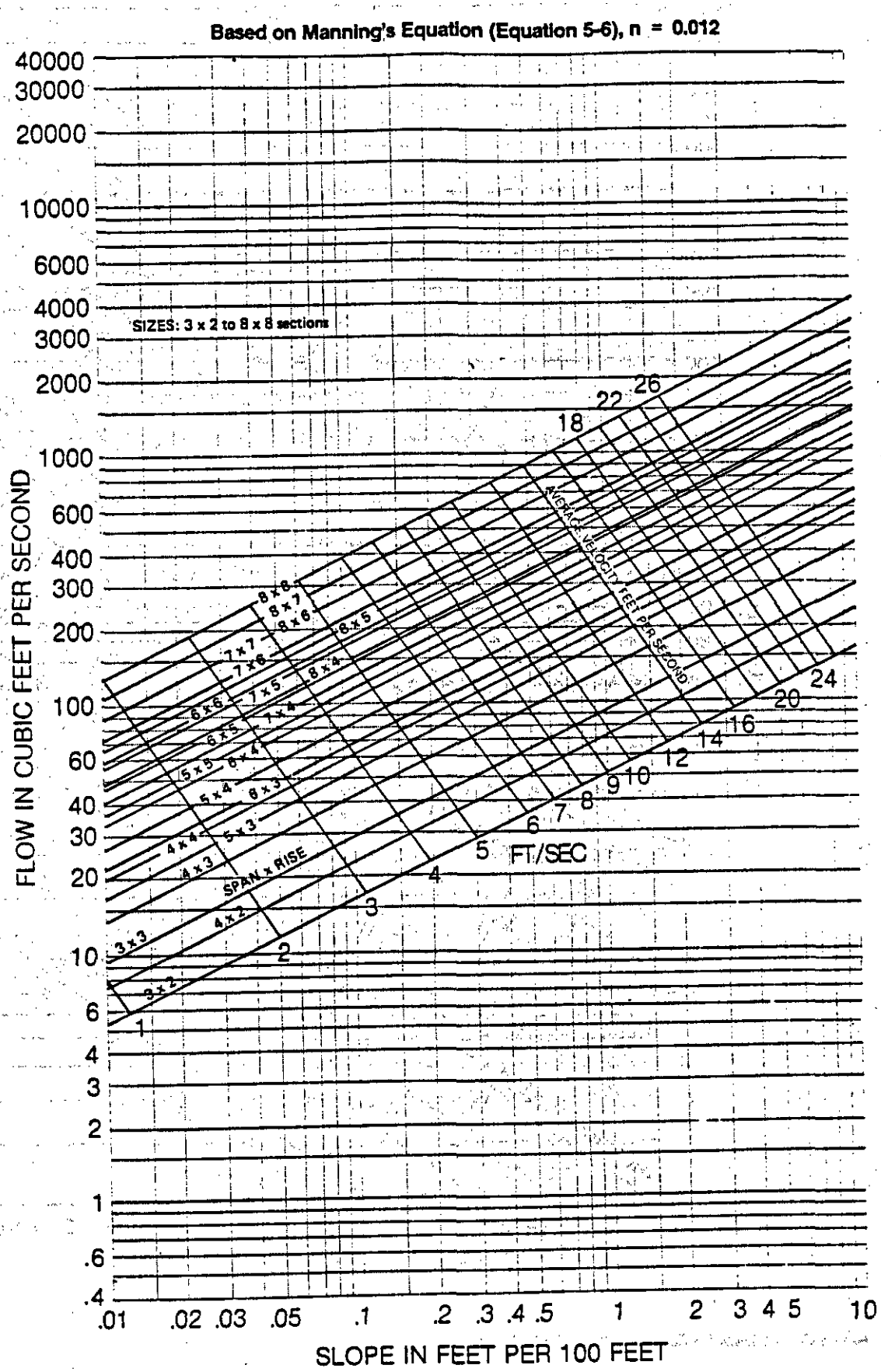
Reference: American Concrete Pipe Association (1980).

FIGURE 5-4
Capacity Chart for Pipe Arch Flowing Full, n = 0.012



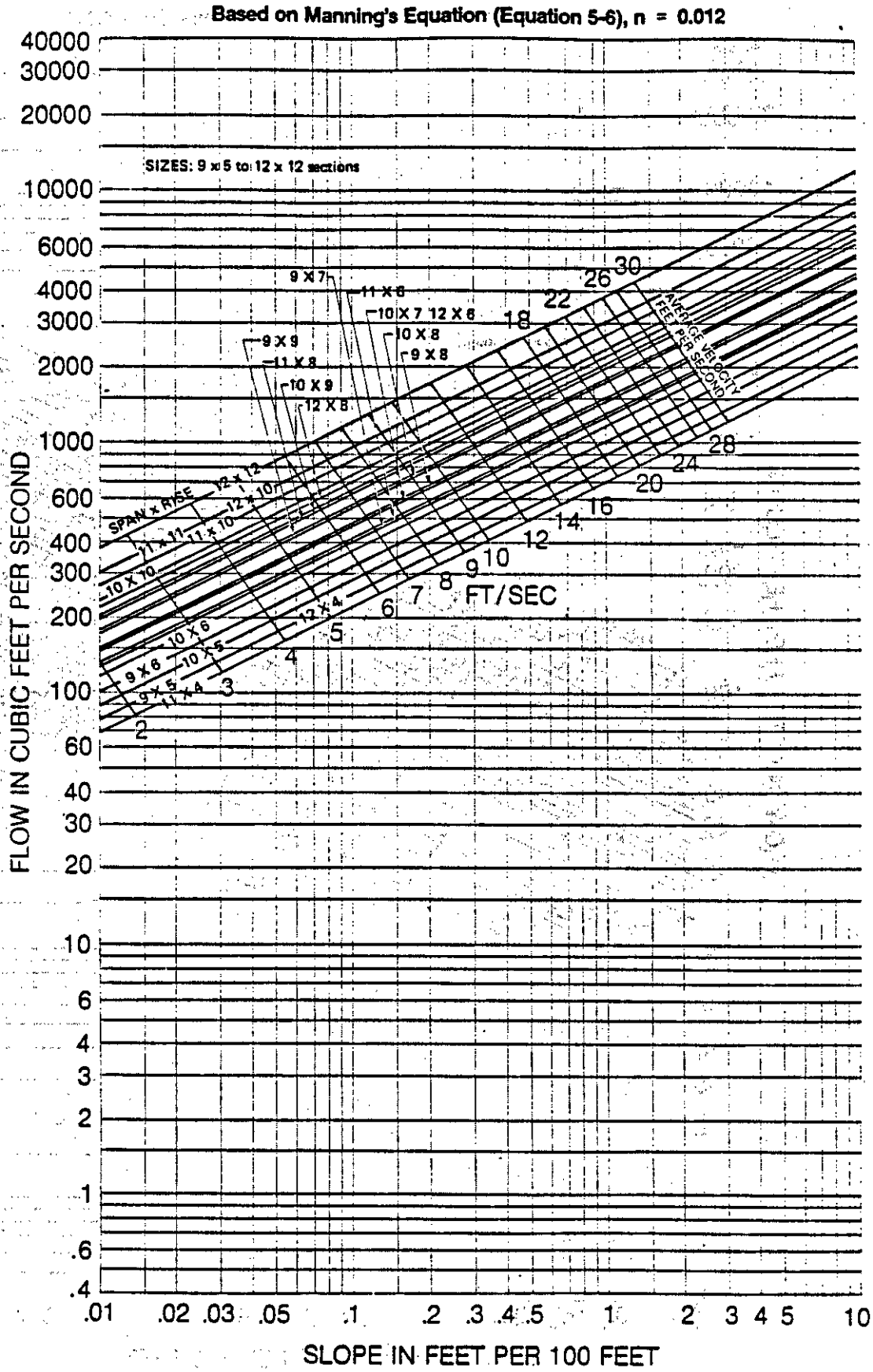
Reference: American Concrete Pipe Association (1980).

FIGURE 5-5
Capacity Chart for Pipe Arch Flowing Full, n = 0.013



Reference: American Concrete Pipe Association (1980).

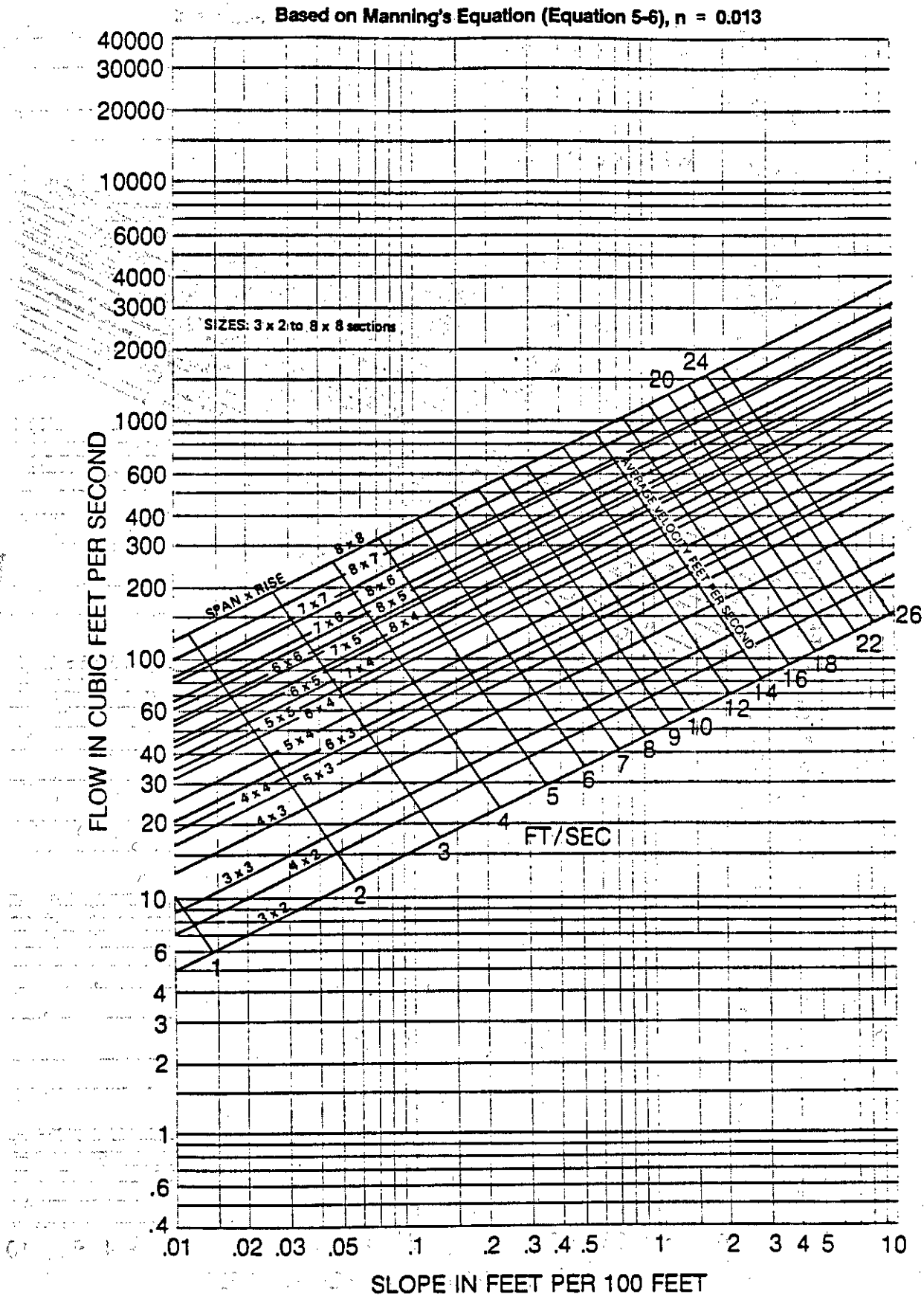
FIGURE 5-6
Capacity Chart for Box Sections Flowing Full, n = 0.012, Sizes 3′ x 2′ to 8′ x 8′



Reference: American Concrete Pipe Association (1980).

FIGURE 5-7

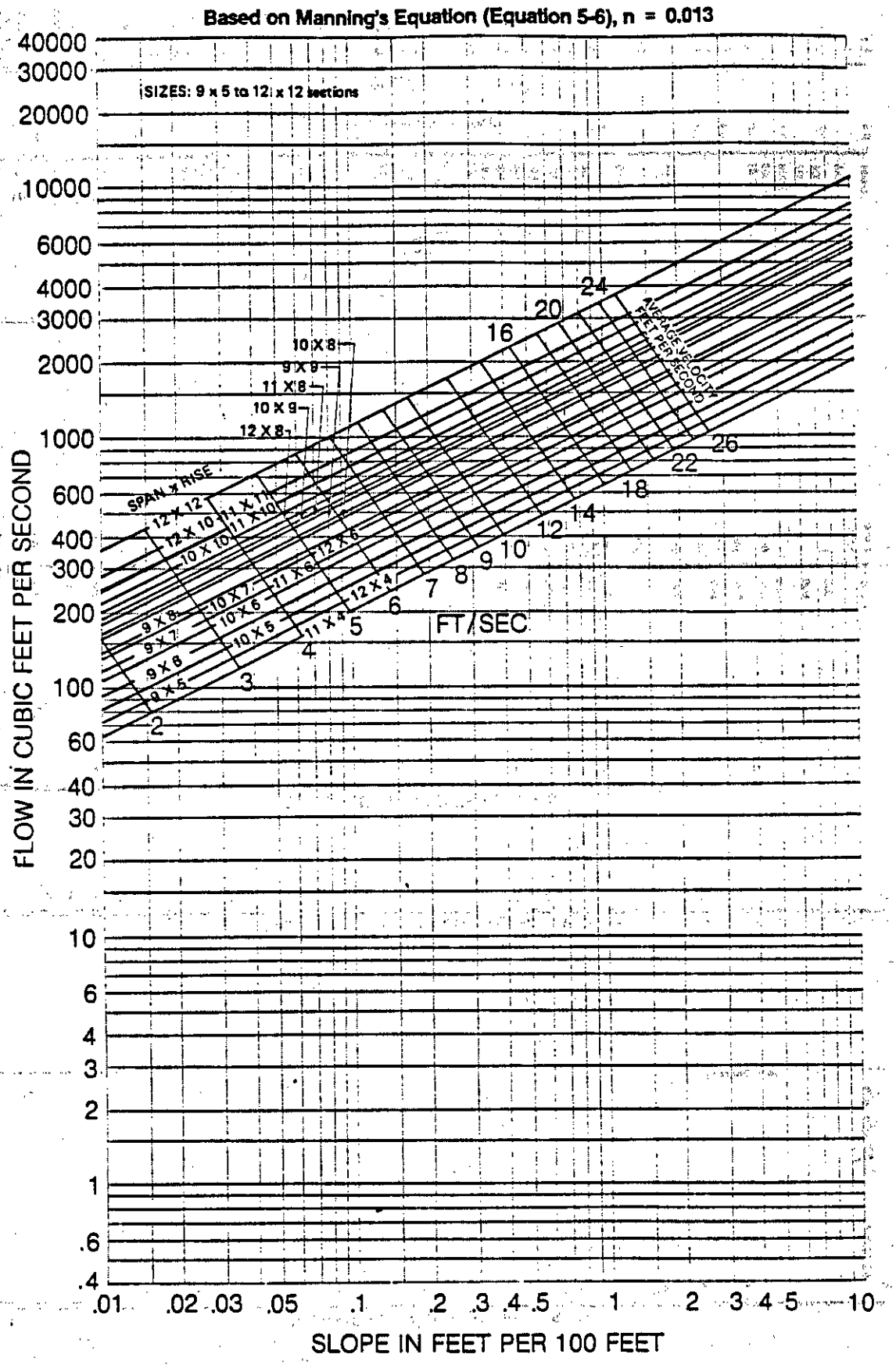
Capacity Chart for Box Sections Flowing Full, n = 0.012, Sizes 9′ x 5′ to 12′ x 12′



Reference: American Concrete Pipe Association (1980).

FIGURE 5-8

Capacity Chart for Box Sections Flowing Full, n = 0.013, Sizes 3′ x 2′ to 8′ x 8′

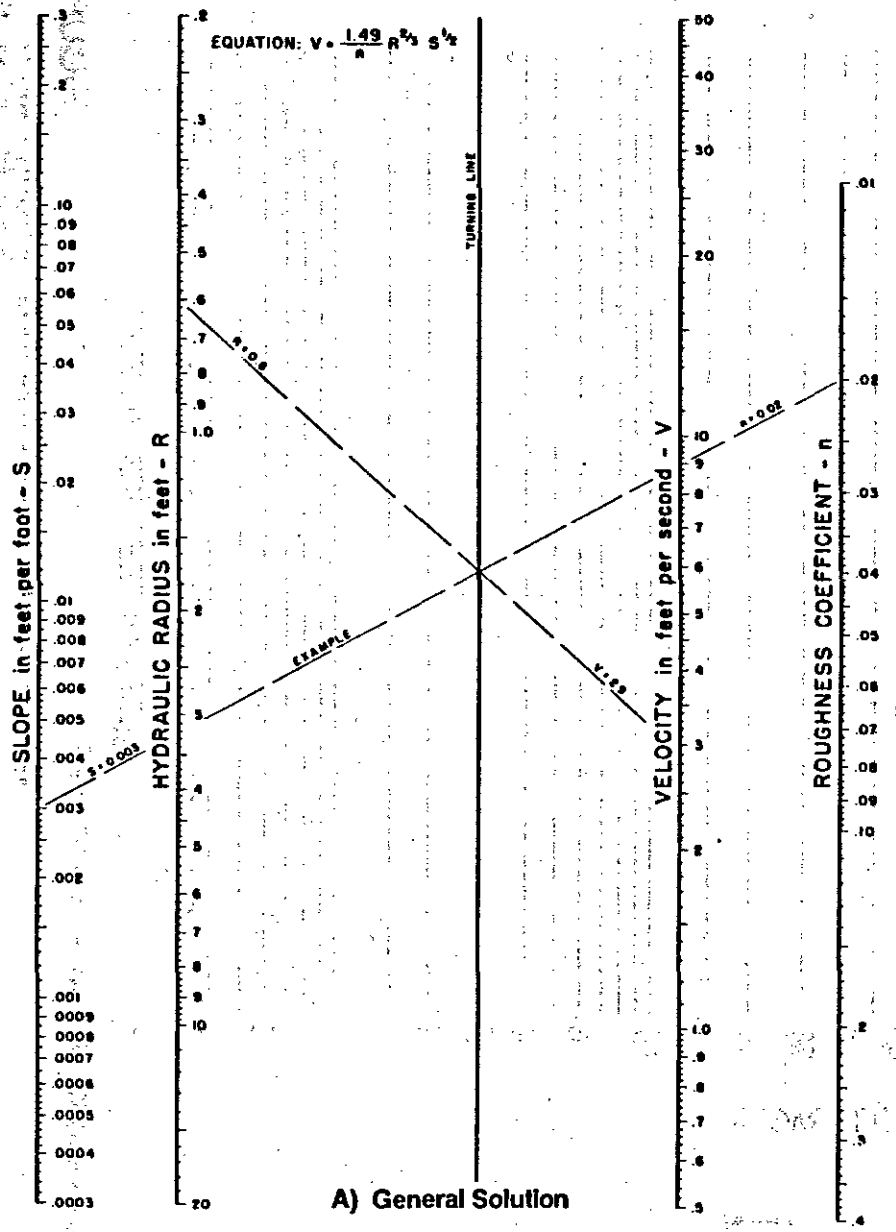


Reference: American Concrete Pipe Association (1980).

FIGURE 5-9

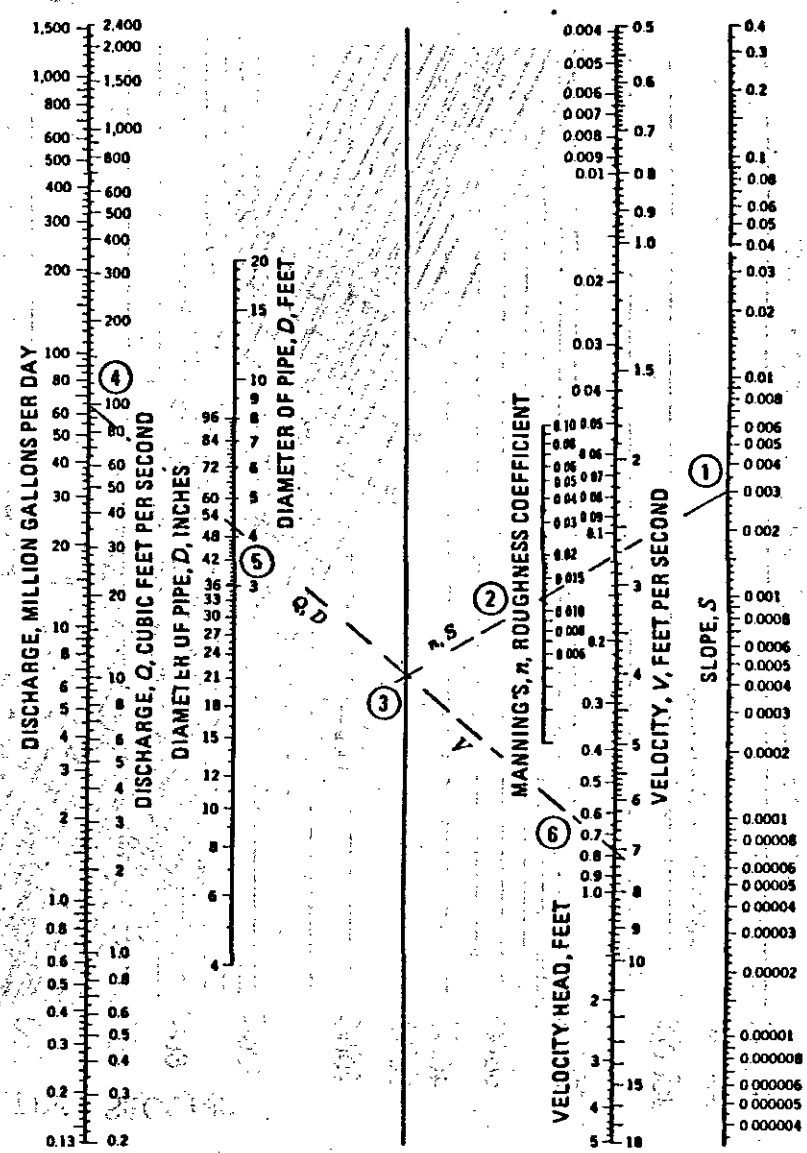
Capacity Chart for Box Sections Flowing Full, n = 0.013, Sizes 9′ x 5′ to 12′ x 12′

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



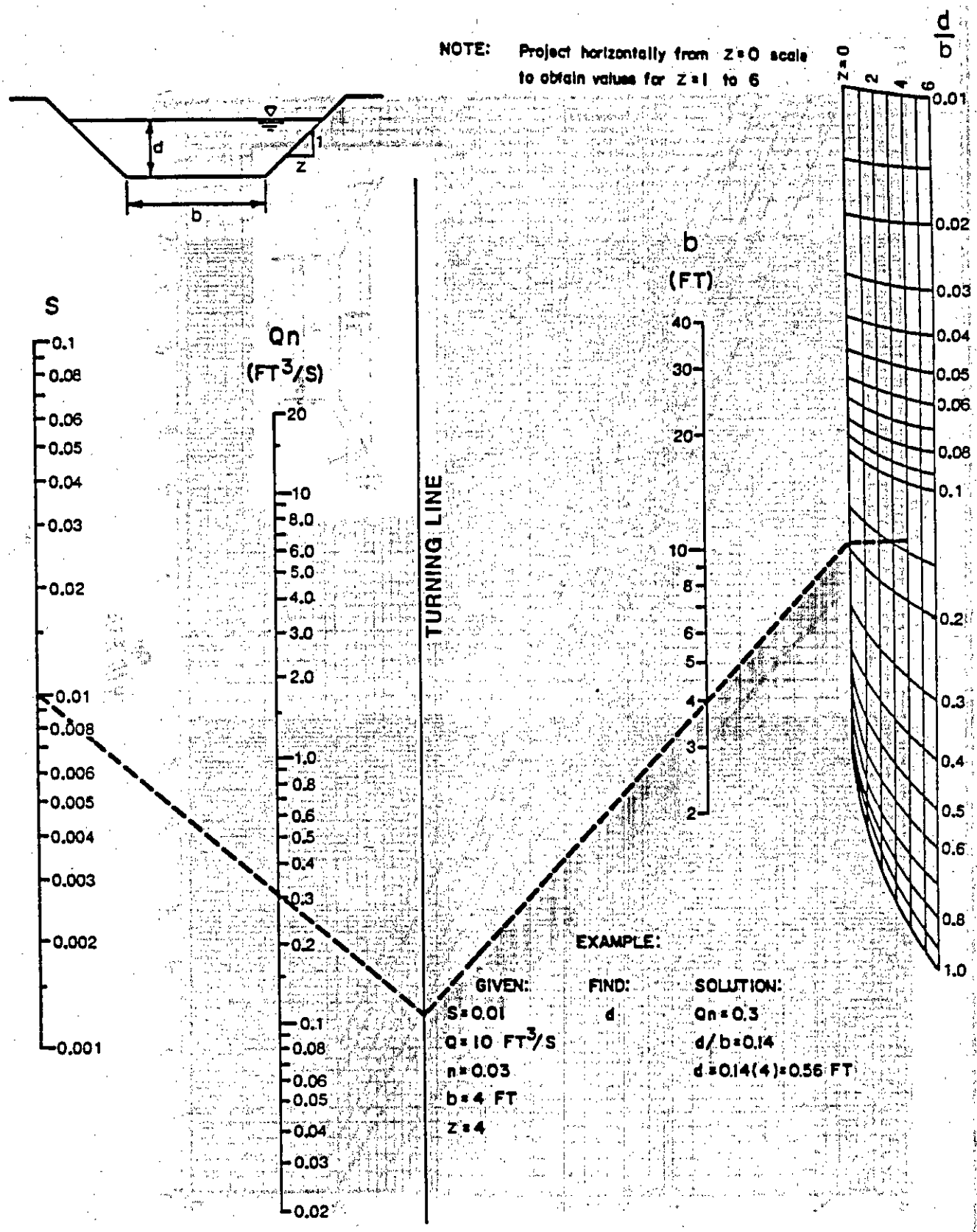
A) General Solution

Reference: American Concrete Pipe Association (1980).



B) Circular Pipe Solution

FIGURE 5-10
Nomographs for the Solution of Manning's Equation



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

FIGURE 5-11
Solution of Manning's Equation for Trapezoidal Channels

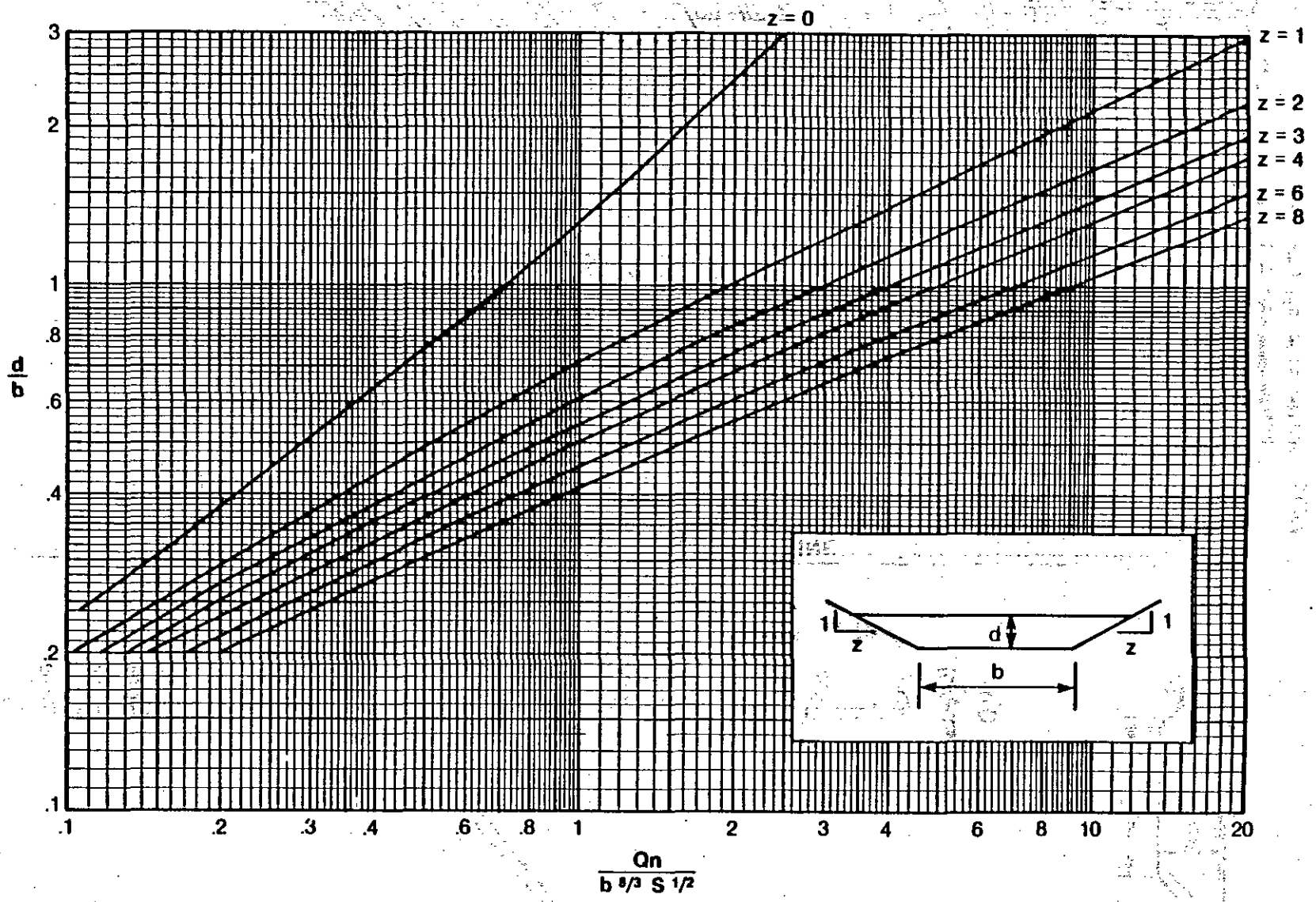


FIGURE 5-12
Trapezoidal Channel Capacity Chart

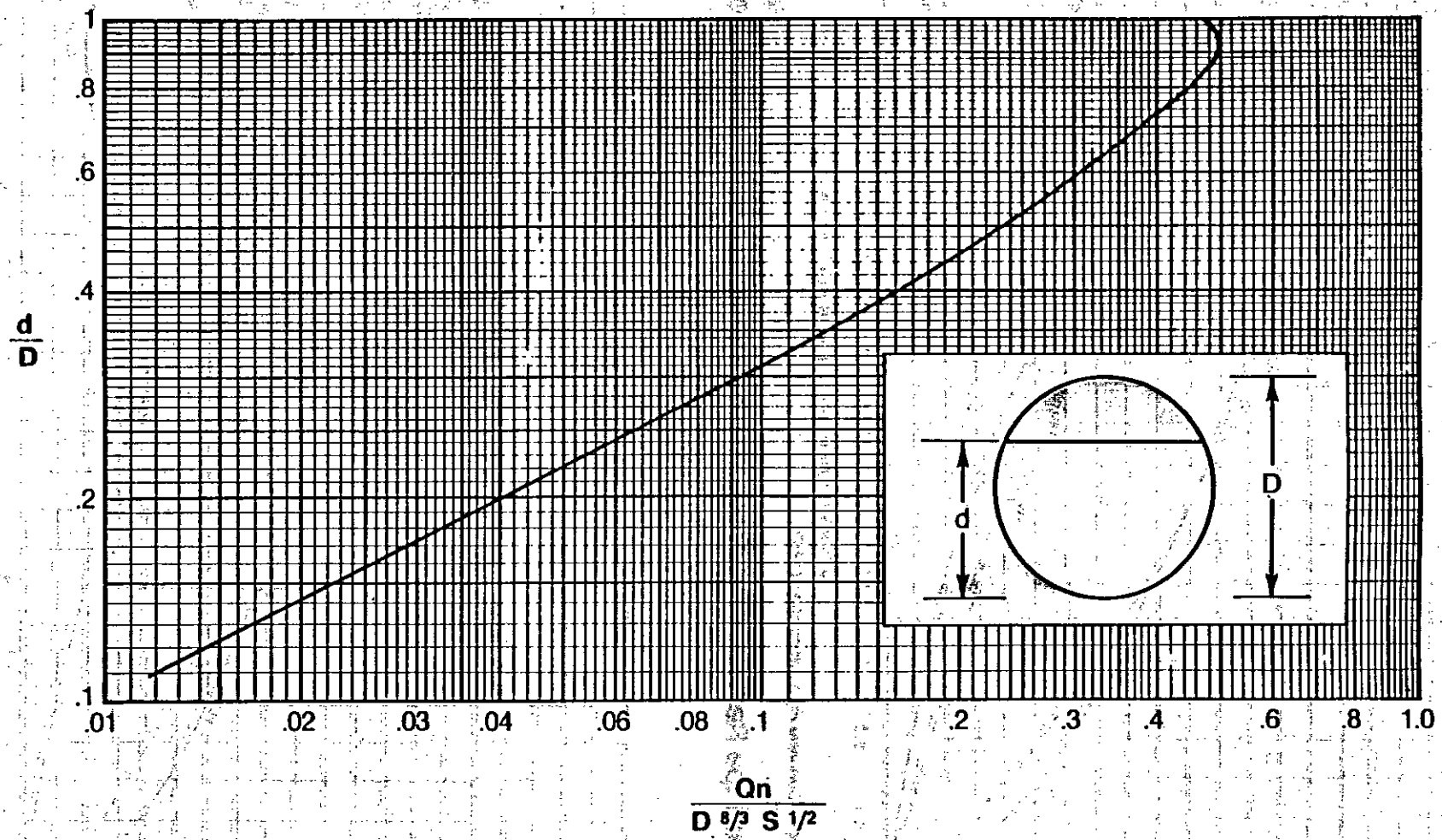
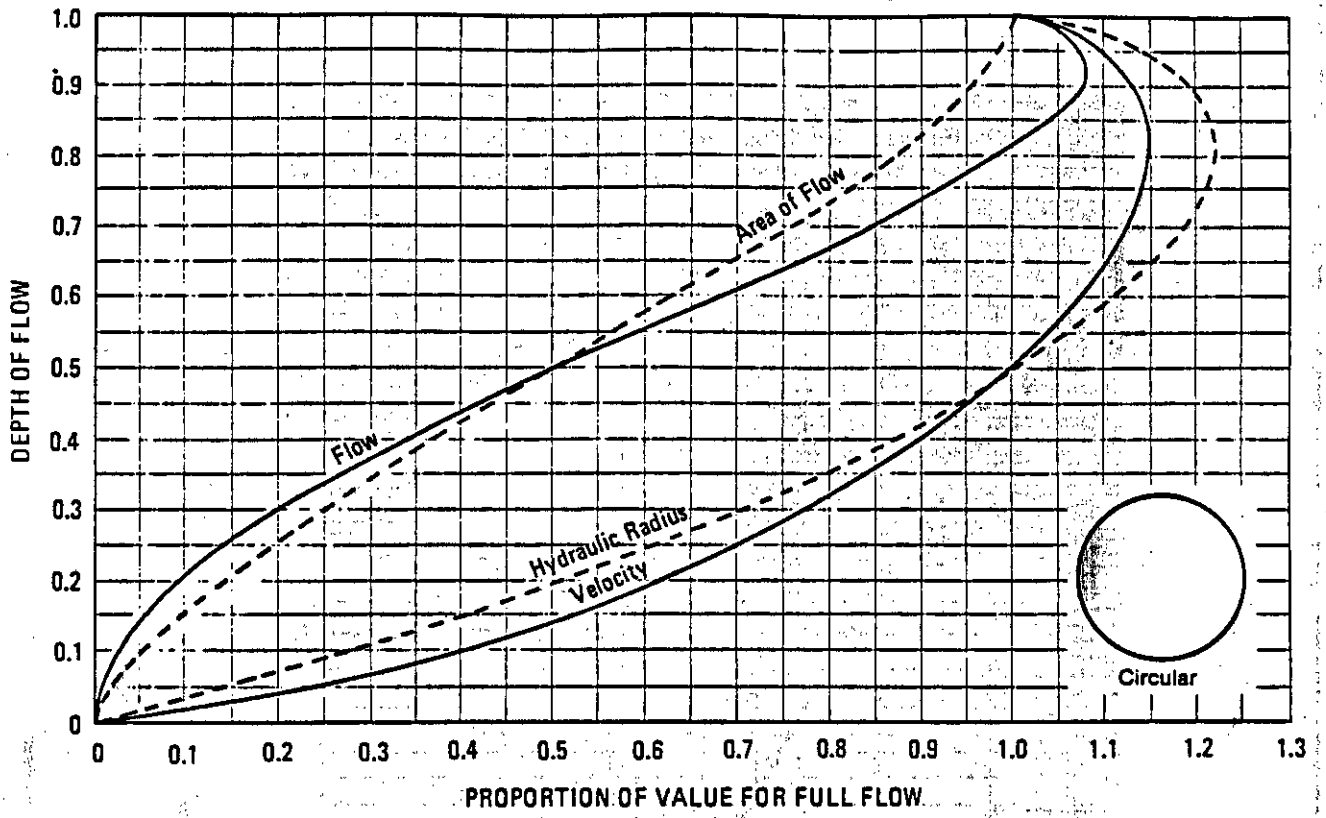


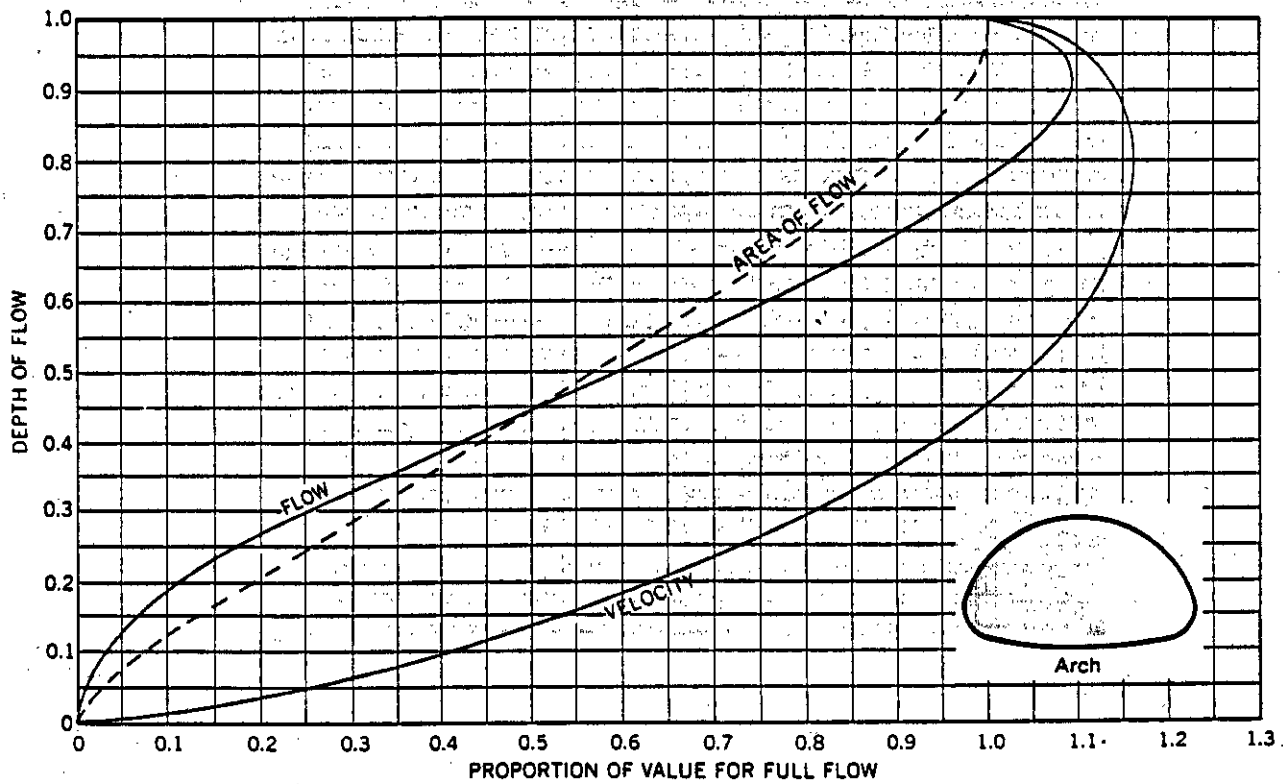
FIGURE 5-13
Circular Pipe Partial Flow Capacity Chart



Reference: American Concrete Pipe Association (1980).

FIGURE 5-14

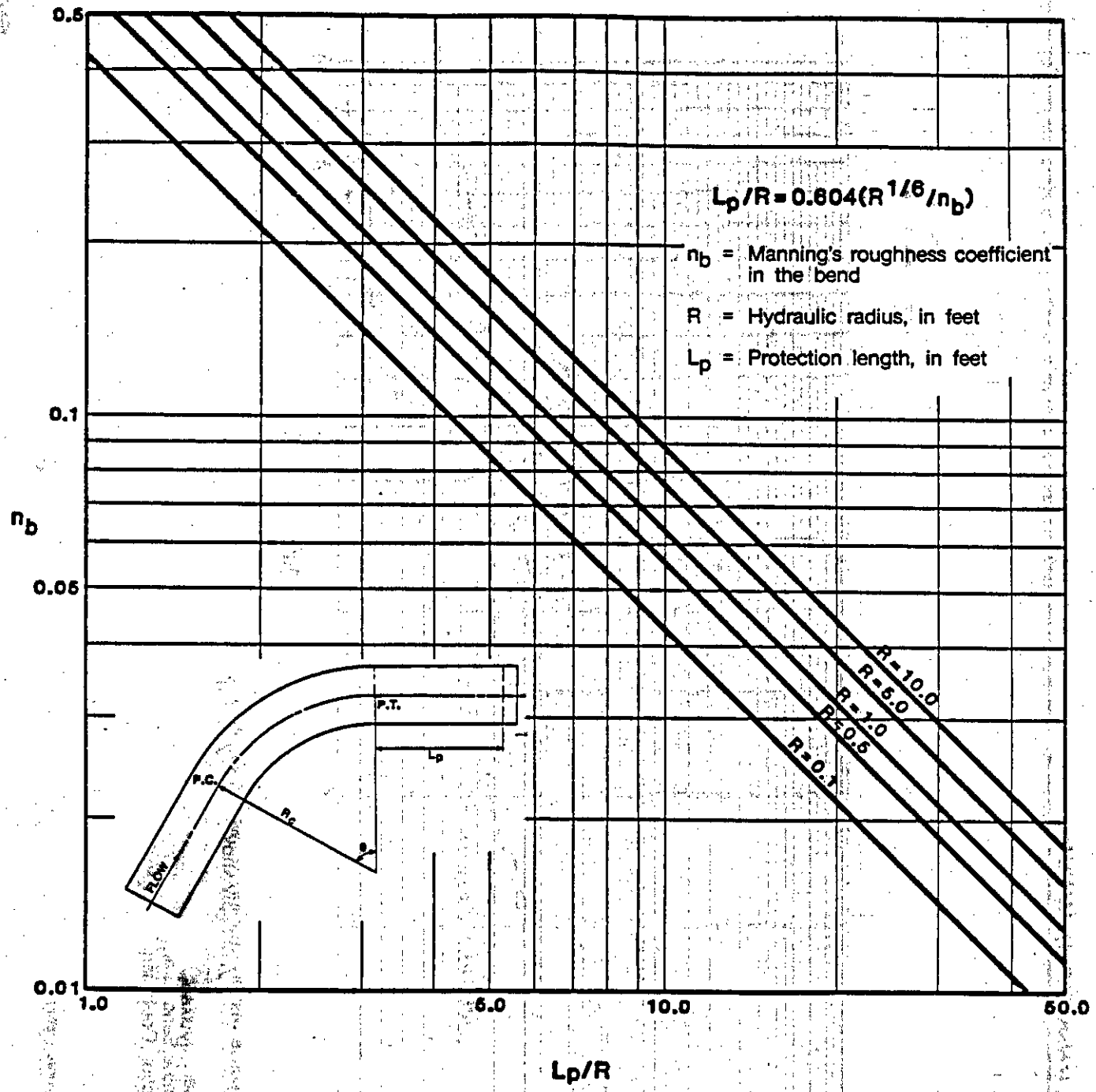
Circular Pipe Relative Flow, Area, Hydraulic Radius, and Velocity for Any Depth



Reference: American Concrete Pipe Association (1980).

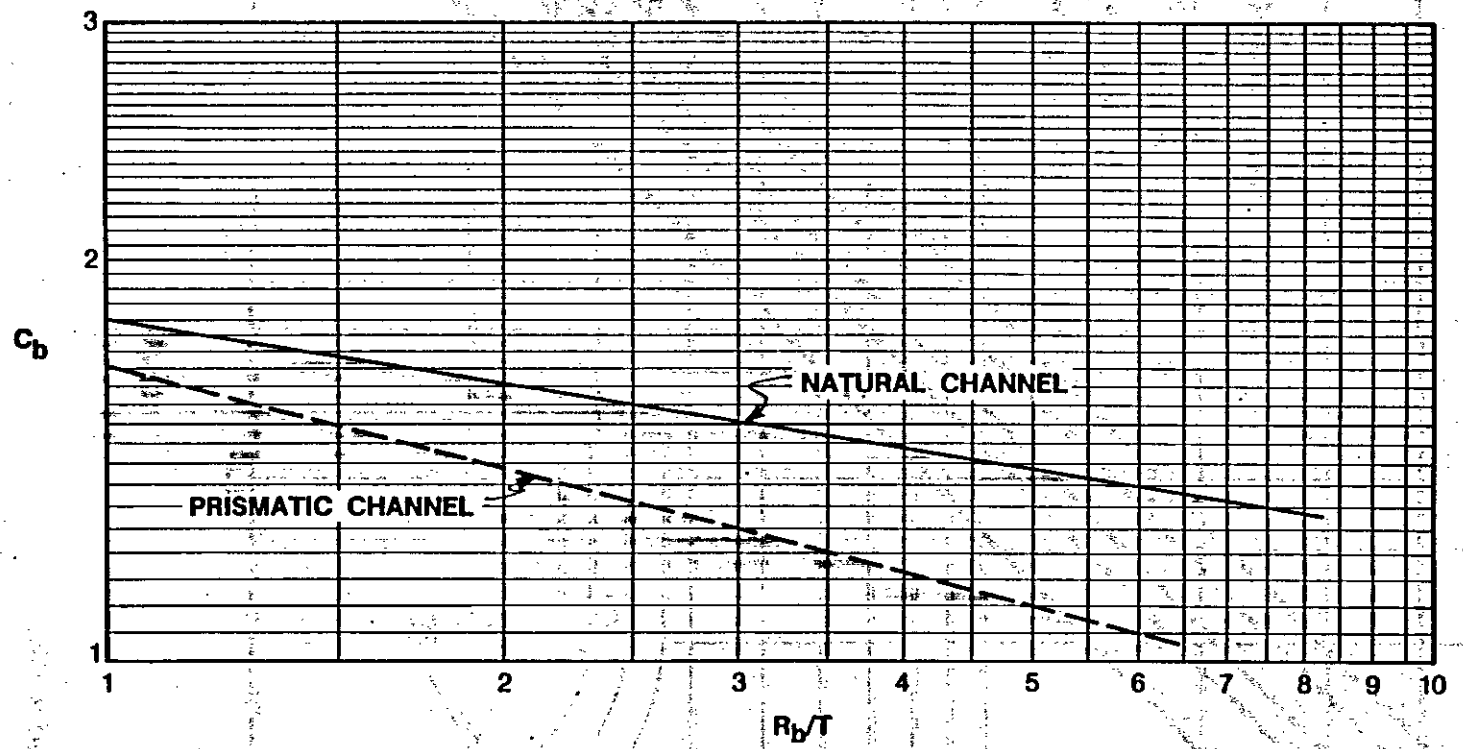
FIGURE 5-15

Pipe Arch Relative Flow, Area, and Velocity for Any Depth



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

FIGURE 5-16
Protection Length, L_p, Downstream of Channel Bend

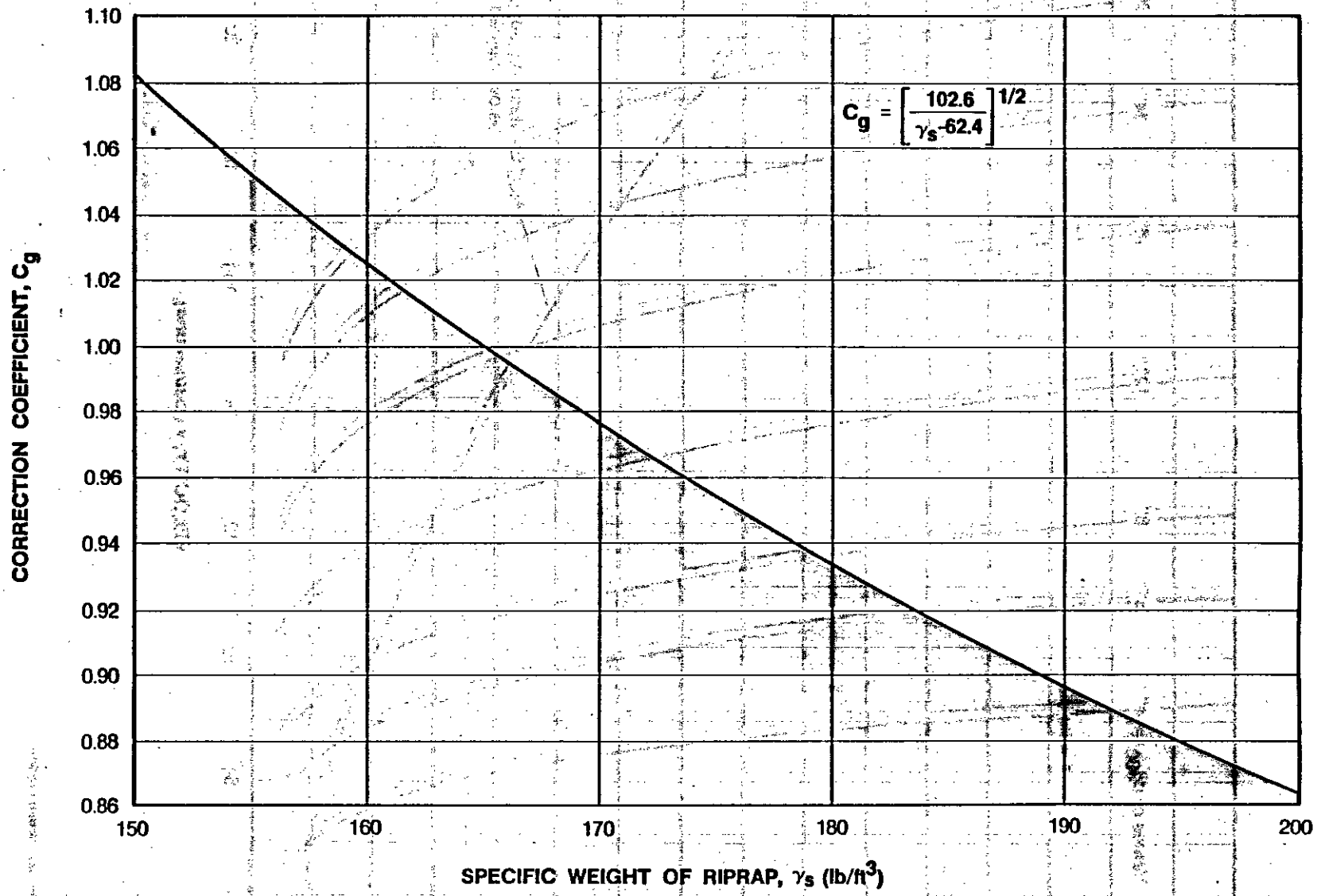


To obtain effective velocity, multiply known velocity by C_b.

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

Reference: Maynord (1987).

FIGURE 5-17
Riprap Lining Bend Correction Coefficient

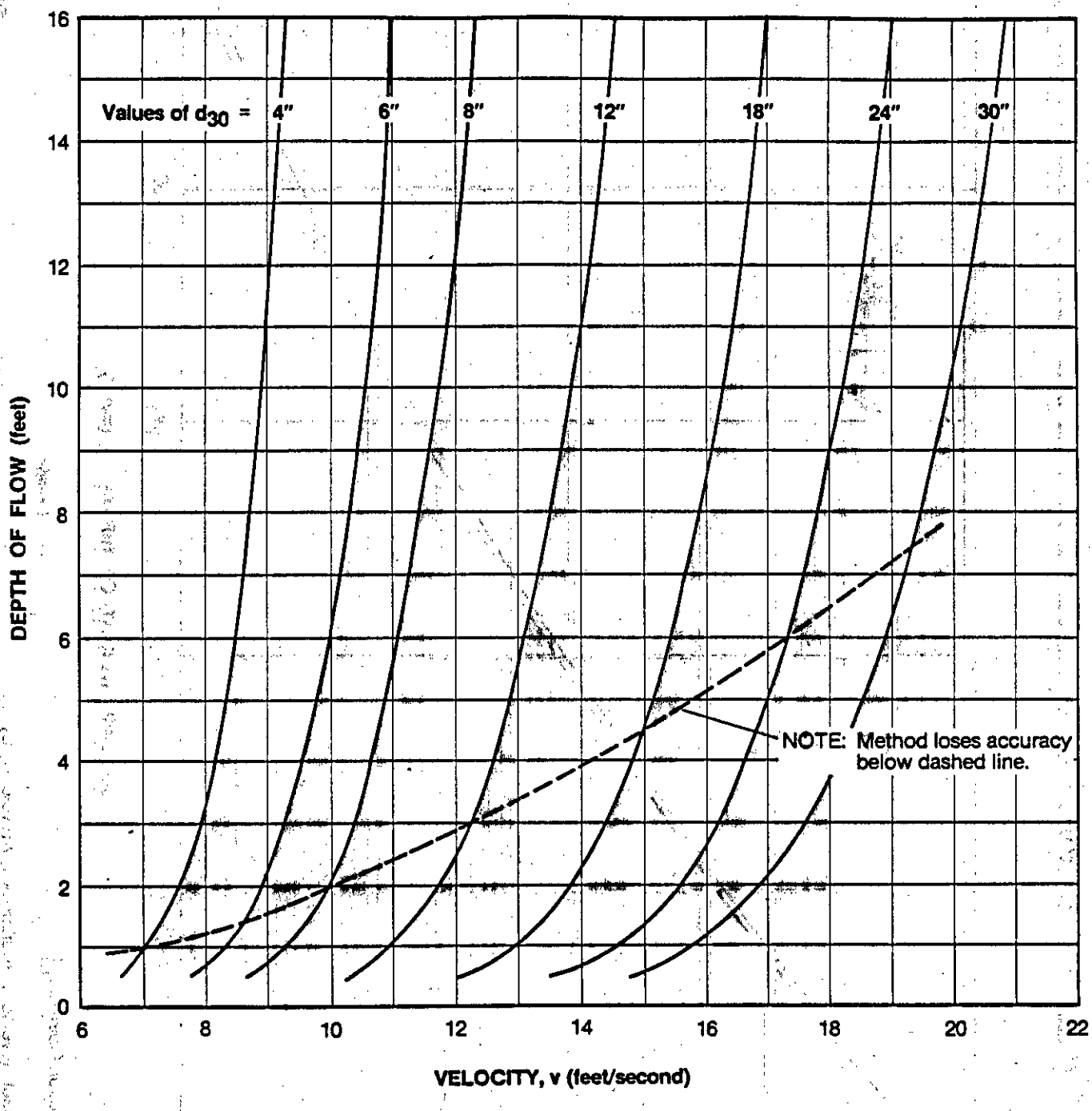


C_g = Correction Coefficient

To obtain effective velocity, multiply known velocity by C_g.

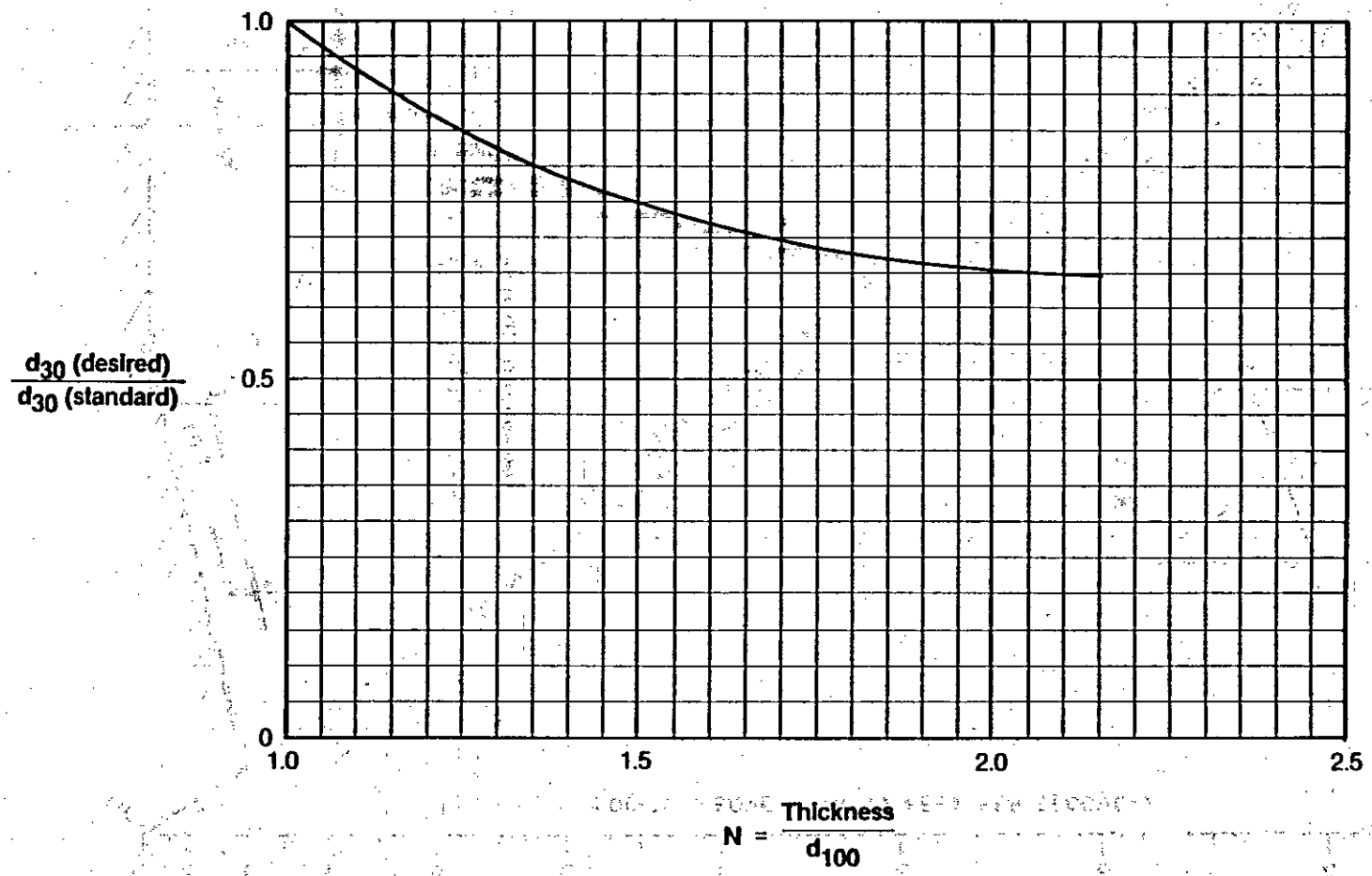
FIGURE 5-18

Riprap Lining Specific Weight Correction Coefficient



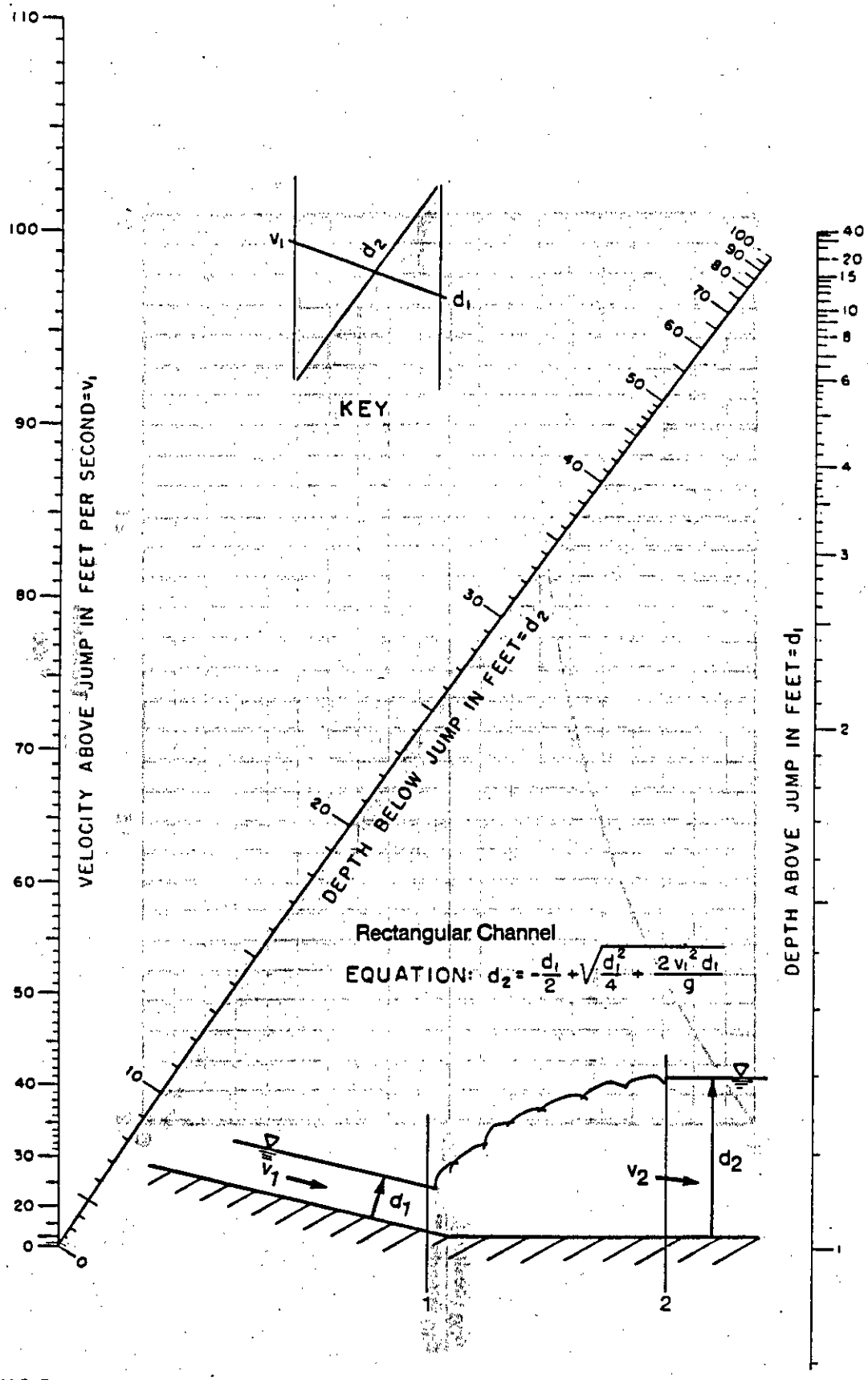
Reference: Reese (1988).

FIGURE 5-19
Riprap Lining d_{30} Stone Size as a Function of Mean Velocity and Depth



Reference: Maynord (1987).

FIGURE 5-20
Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 2.0 to 2.3



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

FIGURE 5-21
Nomograph for Solving the Rectangular Channel Hydraulic Jump Equation

CHAPTER 6
Culvert Hydraulics

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Chapter 6
CULVERT HYDRAULICS

6.1 FUNDAMENTALS

Culvert hydraulics can be clas-sified and analyzed on the basis of a control section. A control section is a location where a unique relationship exists between the rate of flow and depth of flow or water surface elevation. The two basic types of control sections are inlet and outlet control.

6.1.1 INLET CONTROL

Inlet control exists when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section for this condition is located just inside the entrance. Critical depth occurs at or near this location and the flow in the culvert is supercritical. The following variables influence culvert performance at the inlet:

1. Headwater elevation
2. Inlet area
3. Inlet edge configuration
4. Inlet shape

Flow under inlet control may be described mathematically by either the weir formula or the orifice formula, depending on the headwater depth. It is important to note that the tailwater elevation has no influence on capacity.

6.1.2 OUTLET CONTROL

Outlet control occurs when the culvert barrel or outlet has less capacity than the inlet. The control section for this situation is located at the barrel exit or downstream from the culvert. Either partially full subcritical flow or full pipe pressure flow conditions can occur. In addition to the variables influencing inlet performance listed above, the following factors can affect outlet performance:

1. Barrel roughness
2. Barrel area
3. Barrel shape

4. Barrel length
5. Barrel slope
6. Tailwater elevation

For outlet control, the difference between headwater and tailwater elevation represents the energy or head that conveys the flow through the culvert.

6.1.3 CULVERT SELECTION

In most situations, the hydraulic sizing of a culvert is a trial and error process. A trial culvert size is assumed and inlet and outlet performance are evaluated to determine if they will satisfy the condi-tions prevailing at the proposed location. A culvert system is selected by choosing the following items:

1. Inlet structure
2. Barrel material
3. Shape
4. Size
5. Outlet structure

The inlet and outlet structures are usually the same, to achieve a symmetrical installation. If the outlet velocity is high enough to cause erosion, protection or energy dissipation is required.

The procedures presented in this chapter were obtained from USDOT, FHWA, HDS-5 (1985).

6.2 DESIGN CRITERIA

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

1. Discharge
2. Headwater
3. Tailwater
4. Manning's n values
5. Length and slope
6. Velocity limitations

6.2.1 DISCHARGE

The design discharge depends on an appropriate storm frequency (return period) and the protection desired for the public on the roadway or adjacent lands. The following criteria shall apply:

Major and Minor Arterials	25-year
Major Collectors	25-year
Minor Collectors	10-year
Residential and Local	10-year
Roadside Ditches	10-year

In addition to the design flow, the culvert capac-ity should be checked for the 100-year return period, to ensure that overtopping flood conditions do not exceed 1 foot above the top of curb. If ponding occurs at the culvert entrance, and a reduc-tion in discharge attributable to storage is appropriate, reservoir routing calculations can be used to estimate the reduction.

6.2.2 HEADWATER

The allowable headwater elevation is determined from an evaluation of conditions upstream of the culvert and the proposed or existing roadway elevation. The following criteria shall be analyzed:

1. Non-damaging or permissible upstream flooding elevations (e.g., existing buildings or flood elevation data from Section 3.3) should be identified. Headwater should be kept below these elevations.

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.

3. Headwater depth for the design discharge should not cause water to rise above the top of approach channels adjacent to improved land or above the established flood plain (Q_{100}). Impacts to the established base flood (Q_{100}) elevation should be minimal.

4. Other site-specific design considerations should be addressed as required.

The constraint that gives the lowest allowable headwater elevation shall establish the basis for hydraulic calculations.

6.2.3 TAILWATER

The hydraulic conditions downstream of the culvert site shall be evaluated to determine a tailwater depth for the design discharge. The following conditions are typical:

1. 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 presented in Section 6.3.

2. For culverts that dis-charge to an open channel, tailwater depth is established by evaluating the normal depth of flow in the outlet channel using procedures identified in Chapter 5.

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.

4. 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.

6.2.4 MANNING'S n VALUES

All culverts shall be constructed using reinforced concrete pipe unless alternate materials of equal or greater effectiveness and strength are approved by the Engineer of Public Works. Manning's n values for culvert capacity calculations shall be as follows:

Material	n Value
Circular and Arch Concrete Pipe	0.012
Circular Corrugated Metal Pipe (CMP)	0.024
Structural Plate CMP	0.0328-0.0302
Box Concrete	0.012
Oval Concrete	0.012
Arch CMP	0.024
Structural Plate Arch	0.0327-0.0306

6.2.5 LENGTH AND SLOPE

The length and slope of a culvert should consider the following factors:

1. Channel bottom of the stream being conveyed
2. Geometry of the roadway embankment
3. Skew angle of the culvert

In general, the culvert slope should be chosen to approximate existing topography.

6.2.6 VELOCITY LIMITATIONS

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 is required.

Riprap is required at all culvert outlets, unless some other material of equal or greater effectiveness and appearance is approved by the Engineer of Public Works. The maximum velocity shall 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 5), energy dissipation is required.

6.3 DESIGN CALCULATIONS

A flow chart for performing culvert design calculations is provided in Figure 6-1 (see discussion in Section 6.3.1). A

worksheet for performing calculations for standard culvert design is provided in Figure 6-2. Additional worksheets for tapered and mitered inlet design are provided in HDS-5 (USDOT, FHWA, 1985).

For standard culvert design, the following inlet and outlet control charts, found in HDS-5 and duplicated as the figures shown, are required culvert capacity calculations:

Standard Culvert Type	Figure Numbers Inlet Control	Figure Numbers Outlet Control
Circular Concrete Pipe	6-3	6-4
Circular Corrugated Metal Pipe (CMP)	6-5	6-6
Structural Plate CMP	6-5	6-7
Concrete Box	6-8	6-9
Oval Concrete Pipe--Long Axis Horizontal	6-10	6-11
Oval Concrete Pipe--Long Axis Vertical	6-12	6-11
CMP Arch	6-13	6-14
Structural Plate CMP Arch (18-inch Corner Radius)	6-13	6-15
Circular Pipe With Beveled Ring	6-16	6-4 or 6-6
Concrete Pipe Arch	6-17	6-18

6.3.1 GENERAL PROCEDURE

The following procedure, illustrated in Figure 6-1, shall be used to select a culvert size with the charts from HDS-5:

1. Perform hydrologic calculations (see Chapter 4).

2. List the following design data (a suggested tabulation form is provided in Figure 6-2, which includes a drawing labeled with design variables):

   a. Design discharge, Q, in cfs, with average return period (e.g., Q_{25}). When more than

one barrel is used, show Q divided by the number of barrels.

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 (see Section 6.2.2).

e. Mean and maximum flood velocities in natural stream (optional).

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:

$$A = \frac{Q}{v} \qquad (6\text{-}1)$$

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 appropriate figures in this chapter). A trial size is determined by assuming HW/D, e.g., HW/D = 1.5, and using the given Q.

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

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

   a. For inlet control, perform the following calculations (see Section 6.3.2 for additional details):

      (1) Use an appropriate inlet control chart from this chapter or HDS-5 and the trial size from Step 3 to find HW. Tailwater, TW, conditions are neglected in this determination. HW is found by multiplying HW/D, obtained from the nomographs, by the height of culvert D.

      (2) If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control, before computing HW for outlet control.

   b. For outlet control, perform the following calculations (see Section 6.3.3 for additional details):

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

      (2) If the TW elevation determined above is equal to or greater than the top of the culvert at the outlet, set design tailwater, DTW, equal to TW and find HW, using the following equation and the

appropriate outlet control nomograph from this chapter or HDS-5:

$$HW = H + DTW - LS_o \qquad (6\text{-}2)$$

where:

HW = Headwater depth for outlet control, in feet

H = Total head loss, obtained from the appropriate outlet control nomograph from this chapter or HDS-5, in feet

DTW = Design tailwater, in feet

L = Barrel length, in feet

S_o = Barrel slope, in feet/foot

(3) If the TW elevation determined above is less than the top of the culvert at the outlet, find HW using Equation 6-2, except that DTW is the greater of the following two parameters:

$$h_o = \frac{d_c + D}{2} \qquad (6\text{-}3)$$

or

TW

where:

h_o = Equivalent hydraulic elevation at outlet, in feet

d_c = Critical depth, in feet (from HDS-5 charts, duplicated as figures

listed below). Note: d_c cannot exceed D.

D = Height of culvert opening, in feet

TW = Downstream tailwater elevation, in feet

Culvert Type	Figure
Rectangular	6-19
Circular	6-20
Oval--Long Axis Vertical	6-21
Oval--Long Axis Horizontal	6-22
Standard CMP Arch	6-23
Structural Plate CMP Arch	6-24
Concrete Pipe Arch	6-25
	6-26

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

$$HW \leq D + (1 + k_e) \frac{v^2}{2g} \qquad (6\text{-}4)$$

where:

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. Compare the headwater values from Step 4a (inlet control) and Step 4b (outlet control). The higher headwater governs and indicates the type of flow

control existing under the given conditions for the trial size and inlet configuration selected.

6. If outlet control governs and the HW is higher than the acceptable AHW, select a larger trial size and find HW as instructed under Step 4b. (Inlet control does not need to be checked, because 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 and HW by the above procedure.

8. If the culvert operates under inlet control, design a tapered inlet following procedures in HDS-5, if an improved inlet is desirable.

9. If roadway overtopping occurs, calculate capacity following procedures in Section 6.3.4.

10. If storage routing is considered important, follow procedures in Chapter 9.

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 6-1. If d_c or TW is less than the height of the culvert barrel, use the cross-sectional area corresponding to d_c or TW depth, whichever gives the greater area of flow. The total cross-sectional area, A, of the culvert barrel should not be exceeded.

    b. If inlet control governs in Step 5, the outlet velocity can be assumed to equal the mean velocity for open channel flow conditions in the barrel, computed by Manning's Equation (see Chapter 5) for the rate of flow, barrel size, roughness, and slope of culvert selected.

12. Determine to what extent channel protection is required downstream of the outlet (see Chapter 5). Properly sized riprap is required as a minimum.

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

6.3.2 INLET CONTROL

Inlet control charts are presented in this manual for the following standard culvert types:

Standard Culvert Type	Figure Numbers for Inlet Control Charts
Circular Concrete Pipe	6-3
Circular CMP	6-5
Concrete Box	6-8
Oval Concrete Pipe--Long Axis Horizontal	6-10
Oval Concrete Pipe--Long Axis Vertical	6-12
CMP Arch	6-13
Structural Plate CMP Arch (18-inch Corner Radius)	6-13
Circular Pipe with Beveled Ring	6-16
Concrete Pipe Arch	6-17

Metal pipes are not allowed unless approved by the Engineer of Public Works.

The following three types of calculations can be performed using the inlet control charts:

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/D scale marked (1).

   b. If the HW/D scale marked (1) represents the inlet configuration used, read HW/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 1b horizontally to scale (2) or (3) and read HW/D.

c. Compute HW by multiplying HW/D by D.

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

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

a. Compute HW/D for given conditions.

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

c. Use a straightedge to connect the point on HW/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/D.

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

c. Use a straightedge to connect the point on HW/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.

6.3.3 OUTLET CONTROL

Outlet control charts are presented in this manual for the following standard culvert types:

Standard Culvert Type	Figure Numbers for Outlet Control Charts
Circular Concrete Pipe	6-4
Circular CMP	6-6
Structural Plate CMP	6-7
Concrete Box	6-9
Oval Concrete Pipe--Long Axis Horizontal or Vertical	6-11
CMP Arch	6-14
Structural Plate CMP Arch (18-inch Corner Radius)	6-15
Concrete Pipe Arch	6-18

All culverts shall be constructed using reinforced concrete pipe unless alternate materials are approved by the Engineer of Public Works.

The following steps outline the use of the outlet control charts:

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

   a. Locate the appropriate nomograph for the type of culvert selected. Find the entrance loss coefficient, k_e, for the inlet configuration using data from Table 6-1.

   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 values.

(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:

$$L_1 = L \left(\frac{n_1}{n}\right)^2 \qquad (6\text{-}5)$$

(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. Read head in feet on the H scale located on the far right. For values beyond the limit of the printed scales, find H by solving the equation:

$$H = \left[1 + k_e + \frac{29n^2L}{R^{1.33}}\right] \frac{v^2}{2g} \qquad (6\text{-}6)$$

where:

H = Total head loss, or the elevation difference between HW and DTW, in feet (see Figure 6-2 for sketch)

k_e = Entrance loss coefficient (see Table 6-1)

n = Manning's roughness coefficient (see appropriate culvert nomograph)

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/second²

2. Values of n, which are the basis for the nomographs, are presented on each nomograph.

3. To use the box culvert nomograph (Figure 6-9) 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 6-6 for more accurate results.

6.3.4 ROADWAY OVERTOPPING

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

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated (see Sections 6.3.2 and 6.3.3).

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

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

$$Q_o = C_d \, L \, HW_r^{1.5} \qquad (6\text{-}7)$$

where:

Q_o = Overtopping flow rate, in cfs

$C_d = K_t \, C_r$ = Discharge coefficient

K_t = Submergence factor (from Figure 6-27, Part C)

C_r = Unsubmerged discharge coefficient (from Figure 6-27, Part A or B)

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.

6.3.5 EXAMPLE PROBLEM

Example 6-1. Circular Concrete Culvert

Given the following data, design a concrete pipe culvert using appropriate nomographs:

Q_{25} = 160 cfs

Q_{100} = 220 cfs

L = 200 feet

S_0 = 0.01 foot/foot

AHW = 10 feet

25-year TW = 3.5 feet

100-year TW = 4.0 feet

Entrance type is a square edge with headwall, n = 0.012.

1. Select a trial culvert size assuming HW/D = 1.5; therefore:

$$D = \frac{HW}{1.5} = \frac{10}{1.5} = 6.67 \text{ feet}$$

Try a 72-inch concrete pipe.

2. Find the actual headwater depth for the trial culvert size.

a. For inlet control, using Figure 6-3, Q = 160 cfs, D = 72 inches, gives HW/D = 0.85.

$$HW = (0.85)(6.0) = 5.1 \text{ feet}$$

Since 5.1 feet is considerably less than the AHW of 10 feet, try a 60-inch concrete pipe, which yields HW/D = 1.23 (from Figure 6-3).

$$HW = (1.23)(5.0) = 6.15 \text{ feet}$$

Q = 220 cfs, D = 60 inches, yielding HW/D = 1.70

$$HW = (1.70)(5.0) = 8.5 \text{ feet}$$

b. For outlet control, using Figures 6-4 and 6-20, TW = 3.5 and 4.0 is less than D = 5.0 feet.

From Table 6-1, $k_e = 0.5$, Q = 160 cfs, D = 60 inches, yielding d_c = 3.6 feet (Figure 6-20).

Since d_c is less than D = 5.0 feet, h_o is determined using Equation 6-3:

$$h_o = \frac{3.6 + 5.0}{2} = 4.3 \text{ feet}$$

From Figure 6-4, H = 2.2 feet. Using Equation 6-2:

$$HW = 2.2 + 4.3 - (0.01)(200) = 4.5 \text{ feet}$$

Q = 220 cfs, D = 60 inches, gives d_c = 4.2 feet (Figure 6-20).

Since d_c is less than D = 5.0 feet, h_o is determined using Equation 6-3:

$$h_o = \frac{4.2 + 5.0}{2} = 4.6 \text{ feet}$$

From Figure 6-4, H = 4.2 feet. Using Equation 6-2:

$$HW = 4.2 + 4.6 - (0.01)(200) = 600 \text{ feet}$$

c.

Flow (cfs)	Headwater (feet) Inlet Control	Headwater (feet) Outlet Control
160	6.15	4.5
220	8.50	6.8

Since the headwater depths for inlet control are greatest under both Q_{25} and Q_{100} flow conditions, inlet control governs flow through the culvert.

3. A 54-inch pipe would be adequate for Q_{25}, but since HW = 10.8 for Q_{100}, it is not selected.

4. Compute the outlet velocity. Since the culvert will operate under inlet control, use Manning's Equation as expressed by Equation 5-5:

$$v = \frac{0.592}{(0.012)} (5.0)^{2/3} (0.01)^{1/2}$$

$$v = 14.4 \text{ feet/second}$$

Since this velocity exceeds permissible velocities for bare soil (see Table 5-2), erosion protection is required (see Chapter 10).

Table 6-1
CULVERT ENTRANCE LOSS COEFFICIENTS

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

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

[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.

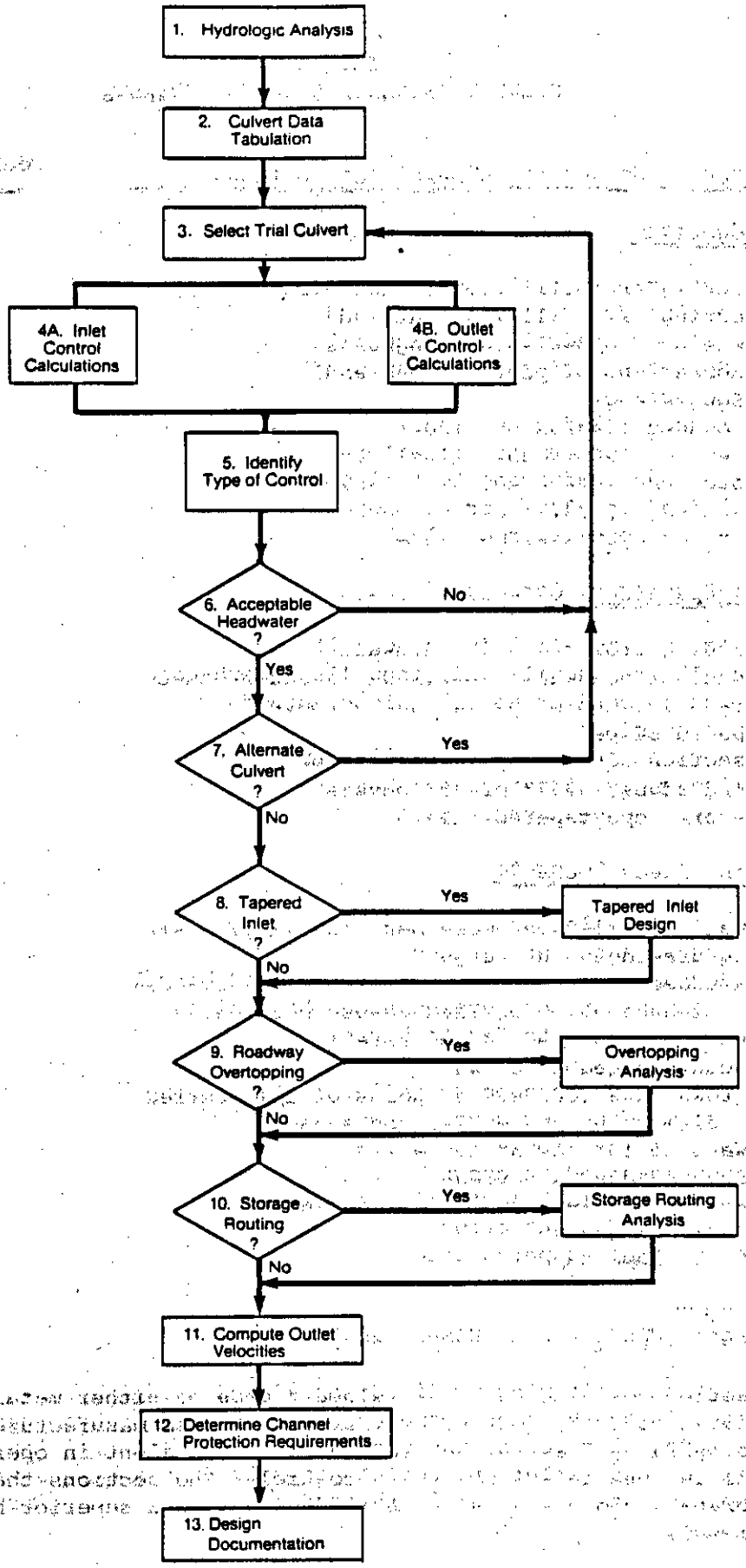


FIGURE 6-1
General Culvert Design Flow Chart

PROJECT: _____

STATION: _____

SHEET _____ OF _____

CULVERT DESIGN FORM

DESIGNER / DATE: _____ / _____

REVIEWER / DATE: _____ / _____

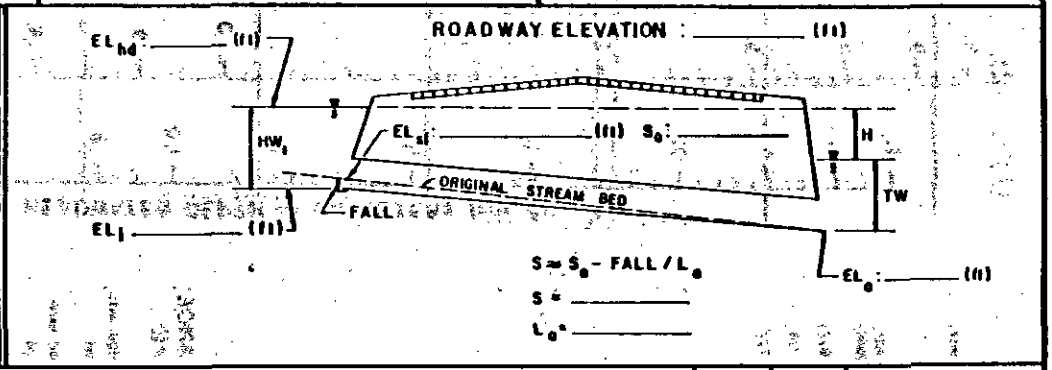
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: _____
- ☐ DRAINAGE AREA: _____ ☐ STREAM SLOPE: _____
- ☐ CHANNEL SHAPE: _____
- ☐ ROUTING: _____ ☐ OTHER _____

DESIGN FLOWS/TAILWATER

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



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)	h_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
- i 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: _____

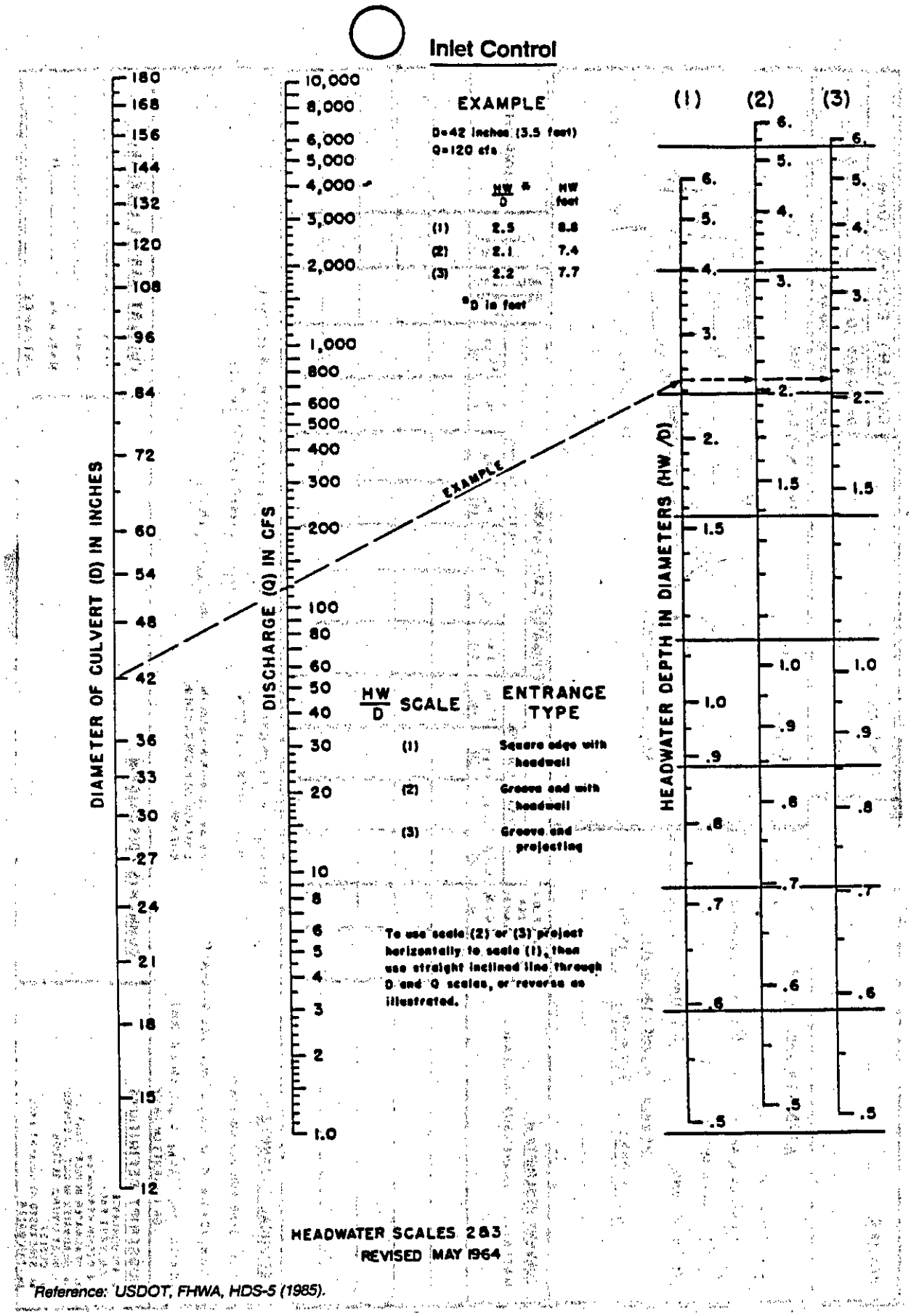
SHAPE: _____

MATERIAL: _____ n _____

ENTRANCE: _____

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

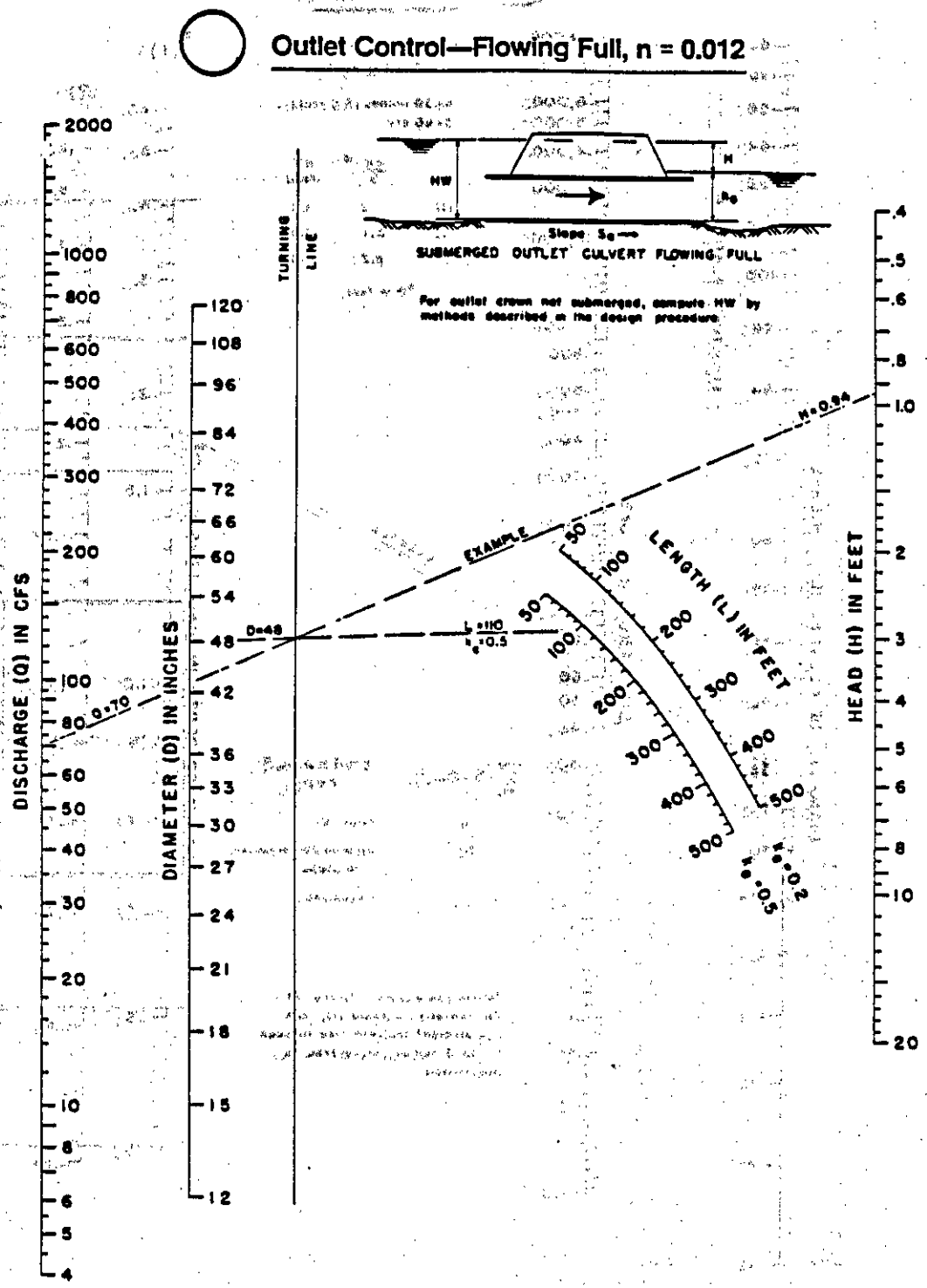
FIGURE 6-2
Culvert Design Form



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

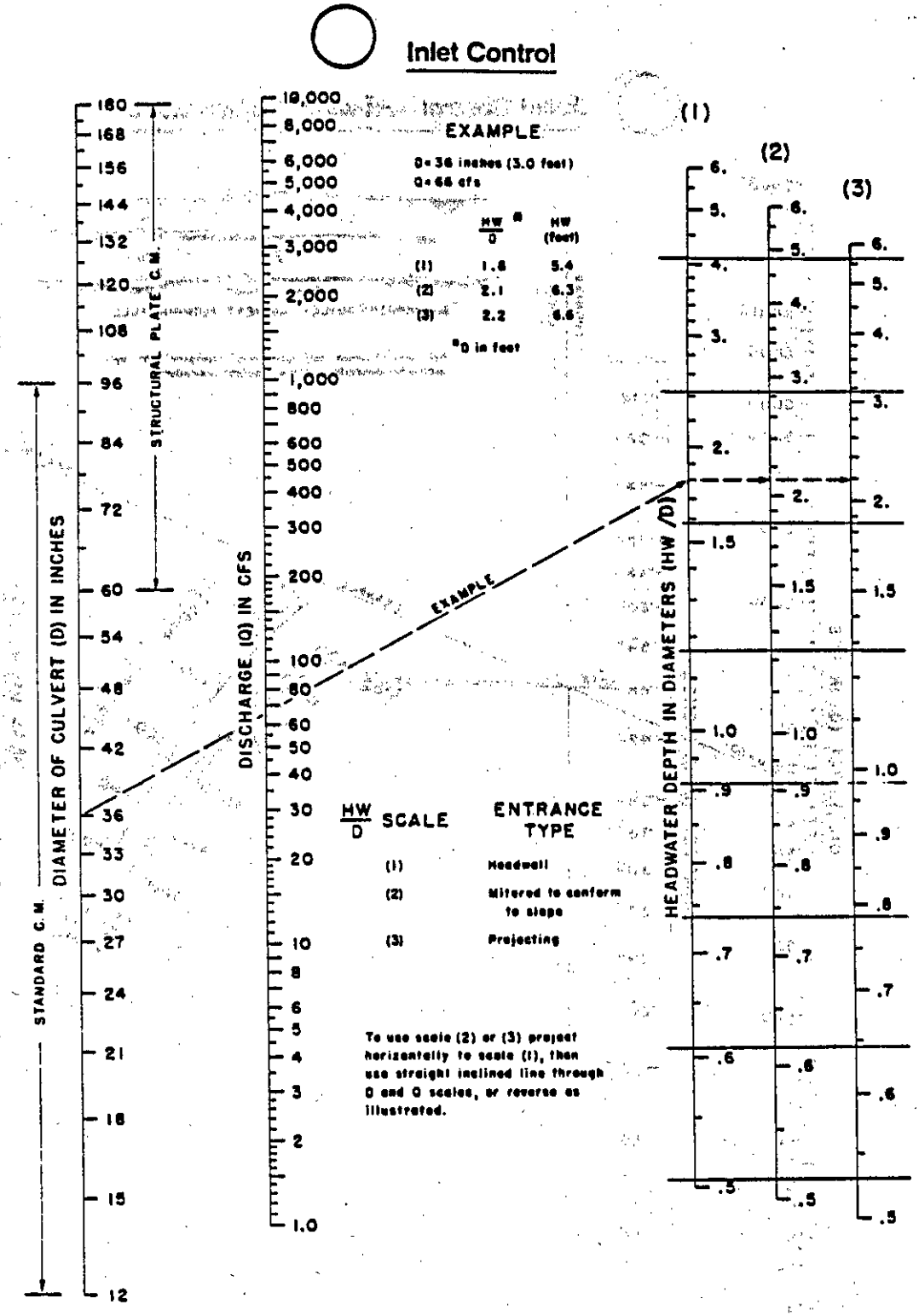
FIGURE 6-3

Inlet Control Chart for Circular Concrete Pipe Culverts



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

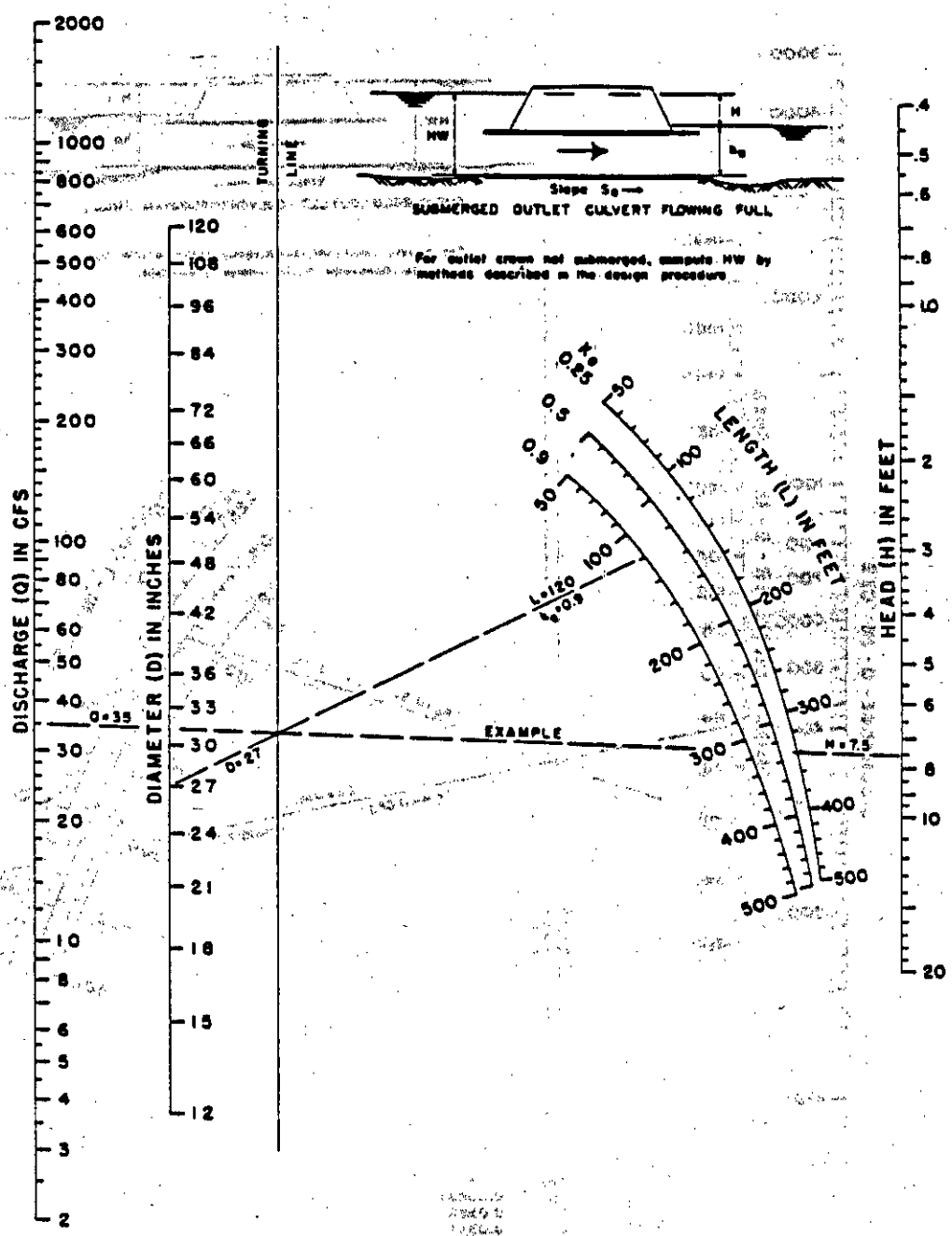
FIGURE 6-4
Outlet Control Chart for Circular Concrete Pipe Culverts Flowing Full



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

FIGURE 6-5
Inlet Control Chart for Circular CMP Culverts

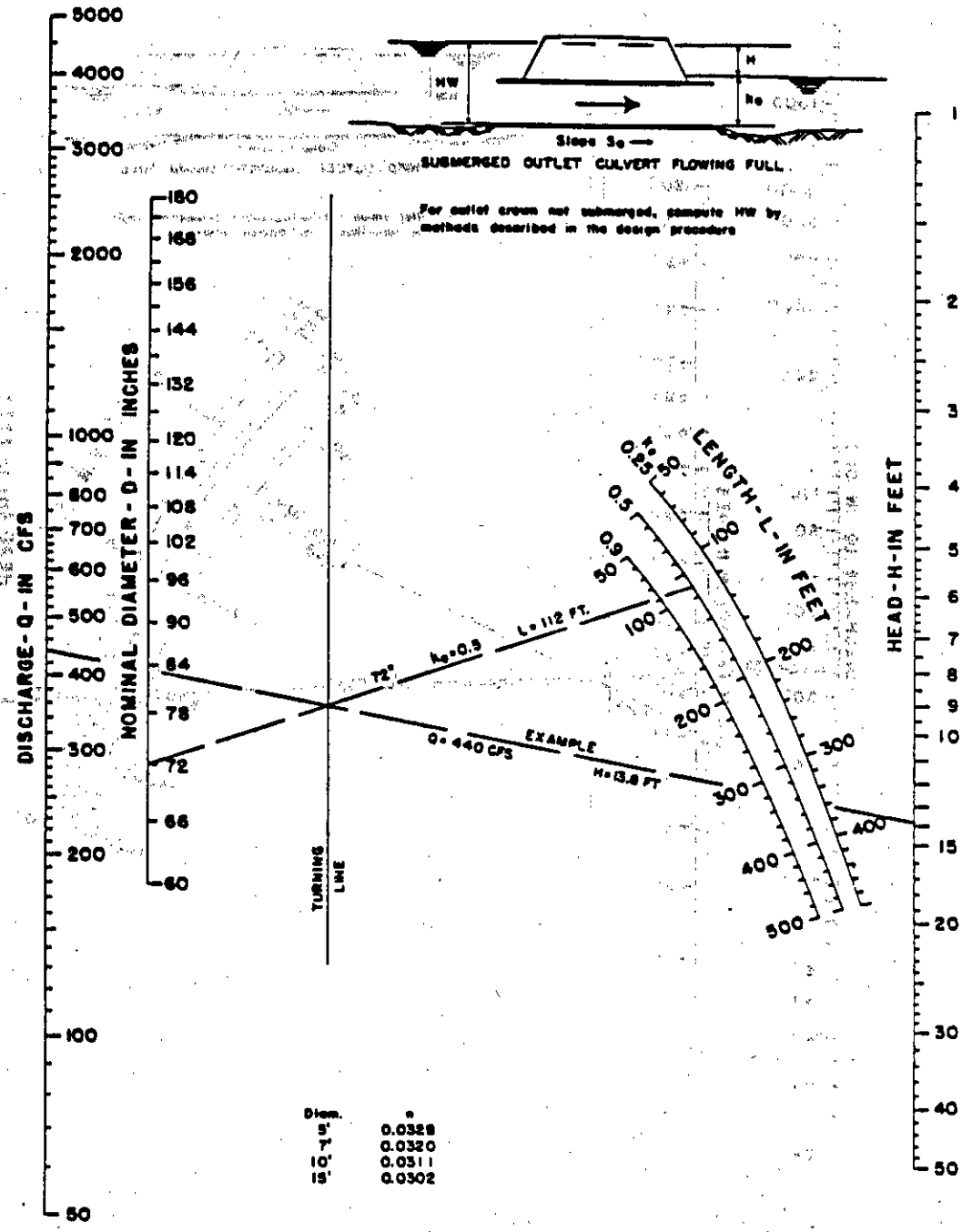
Outlet Control—Flowing Full, n = 0.024



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

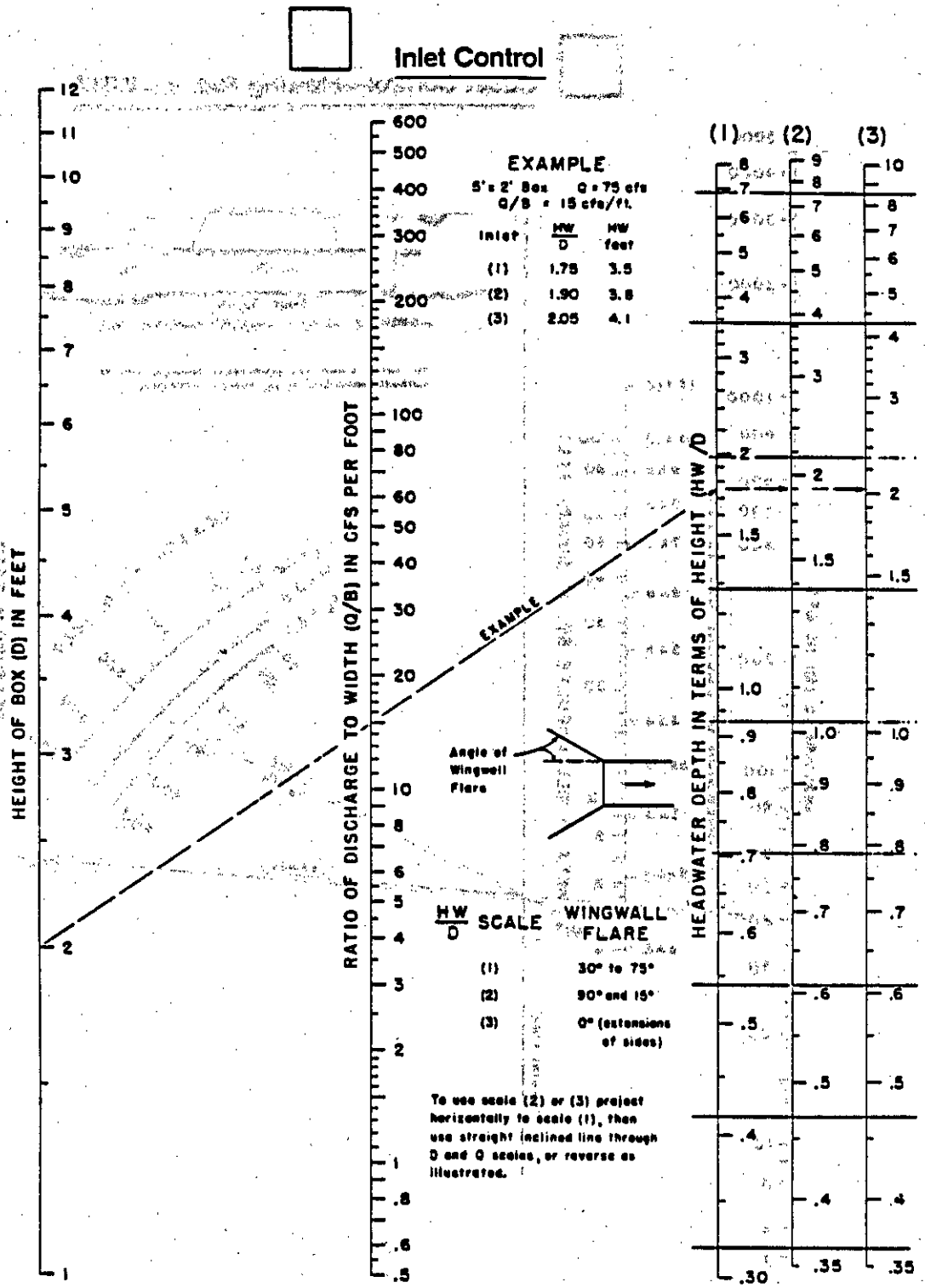
FIGURE 6-6
Outlet Control Chart for Circular CMP Culverts Flowing Full

Outlet Control—Flowing Full, n = 0.0328 to 0.0302



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

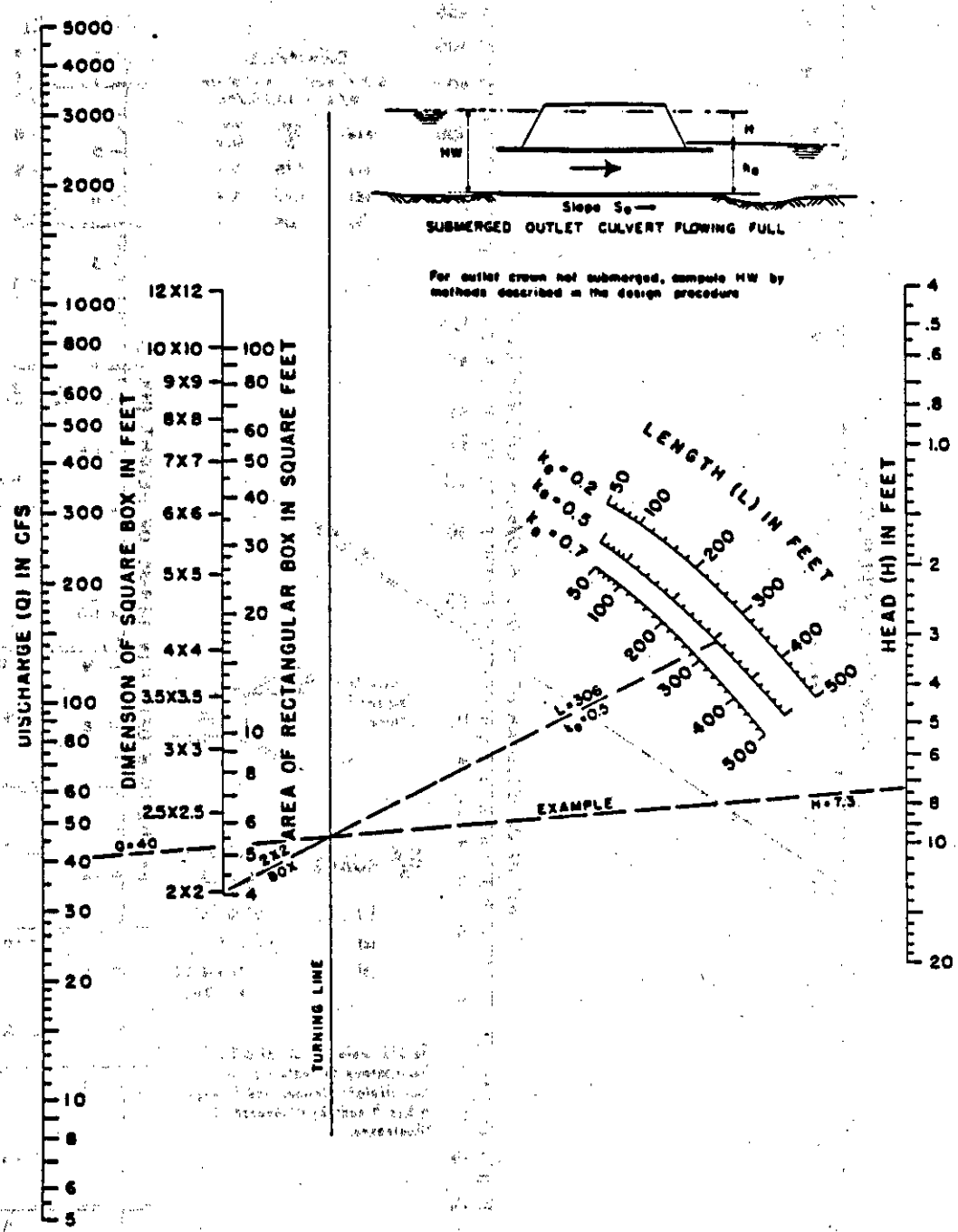
FIGURE 6-7
Outlet Control Chart for Structural Plate CMP Culverts Flowing Full



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

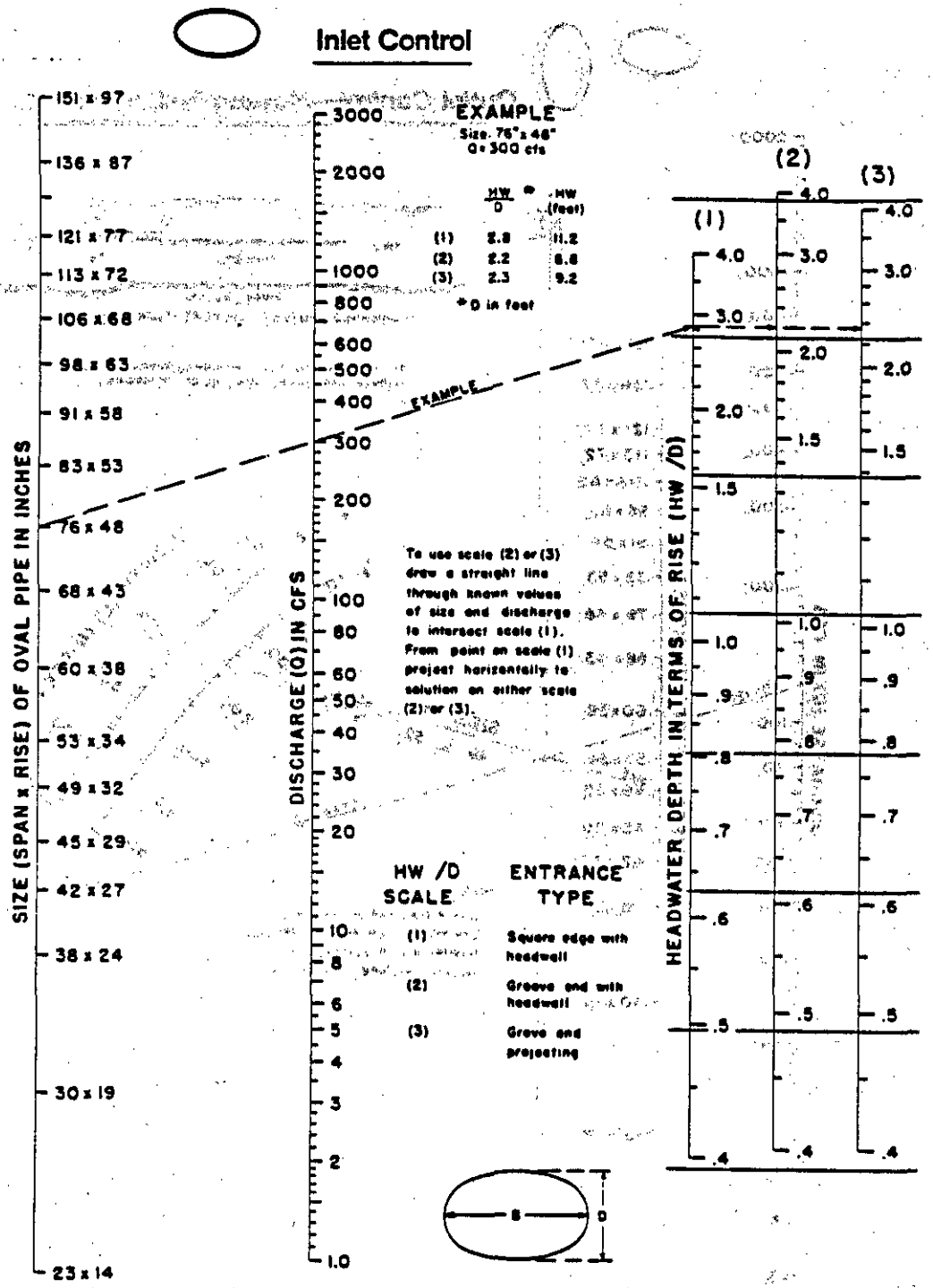
FIGURE 6-8
Inlet Control Chart for Concrete Box Culverts

Outlet Control—Flowing Full, n = 0.012



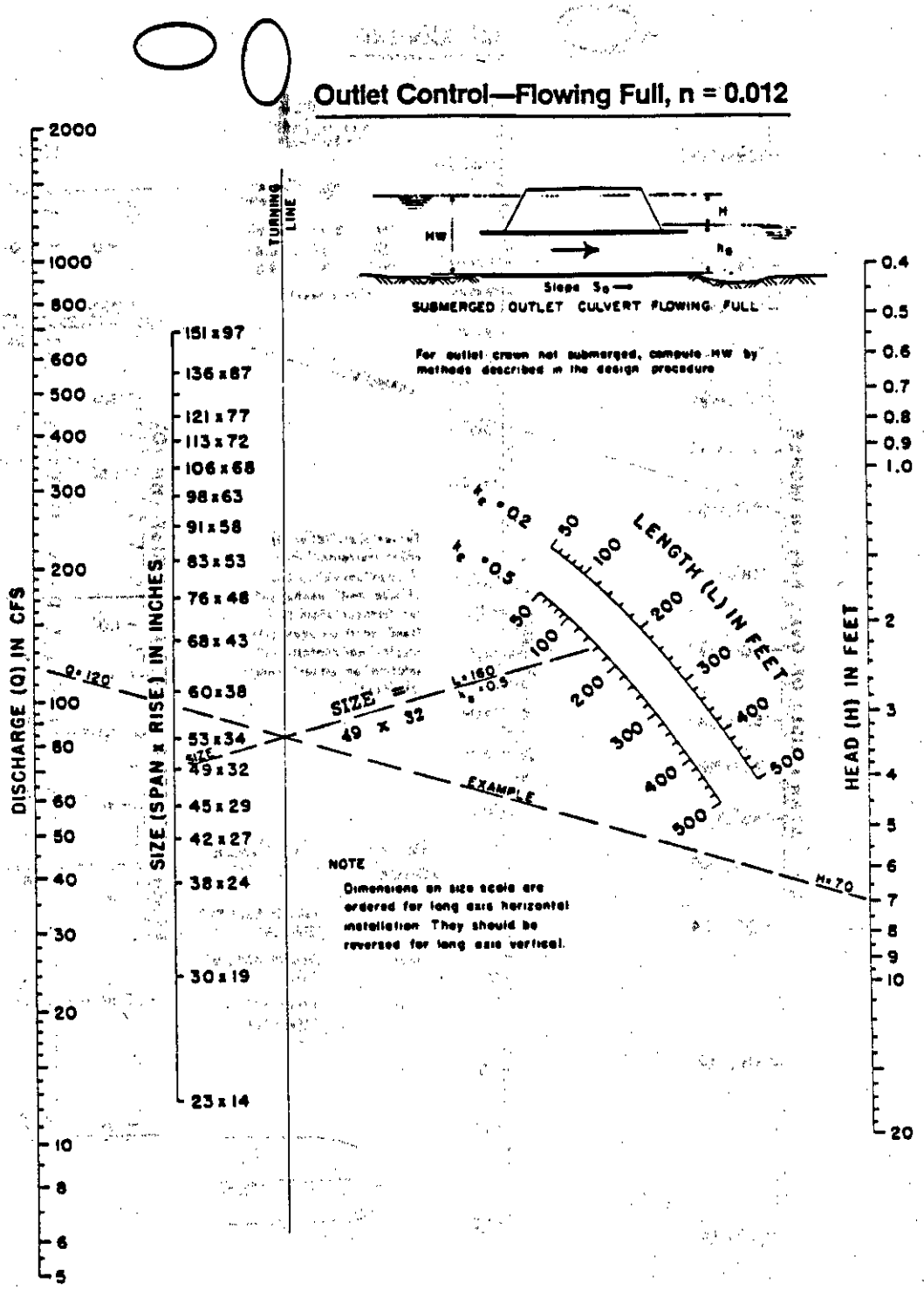
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-9
Outlet Control Chart for Concrete Box Culverts Flowing Full



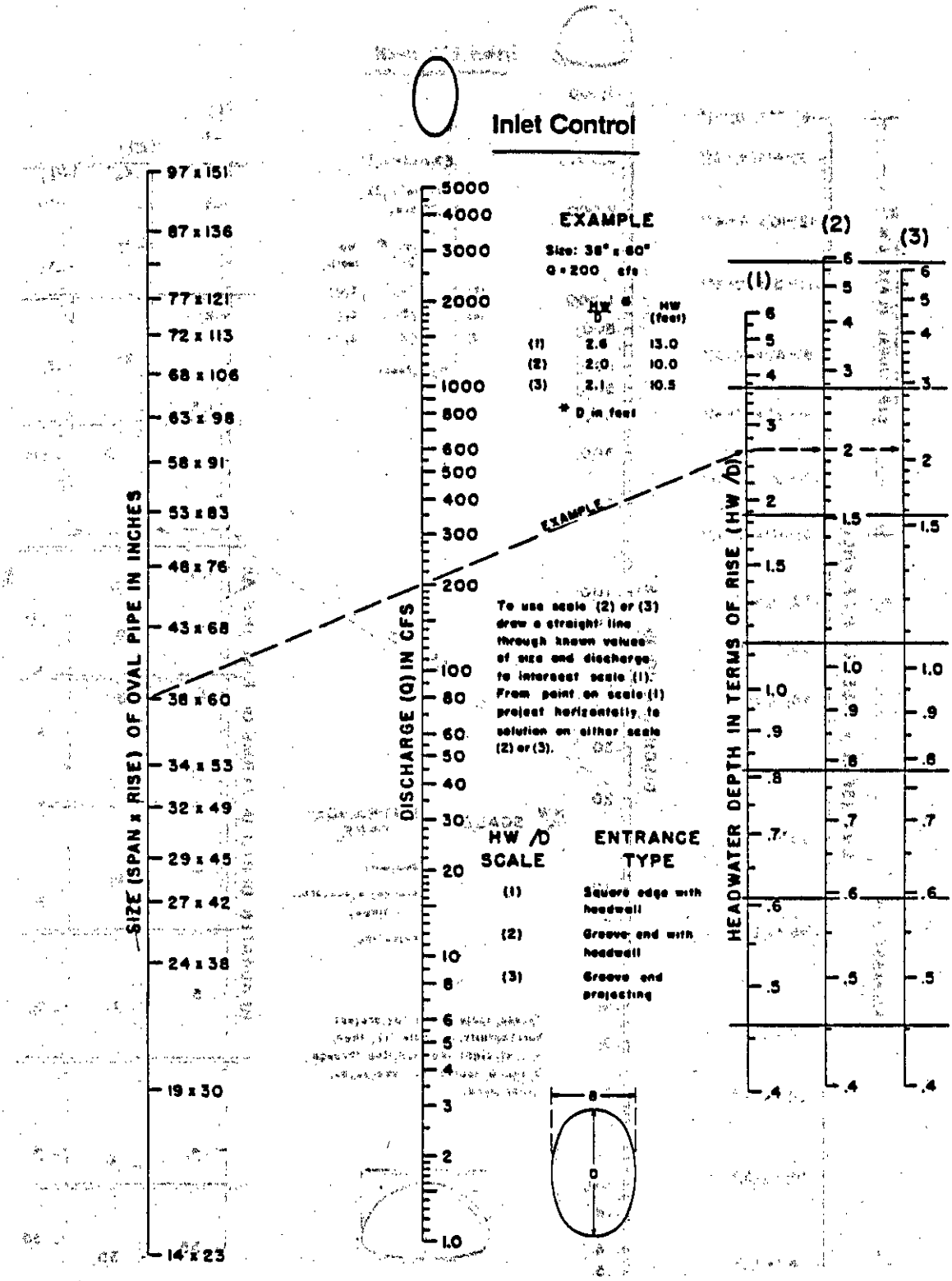
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-10
Inlet Control Chart for Oval Concrete Pipe Culverts—Long Axis Horizontal



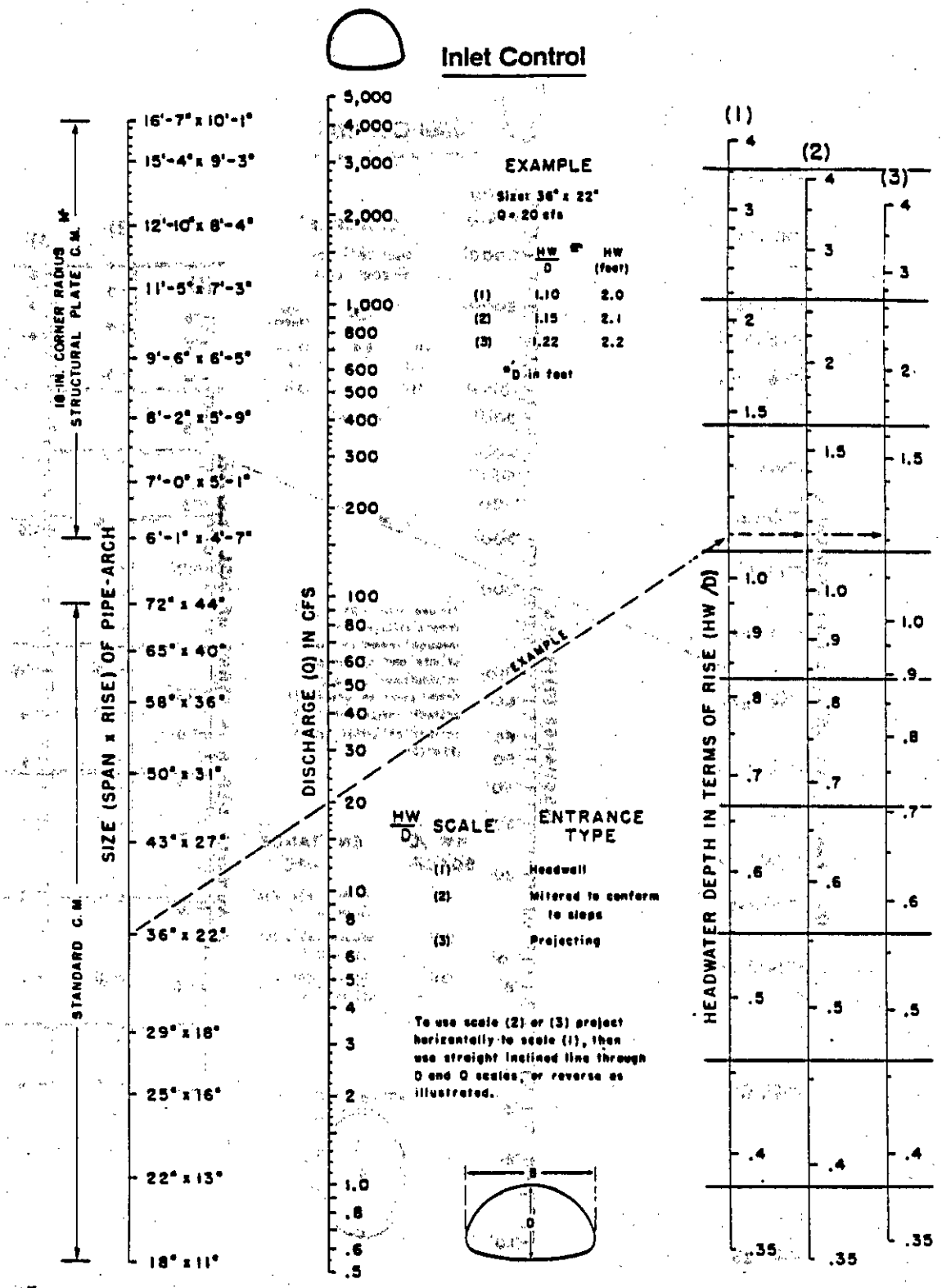
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-11
Outlet Control Chart for Oval Concrete Pipe Culverts Flowing Full—Long Axis Horizontal or Vertical



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

FIGURE 6-12
Inlet Control Chart for Oval Concrete Pipe Culverts—Long Axis Vertical

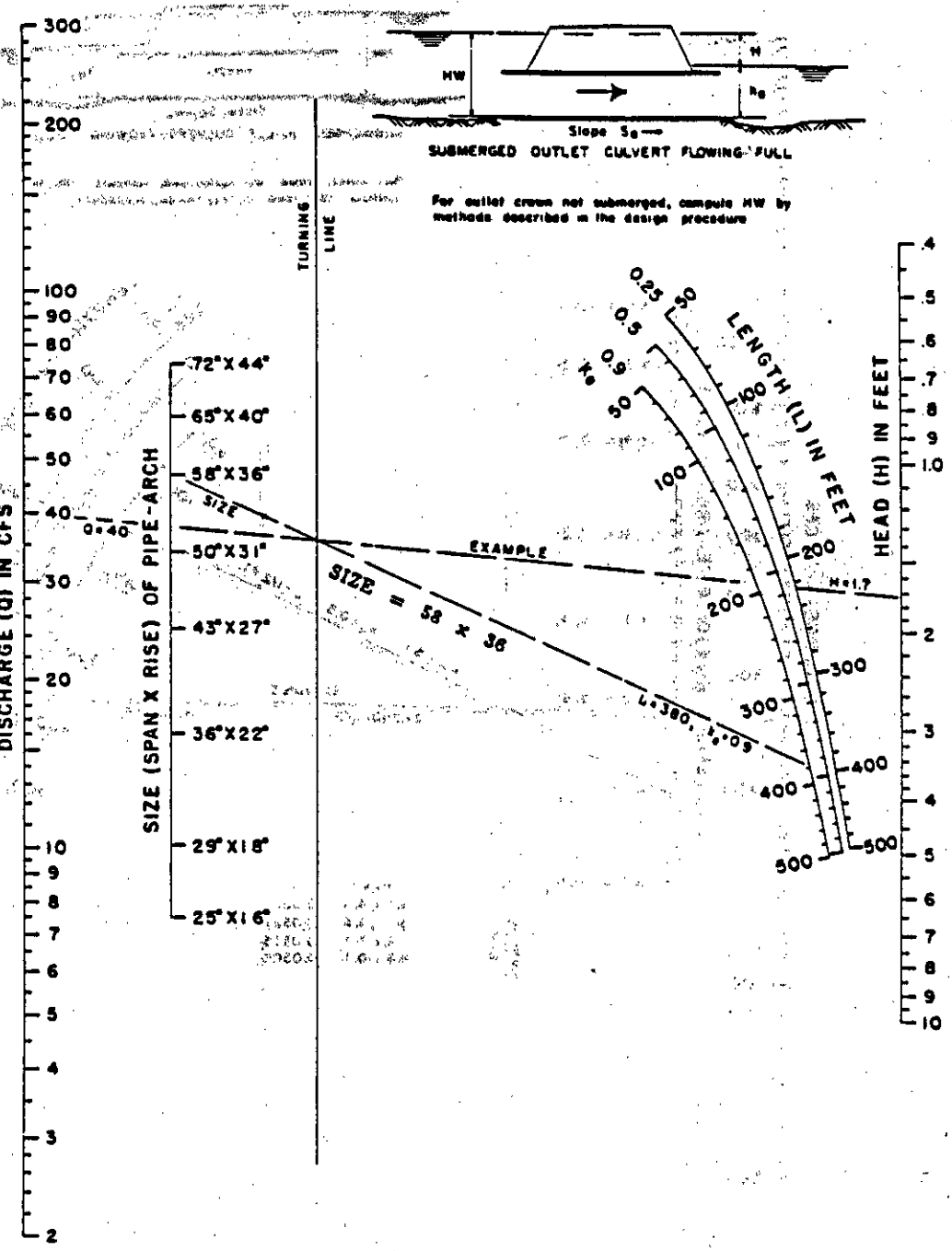


*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

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

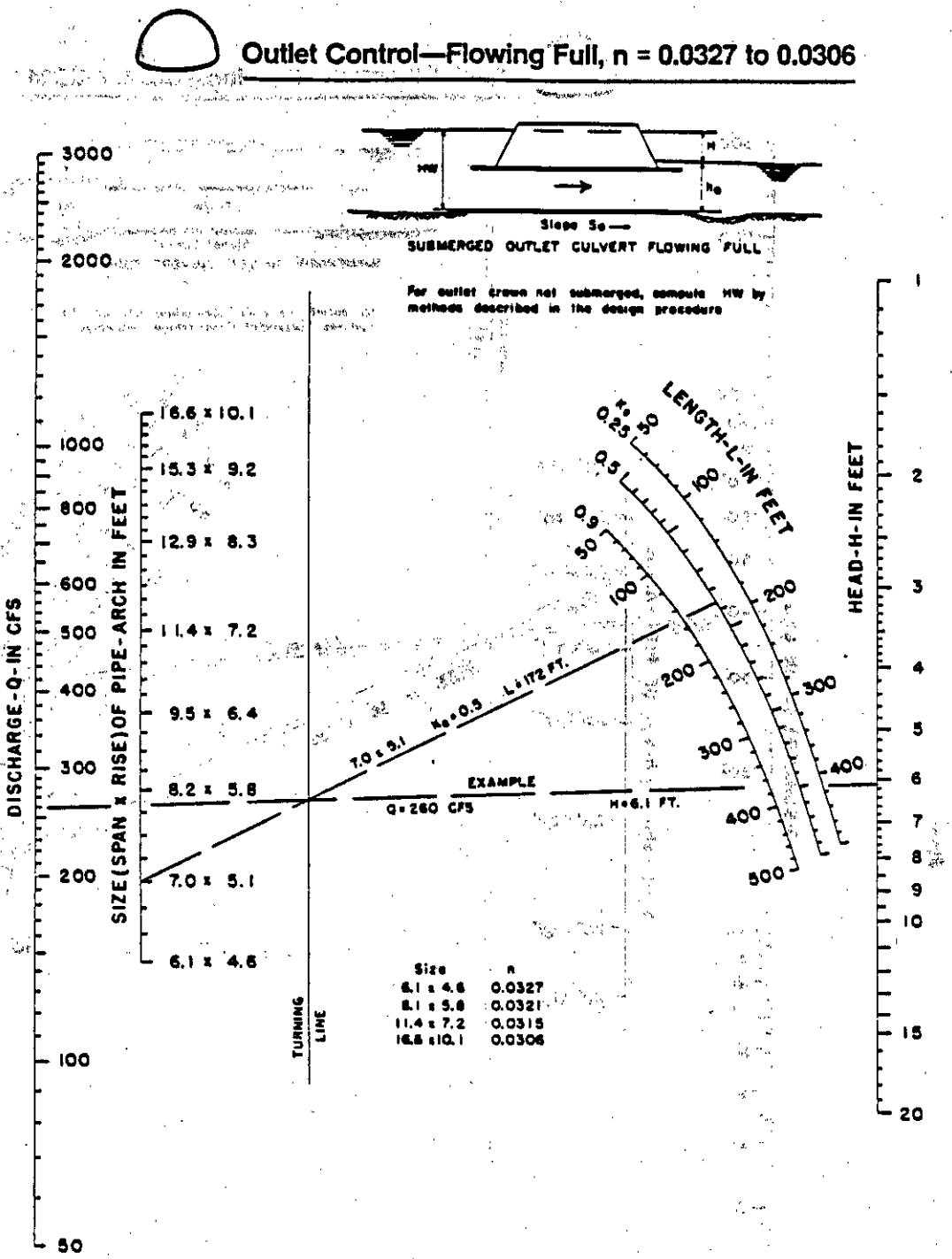
FIGURE 6-13
Inlet Control Chart for CMP Arch Culverts

Outlet Control—Flowing Full, n = 0.024



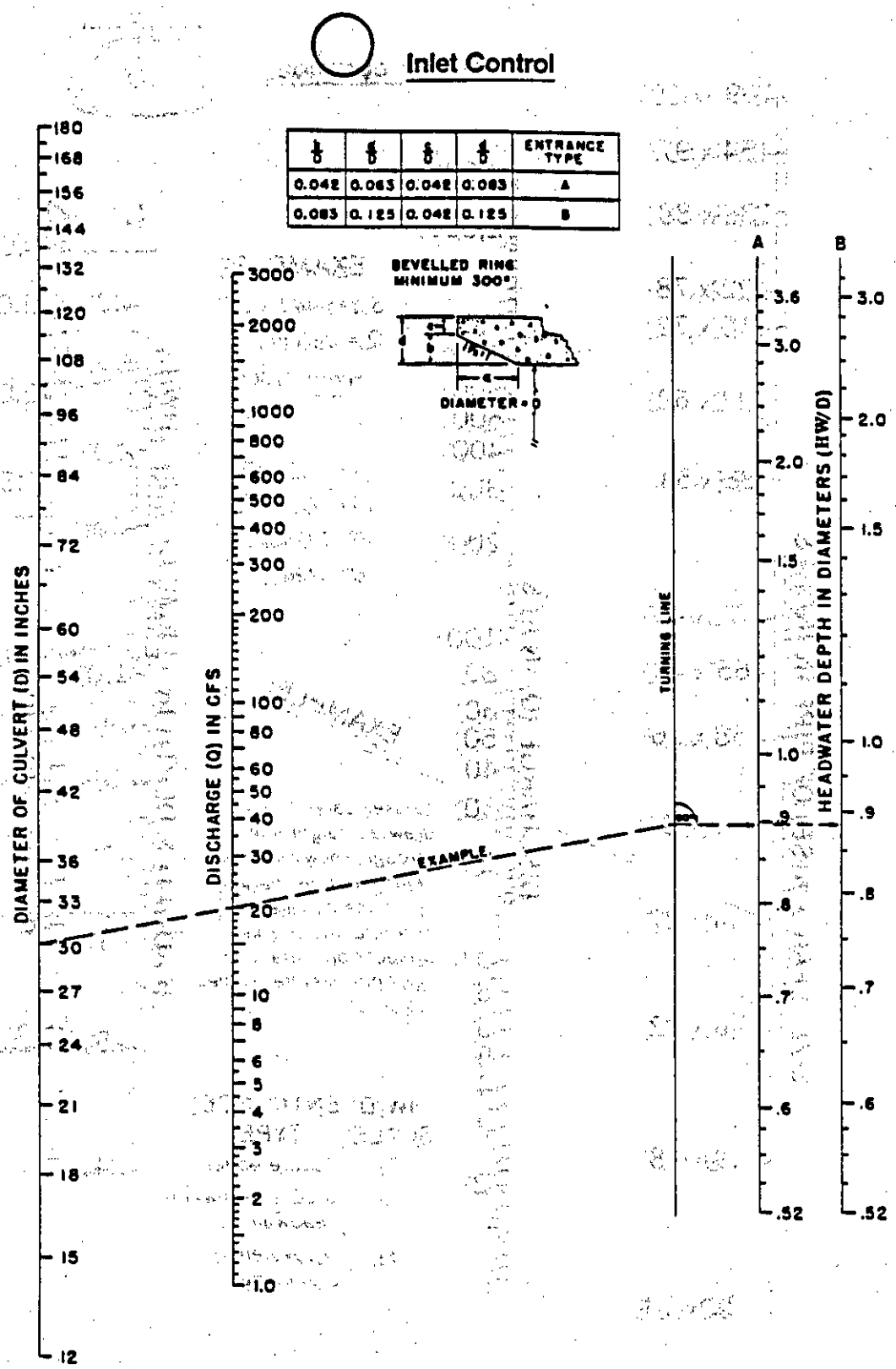
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-14
Outlet Control Chart for CMP Arch Culverts Flowing Full



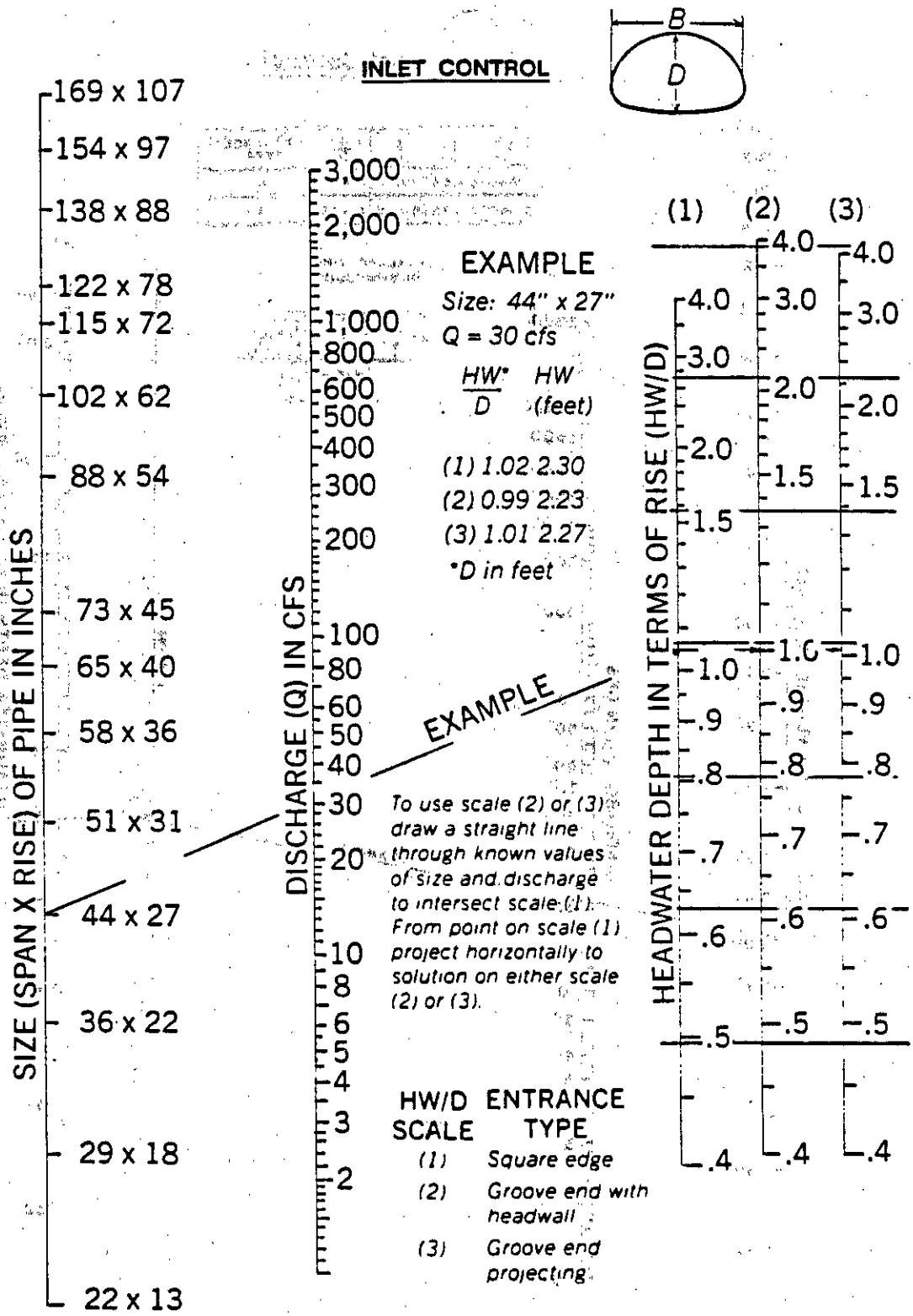
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-15
Outlet Control Chart for Structural Plate CMP Arch Culverts (18-inch Corner Radius) Flowing Full



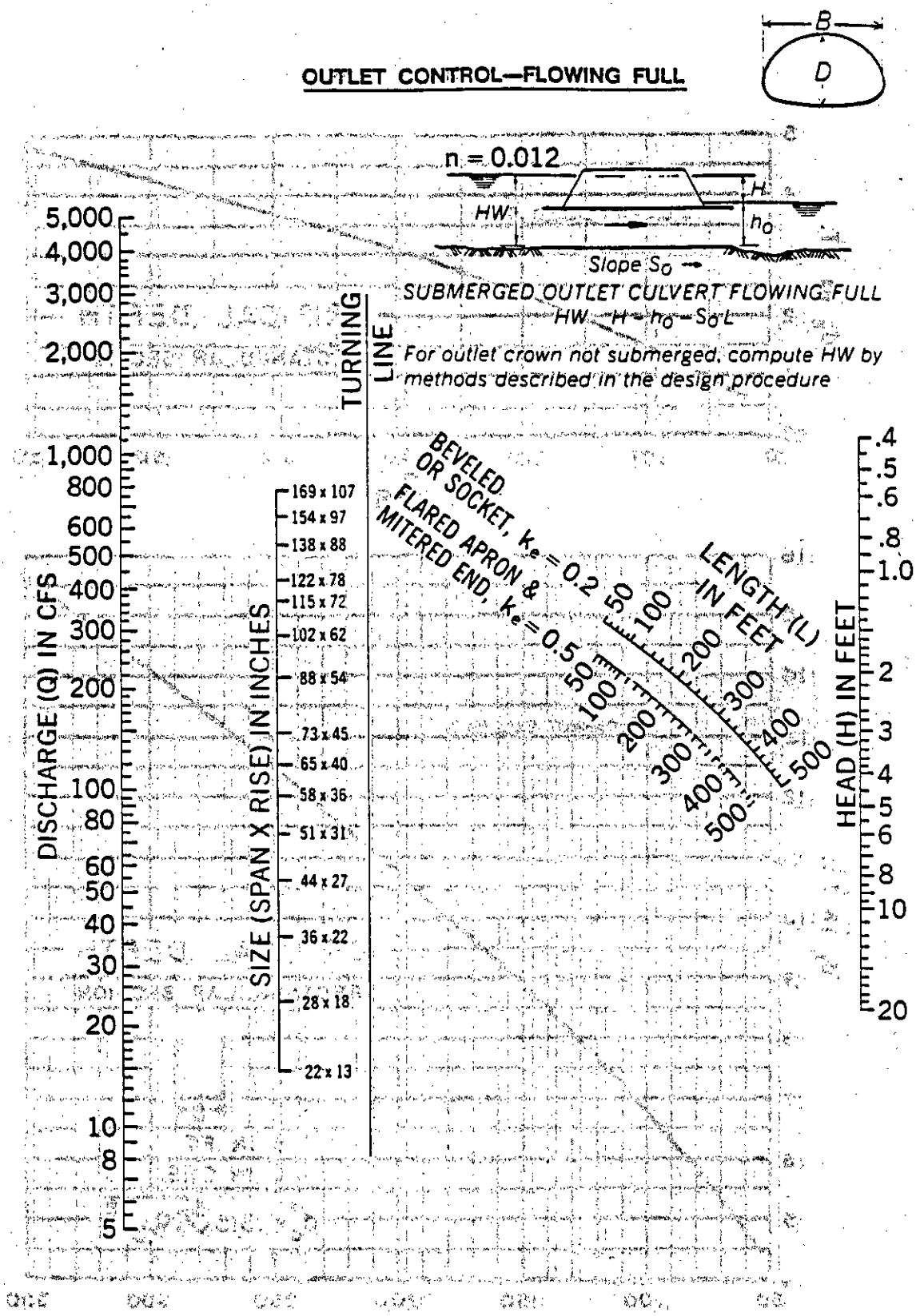
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-16
Inlet Control Chart for Circular Pipe Culverts with Beveled Ring



Reference: American Concrete Pipe Association (1980).

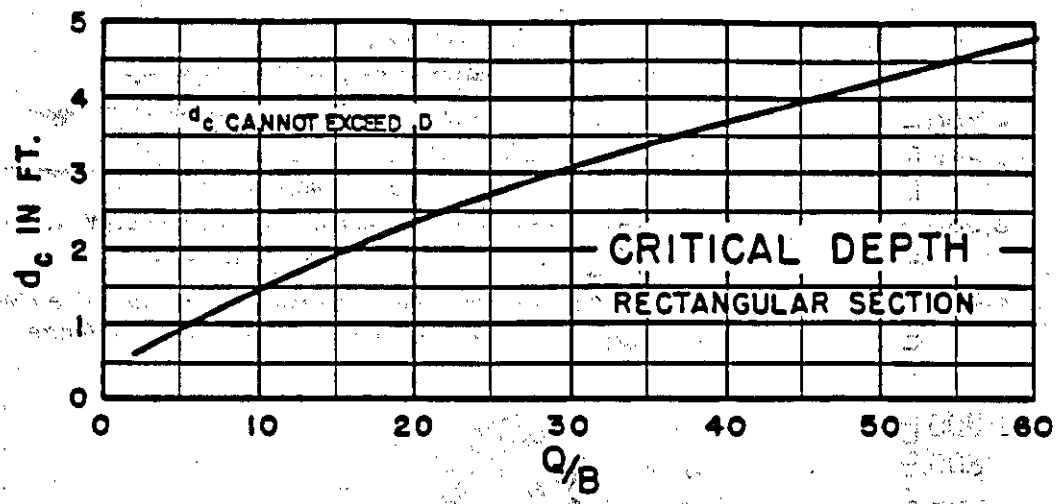
FIGURE 6-17
Inlet Control Chart for Concrete Pipe Arch Culverts

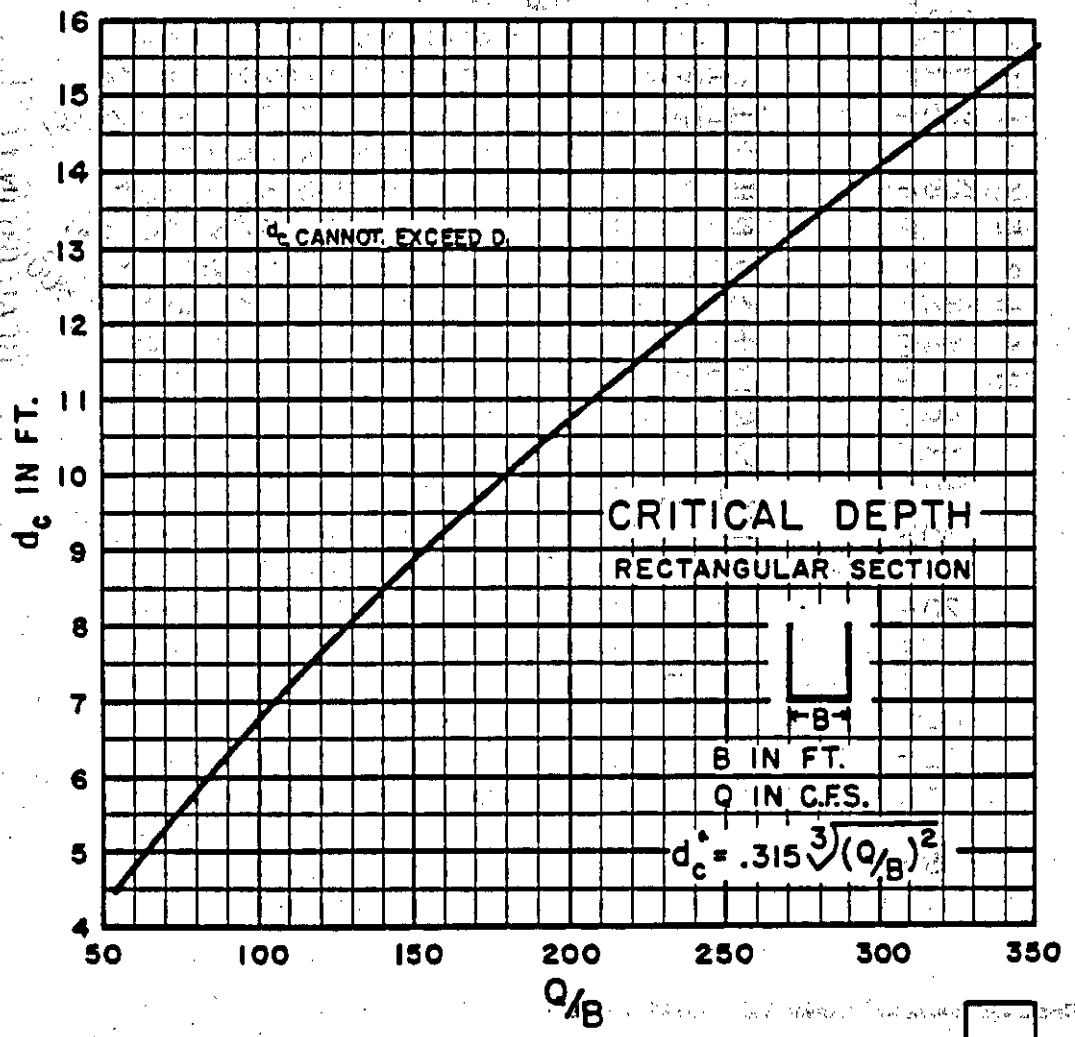


Reference: American Concrete Pipe Association (1980).

FIGURE 6-18

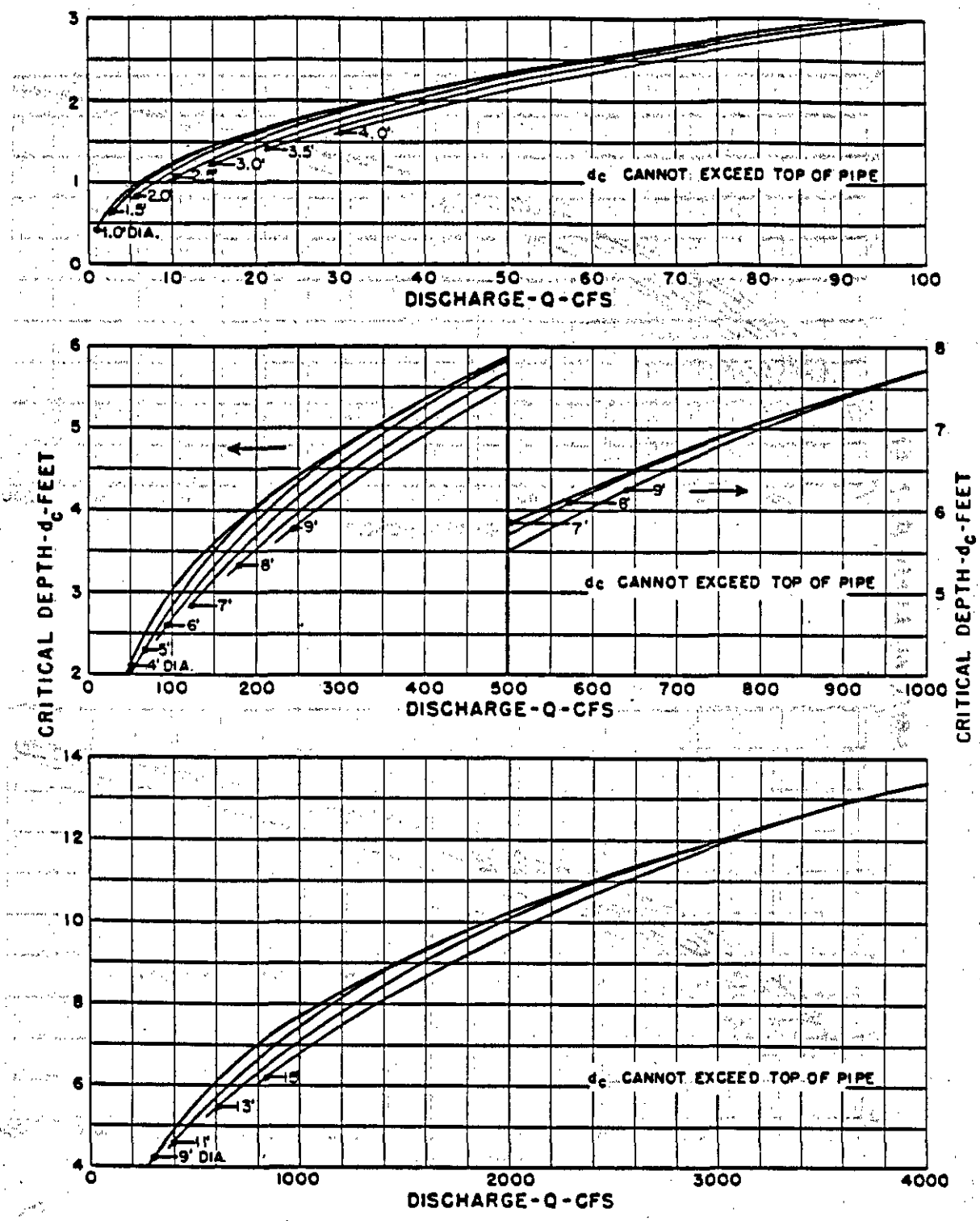
Outlet Control Chart for Arch Concrete Pipe Culverts Flowing Full





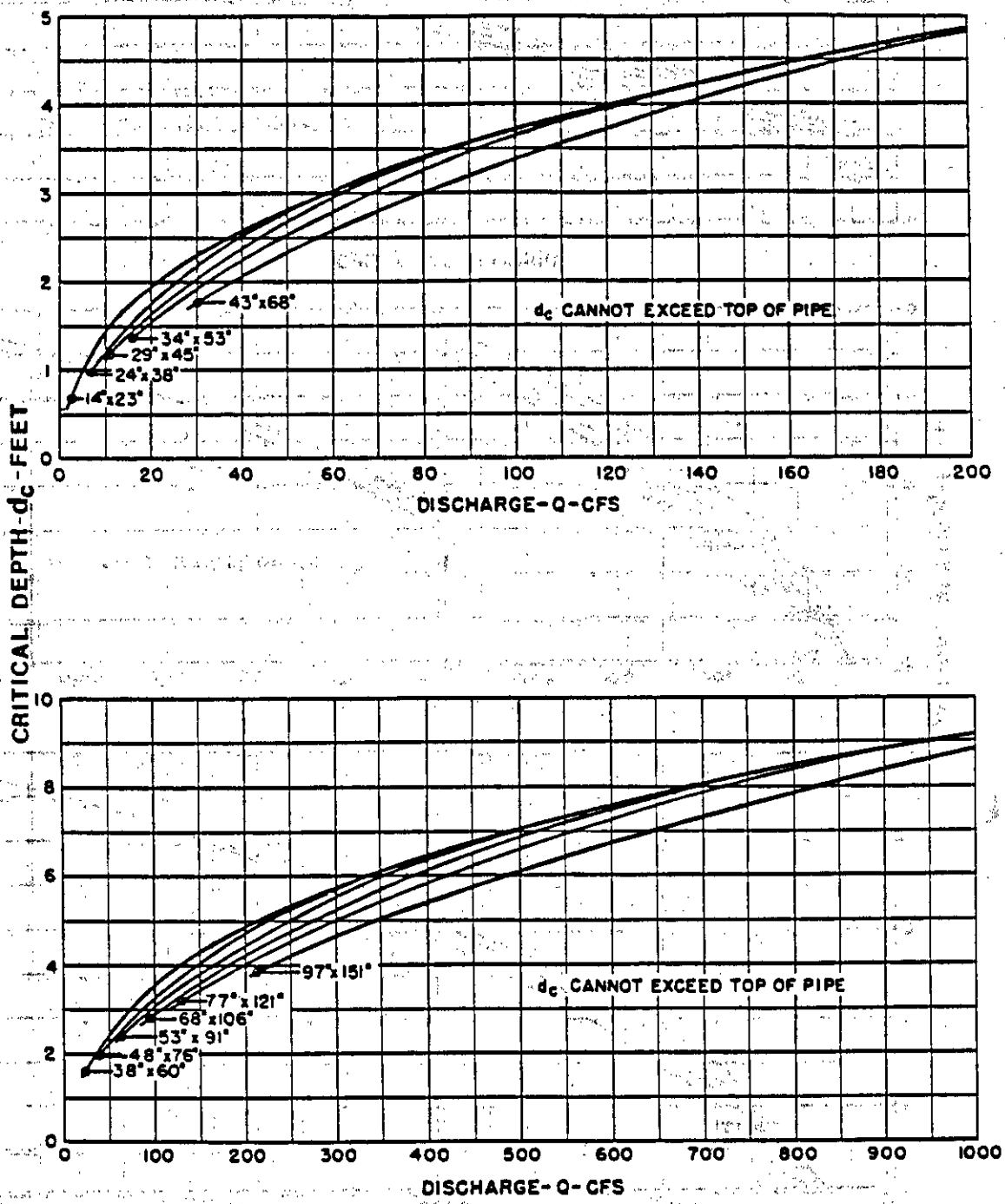
Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-19
Critical Depth Chart for Rectangular Sections



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

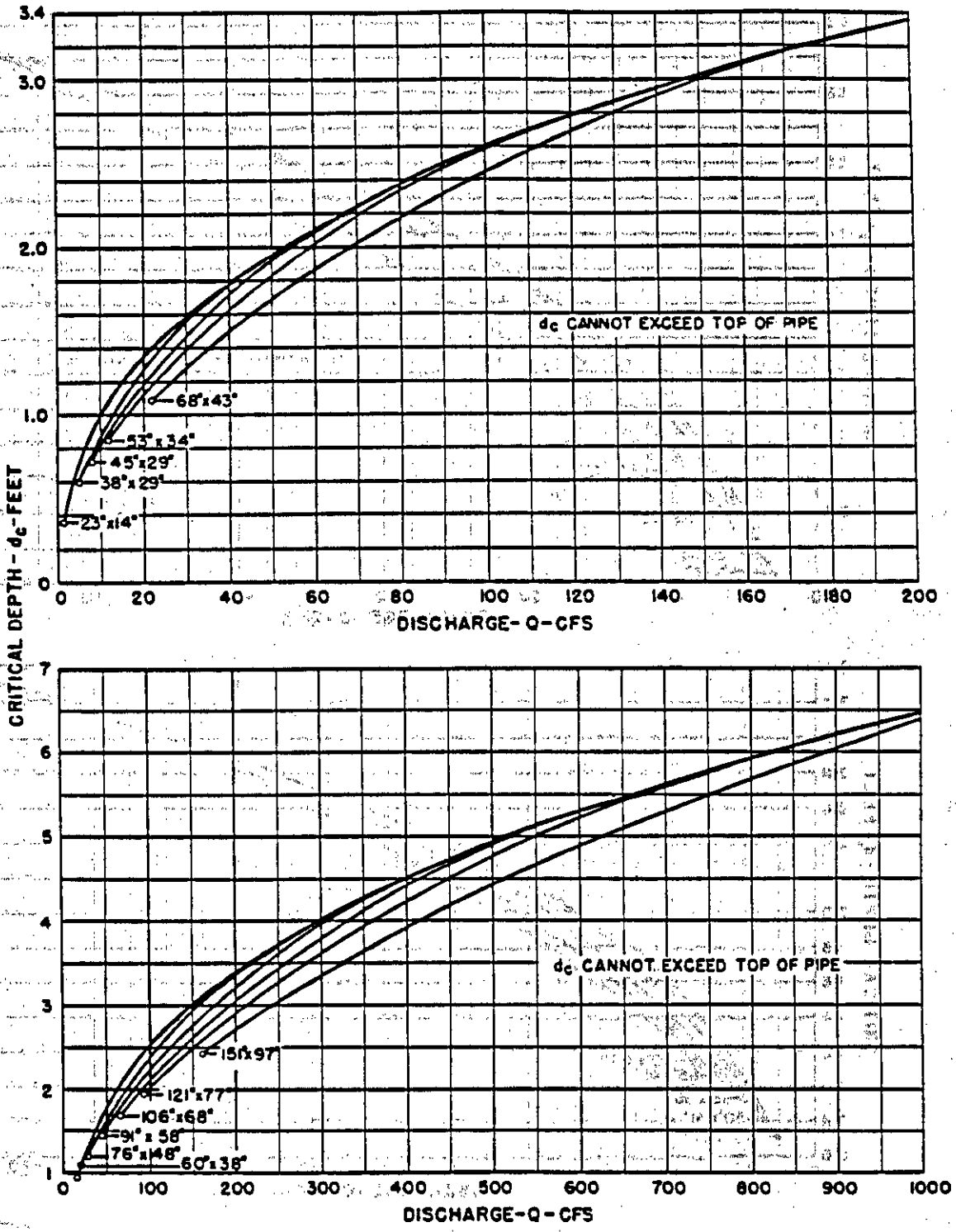
FIGURE 6-20
Critical Depth Chart for Circular Pipe



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

FIGURE 6-21

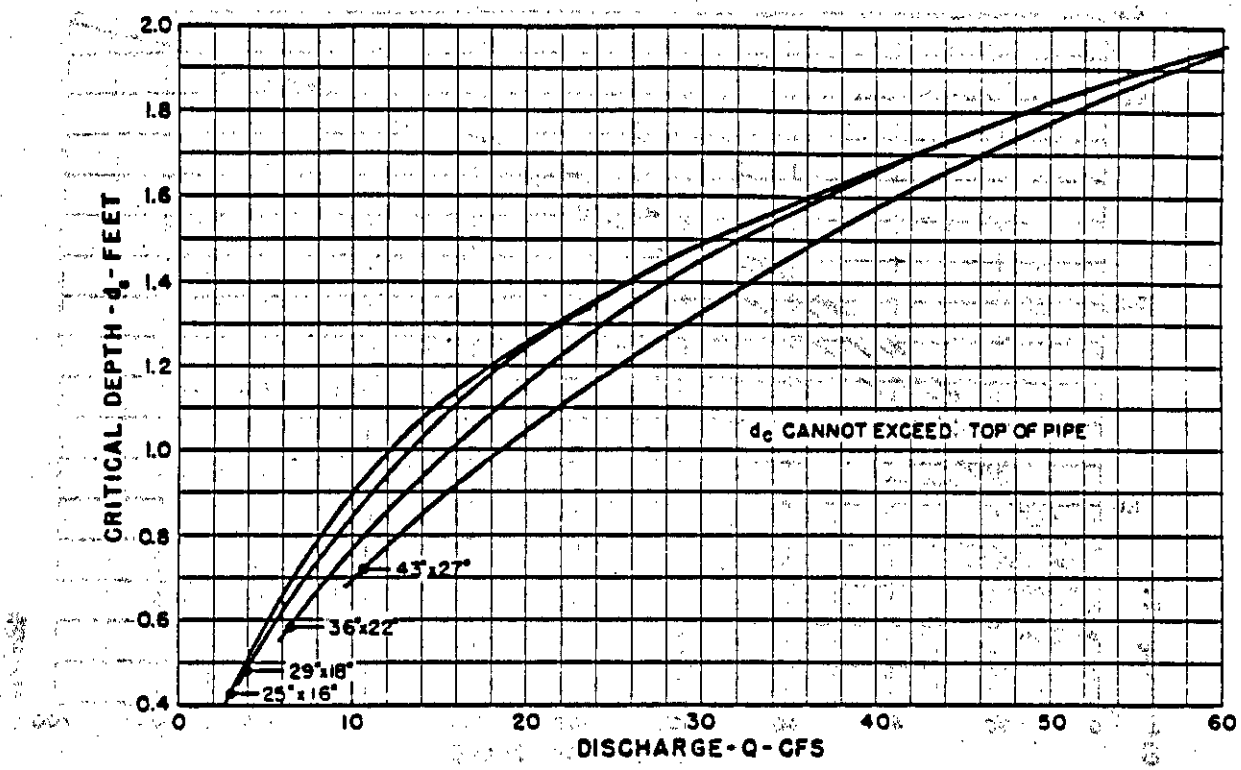
Critical Depth Chart for Oval Concrete Pipe—Long Axis Vertical

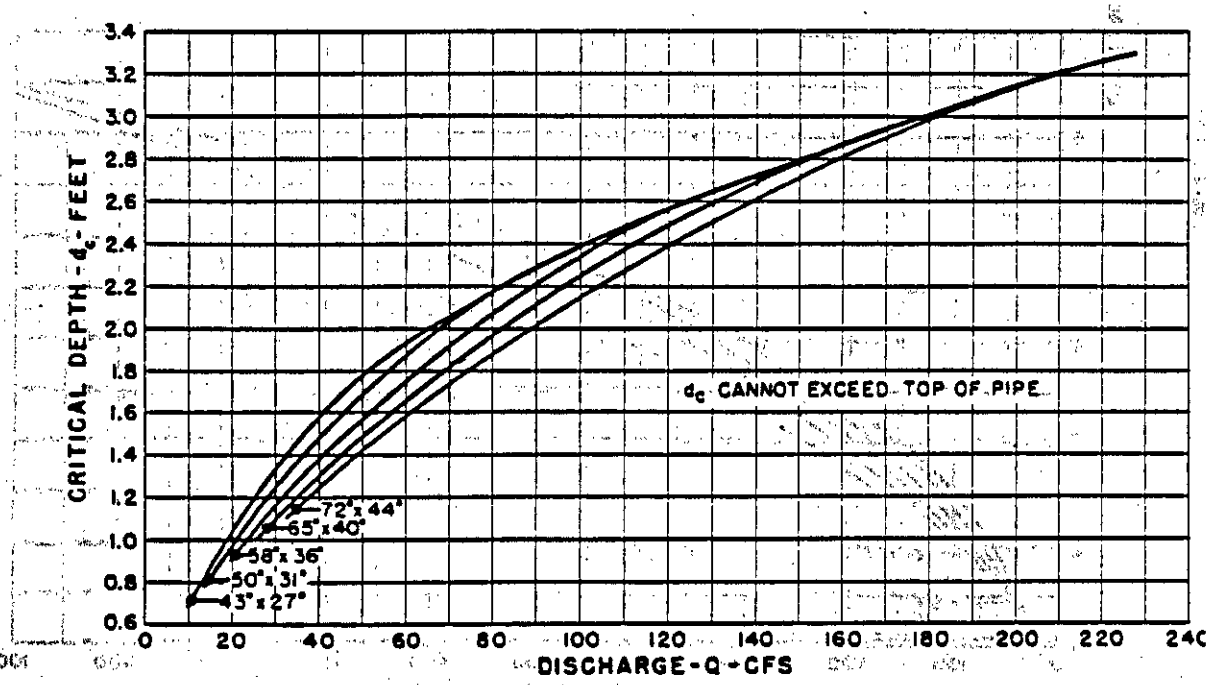


Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-22

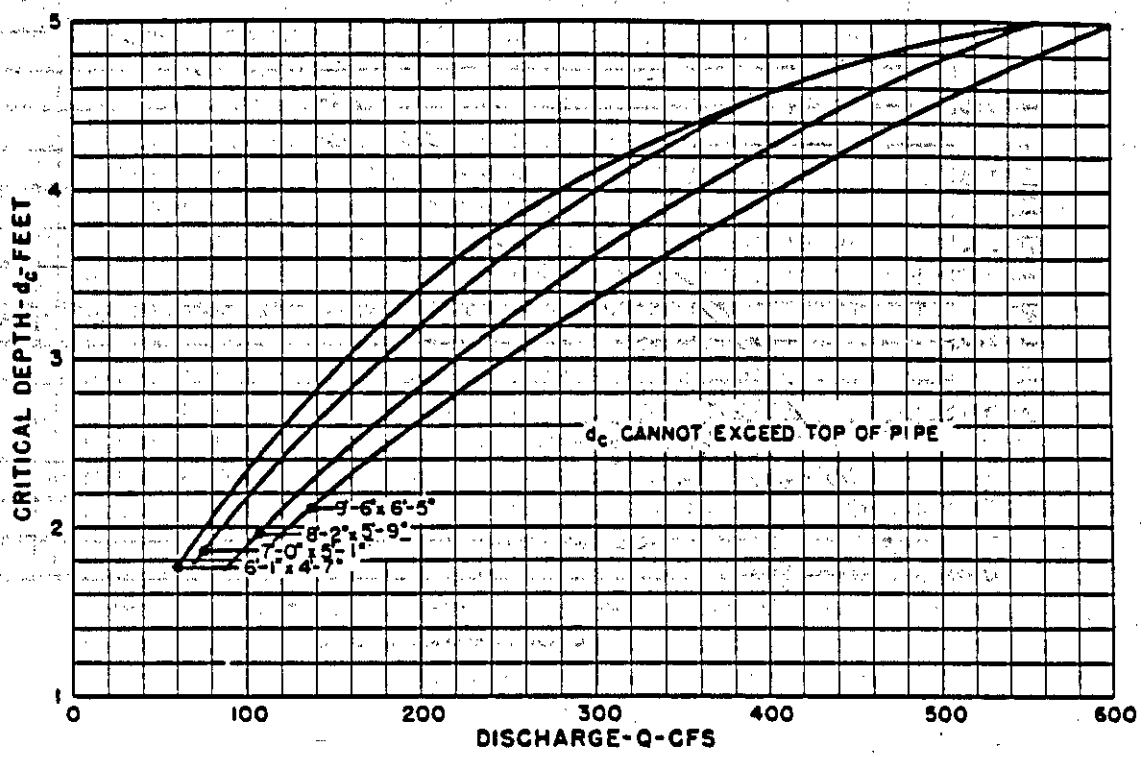
Critical Depth Chart for Oval Concrete Pipe—Long Axis Horizontal

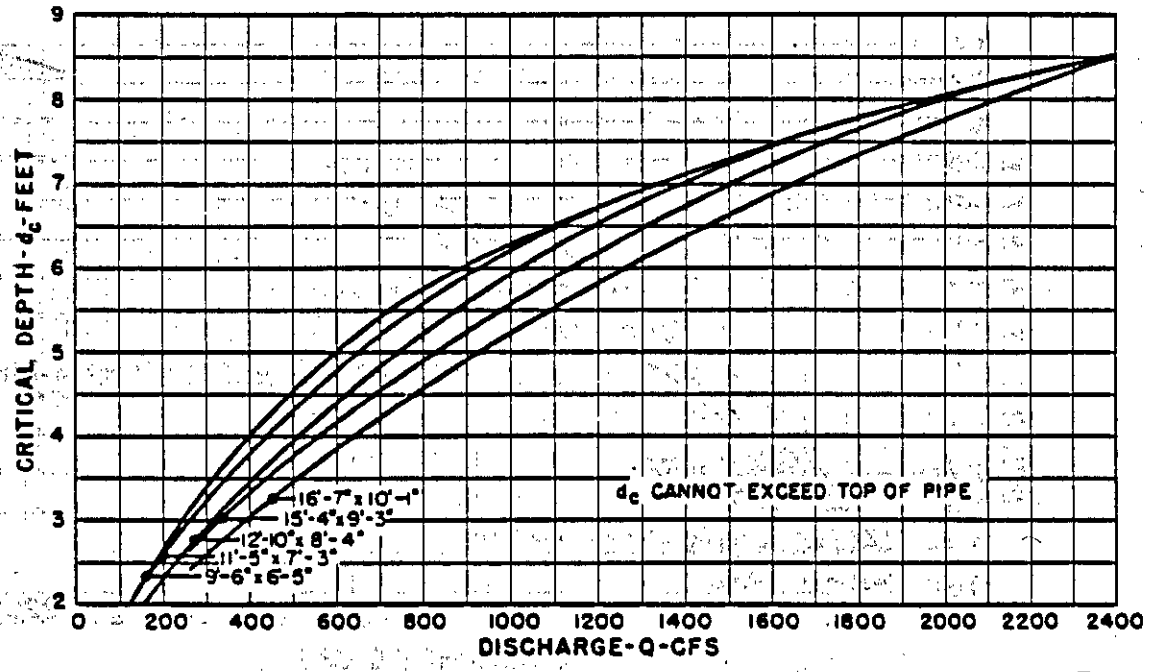




Reference: USDOT, FHWA, HDS-5 (1985).

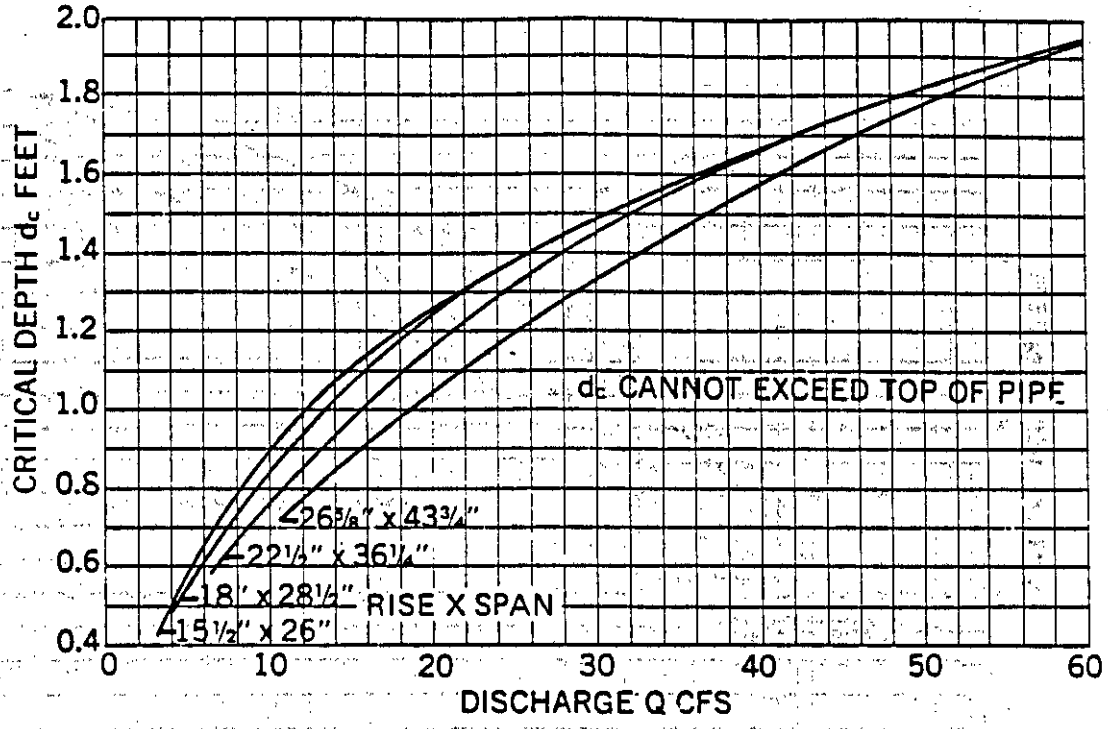
FIGURE 6-23
Critical Depth Chart for Standard CMP Arch

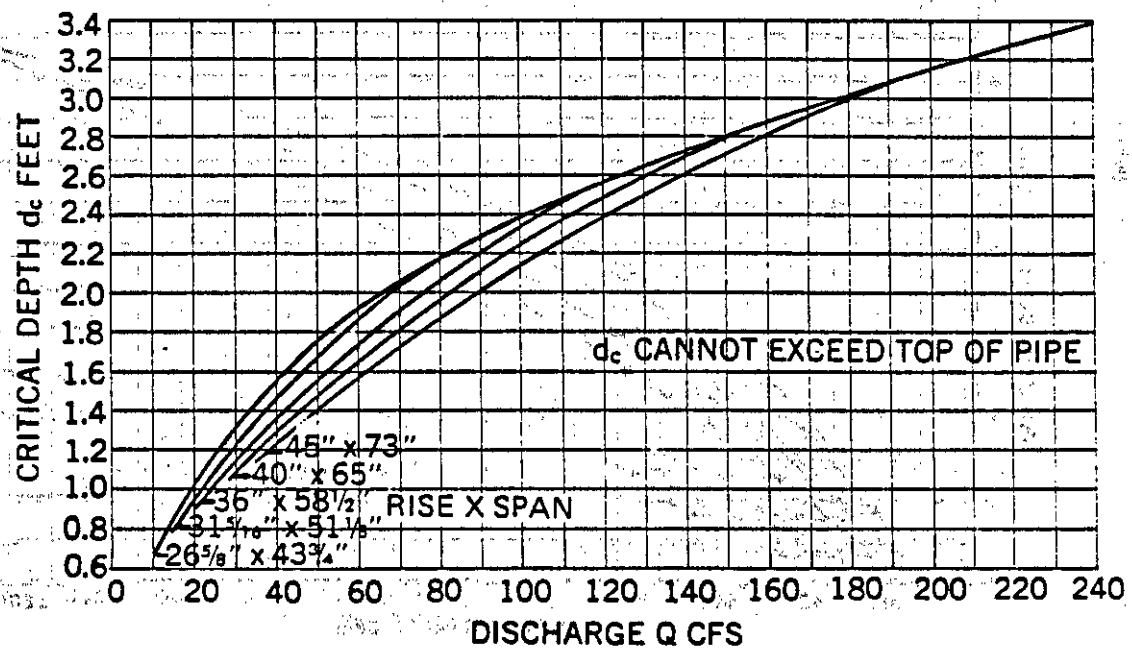


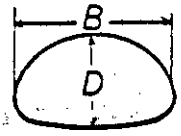


Reference: USDOT, FHWA, HDS-5 (1985).

FIGURE 6-24
Critical Depth Chart for Structural Plate CMP Arch

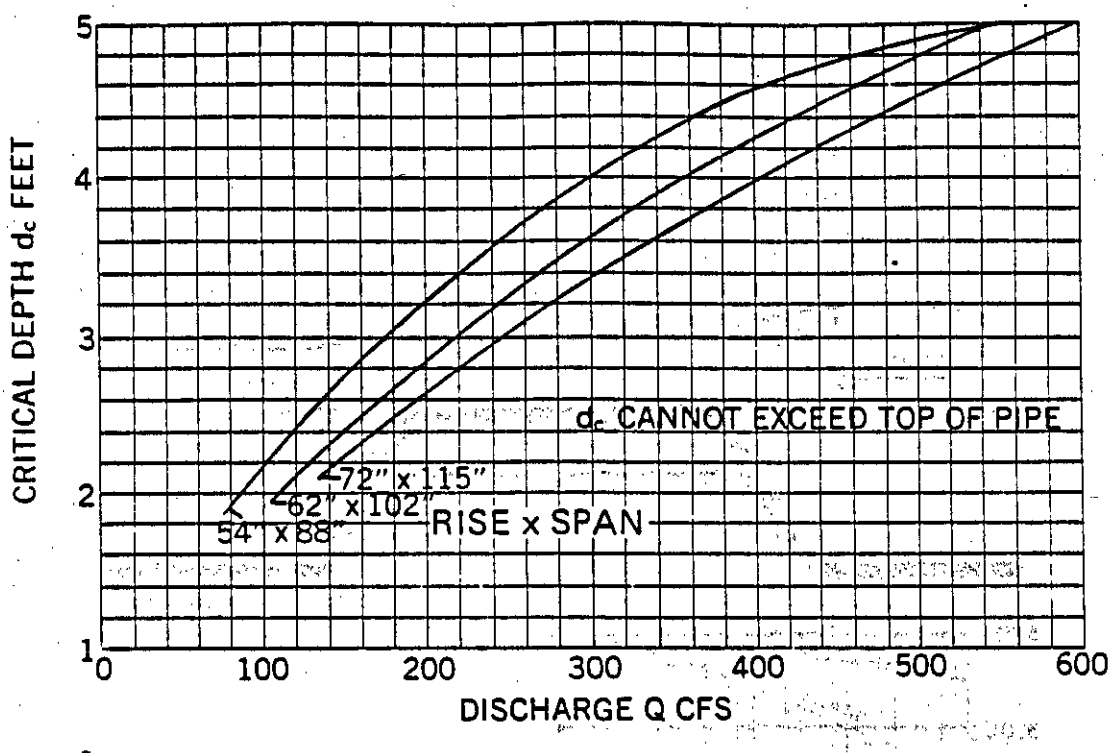


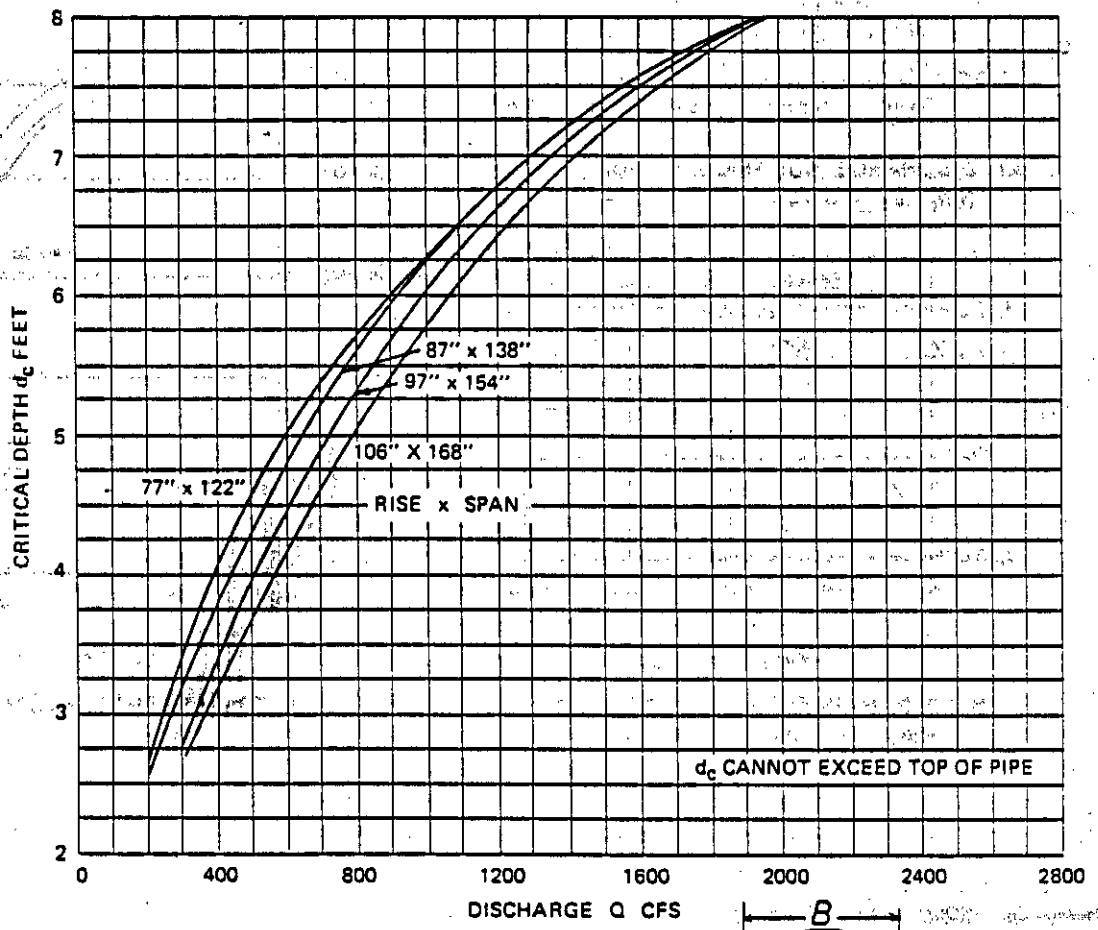


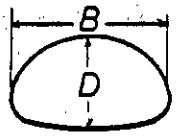


Reference: American Concrete Pipe Association (1980).

FIGURE 6-25
Critical Depth Chart for Arch Concrete Pipe
Sizes 15½" x 26" to 45" x 73"

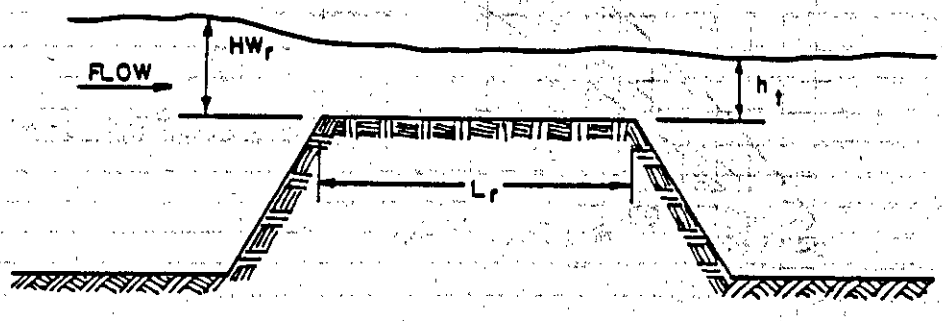


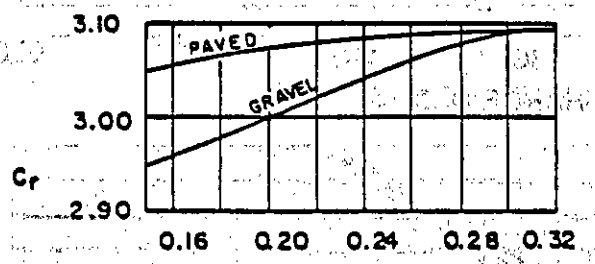




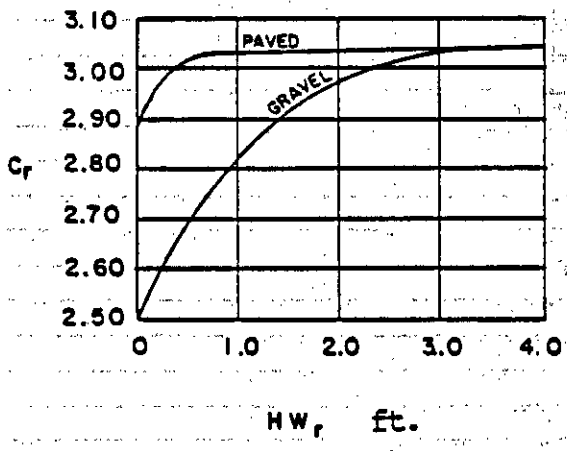
Reference: American Concrete Pipe Association (1980).

FIGURE 6-26
Critical Depth Chart for Arch Concrete Pipe
Sizes 54" x 88" to 106" x 168"

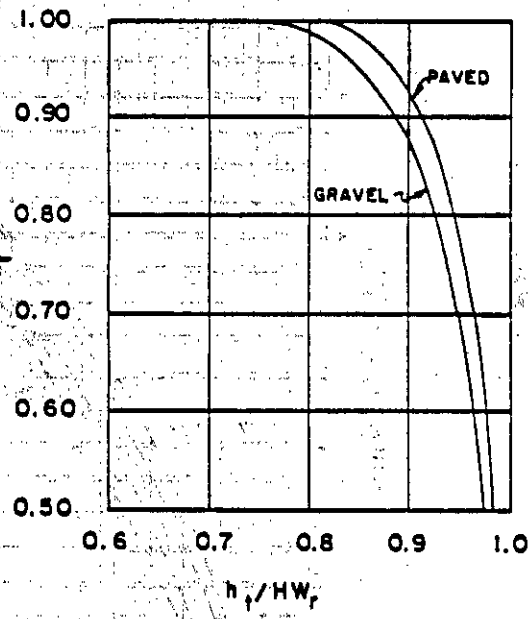




A) DISCHARGE COEFFICIENT FOR $HW_r / L_r > 0.15$



B) DISCHARGE COEFFICIENT FOR $HW_r / L_r \leq 0.15$



C) SUBMERGENCE FACTOR

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

FIGURE 6-27
Discharge Coefficients for Roadway Overtopping

CHAPTER 7
Gutter and Inlet Hydraulics

CHAPTER 7
Gutter and Inlet Hydraulics

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Chapter 7
GUTTER AND INLET HYDRAULICS

7.1 DESIGN CRITERIA

The following design criteria are factors to consider for gutter and inlet capacity calculations:

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

7.1.1 RETURN PERIOD

The design storm return period for pavement drainage shall be consistent with the value selected for other components of the drainage system, as follows:

Street Classification	Return Period
Major and Minor Arterials	25-year
Major Collectors	25-year
Minor Collectors	10-year
Residential and Local	10-year

Pavement flooding shall not exceed the spread specified in Section 7.1.2 for design storm conditions.

7.1.2 SPREAD

Spread is defined as the width of water transported on the pavement measured from the face of the curb. For roadways, a 10-foot spread shall not be exceeded during the design storm. Where streets are approved with a total lane width of less than 12 feet, an 8-foot maximum spread will be allowed.

7.1.3 MANNING'S n VALUES

Curb and gutter flow characteristics, including spread, shall be calculated using a minimum Manning's n value of 0.016. Higher design n values may be required by the Engineer of Public Works where small slope, unusual roughness, or sediment are present.

7.1.4 LONGITUDINAL SLOPE

Curb and gutter longitudinal slopes shall not exceed 12 percent or fall below 1/2 of 1 percent.

7.1.5 CROSS SLOPE

The design of pavement cross slope on residential streets shall conform to the standard roadway sections provided in the standard drawings (see Appendix 3, bound separately). Shoulders should generally be sloped to drain away from the pavement, except with raised, narrow medians.

7.1.6 CURB AND GUTTER SECTIONS

Concrete curb and gutter at the outside edge of pavements is normal practice for urban roadway facilities. Combination curb and gutter sections may be 2.0 or 2.5 feet wide with a 1-inch per foot cross slope for the gutter portion of the section consistent with the standard drawings (see Appendix 3, bound separately). Curb and gutter capacities for conveying pavement drainage to the stormwater inlets must be considered in the design of the roadway. Allowable gutter flow rates shall be based on a gutter cross slope of 0.5 inch per foot to account for possible debris and sediment accumulation.

7.1.7 INLET SPACING

Inlets shall be located or spaced so that at least 80 percent of the gutter flow is intercepted (i.e., no more than 20 percent of the gutter flow may bypass the inlet). The maximum length of longitudinal curb and gutter section without a Type S inlet shall be 300 feet.

No flow will be allowed to cross intersecting streets unless approved by Engineer of Public Works. Curb inlets will not be allowed within the radii at street intersections.

Yard inlets shall be designed to intercept the total design flow approaching the inlet. Standard grated structures may be used, but open-top structures will not be allowed.

7.1.8 ROADSIDE DITCHES

All new roadways will require curb and gutter to conform with the Subdivision Regulations of the City of Huntsville, unless curb and gutter installation is exempted by the Planning Commission. Where roadside ditches are approved along the roadway by the Engineer of Public Works, the design criteria shall conform to the standard roadway sections as shown in the standard drawings (see Appendix 3, bound separately). Where practicable, the flow from upgradient areas draining toward curbed highway pavements should be intercepted by ditches.

Roadside ditches can be used with uncurbed roadway sections to convey pavement runoff and upgradient area runoff that drains toward the pavement. 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. Roadside ditches shall be sized using open channel hydraulic procedures presented in Chapter 5. Roadside drainage structures shall be designed to minimize hazards to vehicles that leave the traveled roadway (see Chapter 3, Section 3.6).

7.1.9 BRIDGE DECKS

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect debris, and small inlets on scuppers have a higher potential for clogging than curb-opening inlets. Bridge deck constructibility usually requires a constant cross slope, so the guidelines in Section 7.1.5 do not apply. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways shall be intercepted before it reaches a bridge.

7.2 GUTTER FLOW CALCULATIONS

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

$$Q = \frac{0.56}{n} S_x^{5/3} S^{1/2} T^{8/3} \qquad (7\text{-}1)$$

where:

Q = Gutter flow rate, in cfs

n = Manning's roughness coefficient

S_x = Pavement cross slope, in feet/foot

S = Longitudinal slope, in feet/foot

T = Width of flow or spread, in feet

A nomograph for solving Equation 7-1 is presented in Figure 7-1. Manning's n values for various pavement surfaces are presented in Table 7-1.

7.2.1 UNIFORM CROSS SLOPES

The nomograph in Figure 7-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

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

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

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

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

Condition 2: Find gutter flow, given spread.

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

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

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

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

7.2.2 STANDARD GUTTER SECTIONS

Allowable capacity data for Huntsville standard curb and gutter for residential pavement sections are presented in Table 7-2. Table 7-2 provides allowable gutter capacities for 2.5-foot curb and gutter section using a residential pavement cross slope of 0.0256 foot/foot and a maximum spread of 10 feet. These capacity data shall also be used for 2.0-foot curb and gutter section. When the roadway or longitudinal slope is known, the allowable gutter capacity can be estimated directly from Table 7-2.

7.2.3 COMPOSITE GUTTER SECTIONS

Figure 7-2 can be used to find the flow in any composite gutter section with width, W, less than the total spread, T. The following steps are used to evaluate any composite gutter section:

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope, S, cross slope, S_x, depressed section slope, S_w, depressed section width, W, Manning's

n, gutter flow, Q, and a trial value of the gutter capacity above the depressed section, Q_s.

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

$$Q_w = Q - Q_s \qquad (7\text{-}2)$$

3. Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 7-2 to find an appropriate value of W/T.

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

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

6. Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 7-1 (see Section 7.2.1, Condition 2).

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

Condition 2: Find gutter flow, given spread.

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

2. Use Figure 7-1 to determine the capacity of the gutter section above the depressed section, Q_s. Use the procedure in Section 7.2.1, Condition 2, substituting T_s for T.

3. Calculate the ratios W/T and S_w/S_x, and, from Figure 7-2, find the appropriate value of E_o (the ratio of Q_w/Q).

4. Calculate the total gutter flow using the equation:

$$Q = Q_s/(1 - E_o) \quad (7\text{-}3)$$

where:

Q = Gutter flow rate, in cfs

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

E_o = Ratio of frontal flow to total gutter flow (Q_w/Q), obtained from Figure 7-3

5. Calculate the gutter flow in W, Q_w, with Equation 7-2.

7.3 CURB-OPENING INLETS

Curb-opening inlets are relatively free of clogging problems and offer little interference to traffic operation. They are preferable to grates in traffic lanes or where grates would be hazardous for pedestrians or bicyclists. Procedures to determine the capacity of curb-opening inlets placed on continuous grade and at sump locations are presented below.

7.3.1 CONTINUOUS GRADE--TYPE S

Table 7-3 provides allowable gutter flow, inlet intercept, and bypass estimates for the Huntsville 2.5-foot curb and gutter on residential streets and Type S curb-opening inlet on a continuous grade. The estimates are based on the typical sections presented in the standard drawings (see Appendix 3, bound separately), using the general procedure for curb-opening inlets presented below.

Up to roadway slopes of 6 percent, the Type S inlet has sufficient capacity to handle allowable gutter flow with a maximum spread of 10 feet and a bypass of 20 percent for the 2.5-foot curb and gutter section. Above this slope, the allowable gutter flow is controlled by a maximum of 20 percent bypass and the spread must be adjusted accordingly.

7.3.2 CONTINUOUS GRADE--GENERAL

The length of curb opening required for total interception of gutter flow can be estimated for general conditions using

Figure 7-3. The efficiency of inlets that are shorter than the length required for total interception can be evaluated using Figure 7-4. Steps for using these figures are given below:

1. Determine input parameters, including cross slope, S_x, longitudinal slope, S, Manning's n, spread, T, opening length, L, and gutter flow, Q (see Section 7.2).

2. Draw a line between the n and S scales on Figure 7-3 and note where it intersects the left turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate value on the S_x scale and note where it intersects the right turning line.

4. Draw a line between the intersection point from Step 3 and the appropriate value on the Q scale and obtain the value of curb opening length, L_T, required for total interception of gutter flow.

5. If L is less than L_T, find the ratio L/L_T and use Figure 7-4 to find the inlet efficiency, E.

6. Calculate the intercepted flow, Q_i, as the product of E and total gutter flow, Q.

7.3.3 SUMP CONDITIONS

A curb-opening inlet located in sump conditions operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height, h. At depths between 1.0 and 1.4 times h, the flow is in a transition stage. Figures 7-5 and 7-6 provide graphical solutions to the weir and orifice equations for depressed and non-depressed curb-opening inlets, respectively. Steps for using these figures are given below.

Depressed Conditions

1. Determine input parameters, including opening length, L, depressed opening height, h, cross

slope, S_x, depression, a, depression width, W, and gutter spread, T.

2. Determine the depth of gutter flow using the following equation:

$$d = TS_x \qquad (7\text{-}4)$$

where:

d = Depth of gutter flow, in feet

T = Gutter spread, in feet

S_x = Cross slope, in feet/foot

3. Calculate the depth of flow above the bottom lip of the depressed opening, d_i, using the following equation:

$$d_i = d + a/12 \qquad (7\text{-}5)$$

where:

d_i = Depth of flow above the bottom lip of the depressed opening, in feet

d = Depth of gutter flow, in feet, from Step 2

a = Depth of depression, in feet

4. If the value of d_i from Step 3 is less than or equal to the depressed opening height, h, use the weir flow curves at the bottom of Figure 7-5 to find inlet capacity, Q. Enter the y-axis with the value of d from Step 2, run a horizontal line to the right, and locate the intersection with the appropriate value of P = L + 1.8W. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

5. If the value of d_i from Step 3 is greater than or equal to 1.4 times the depressed opening height, h, use the orifice flow curves at the top of

Figure 7-5 to find the inlet capacity, Q. Enter the y-axis with the value of d_i from Step 3, run a horizontal line to the right, and locate the intersection with the appropriate values of A = hL and h. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

6. If the value of d_i from Step 3 is between 1.0 and 1.4 times the depressed opening height, h, transition flow exists. Draw a curve representing the transition between weir and orifice flow for the appropriate values of P = L + 1.8W and A = hL and obtain the capacity, Q, from the intersection of that curve with the value of d_i.

7. To find the depth required to handle a desired inlet capacity, Q, enter the x-axis of Figure 7-5 with the value of Q and use the appropriate curve to find the depth on the y-axis.

Undepressed Conditions

1. Determine input parameters, including opening length, L, undepressed opening height, h, cross slope, S_x, and gutter spread, T.

2. Determine the depth of gutter flow, d, using Equation 7-4.

3. If the value of d from Step 2 is less than or equal to the undepressed opening height, h, use the weir flow curves at the bottom of Figure 7-6 to find inlet capacity, Q. Enter the y-axis with the value of d, run a horizontal line to the right, and locate the intersection with the appropriate value of L. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

4. If the value of d from Step 2 is greater than or equal to 1.4 times the undepressed opening height, h, use the orifice flow curves at the top of Figure 7-6 to find inlet capacity, Q. Enter the y-axis with the value of d, run a horizontal line

to the right, and locate the intersection with the appropriate value of A = hL and h. From that point of intersection, run a vertical line downward to obtain the inlet capacity, Q.

5. If the value of d from Step 2 is between 1.0 and 1.4 times the undepressed opening height, h, transition flow exists. Draw a curve representing the transition between weir and orifice flow for the appropriate values of L and A and obtain the capacity, Q, from the intersection of that transition curve with the value of d.

6. To find the depth required to handle a desired inlet capacity, Q, enter the x-axis of Figure 7-6 with the value of Q and use the appropriate curve to find the depth on the y-axis.

7.4 GRATE INLETS

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

Procedures to determine the capacity of grate inlets placed on continuous grade and at sump locations are available from HEC-12 (USDOT, FHWA, 1984).

7.5 COMBINATION INLETS

7.5.1 CONTINUOUS GRADE

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. A combination inlet with the curb opening located upstream of the grate has an interception capacity equal to the sum of the two inlets, except that the frontal flow of the grate is reduced by the amount of flow

intercepted by the curb opening. By placing the curb opening upstream, debris can generally be intercepted before it clogs the grate.

7.5.2 SUMP LOCATIONS

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

Table 7-1
MANNING'S n VALUES FOR STREET AND PAVEMENT GUTTERS

Type of Gutter or Pavement	Manning's n
Standard Huntsville design value (see Section 7.1.3)	0.016
Concrete gutter, troweled finish	0.012
Asphalt pavement	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement	
Smooth	0.013
Rough	0.015
Concrete pavement	
Float finish	0.014
Broom finish	0.016
For gutters where sediment may accumulate, increase values of n by	0.002

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

Table 7-2
HYDRAULIC CAPACITIES OF HUNTSVILLE STANDARD 2.5-FOOT CURB AND GUTTER FOR RESIDENTIAL PAVEMENT SECTIONS

Roadway Slope (%)	Allowable Flow (cfs)	Roadway Slope (%)	Allowable Flow (cfs)
0.50	2.68	6.50	9.66
0.75	3.28	6.75	9.85
1.00	3.79	7.00	10.03
1.25	4.24	7.25	10.20
1.50	4.64	7.50	10.38
1.75	5.01	7.75	10.55
2.00	5.36	8.00	10.72
2.25	5.69	8.25	10.89
2.50	5.99	8.50	11.05
2.75	6.29	8.75	11.21
3.00	6.56	9.00	11.37
3.25	6.83	9.25	11.53
3.50	7.09	9.50	11.68
3.75	7.34	9.75	11.83
4.00	7.58	10.00	11.99
4.25	7.81	10.25	12.13
4.50	8.04	10.50	12.28
4.75	8.26	10.75	12.43
5.00	8.47	11.00	12.57
5.25	8.68	11.25	12.71
5.50	8.89	11.50	12.85
5.75	9.09	11.75	12.99
6.00	9.28	12.00	13.13
6.25	9.48		

NOTES:

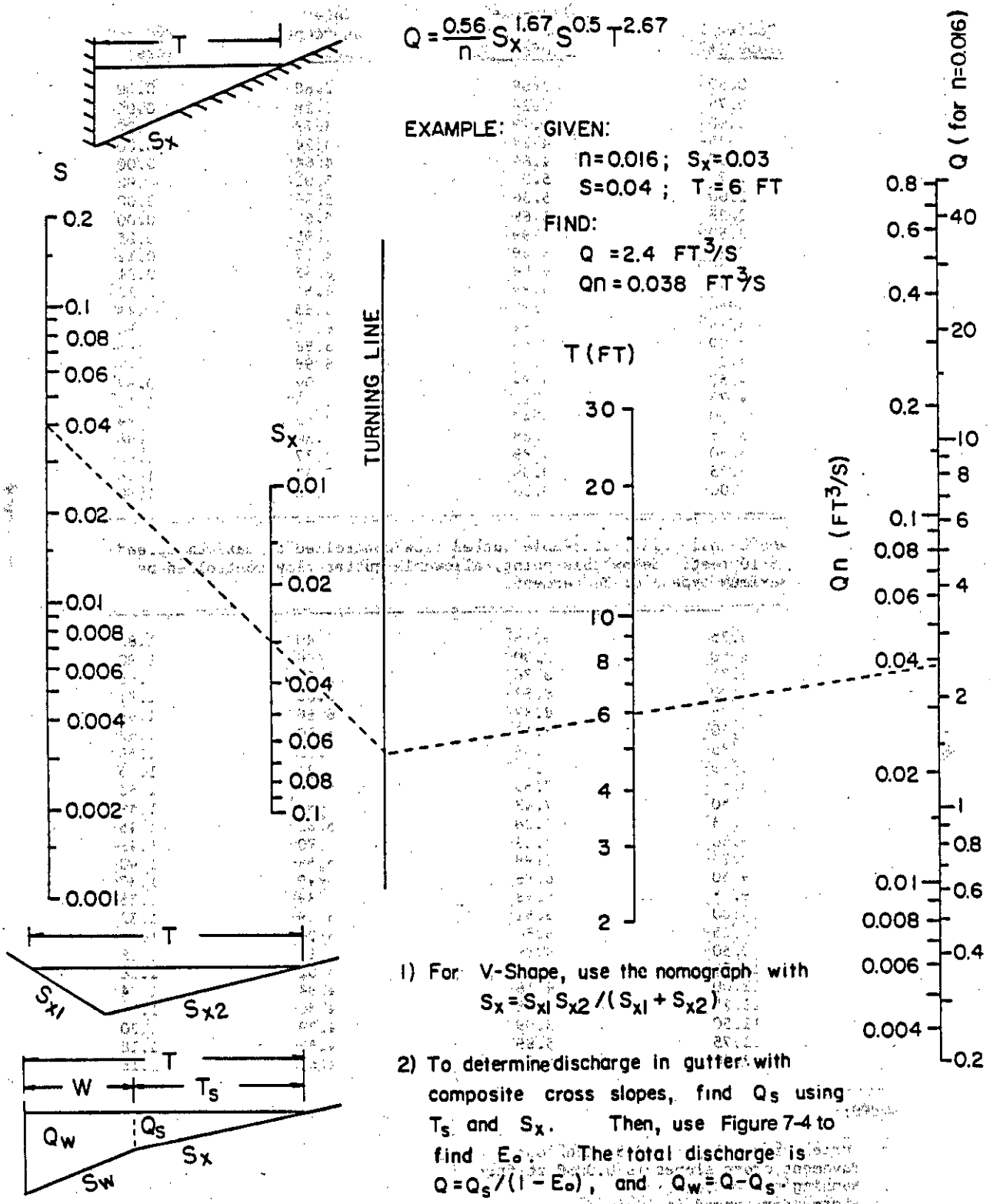
1. These capacity data shall also be used for 2.0-foot curb and gutter sections.
2. Gutter cross slope for allowable flows is based on 0.5 inch/foot instead of the 1 inch/foot required for construction.
3. Pavement cross slope is 0.0256 ft/ft.
4. Manning's n = 0.016.
5. Stormwater spread is 10 feet.
6. The maximum allowable flow equation for the above conditions is $Q = 37.9\ S^{0.5}$.

Table 7-3
INTERCEPT OF TYPE S INLET WITH MAXIMUM ALLOWABLE GUTTER FLOW
FOR RESIDENTIAL STREETS AND 2.5-FOOT CURB AND GUTTER

Roadway Slope (%)	Allowable Gutter Flow (cfs)	Inlet Intercept (cfs)	Bypass (cfs)
0.50	2.68	2.68	0.00
0.75	3.28	3.28	0.00
1.00	3.79	3.79	0.00
1.25	4.24	4.24	0.00
1.50	4.64	4.64	0.00
1.75	5.01	5.01	0.00
2.00	5.36	5.36	0.00
2.25	5.69	5.68	0.00
2.50	5.99	5.95	0.05
2.75	6.29	6.17	0.12
3.00	6.56	6.35	0.21
3.25	6.83	6.51	0.32
3.50	7.09	6.65	0.44
3.75	7.34	6.77	0.56
4.00	7.58	6.88	0.70
4.25	7.81	6.98	0.83
4.50	8.04	7.07	0.97
4.75	8.26	7.16	1.10
5.00	8.47	7.23	1.24
5.25	8.68	7.30	1.38
5.50	8.89	7.37	1.52
5.75	9.09	7.42	1.66
6.00	9.28	7.48	1.80
Above this point, allowable gutter flow controlled by maximum spread of 10 feet. Below this point, allowable gutter flow controlled by maximum bypass of 20 percent.			
6.25	9.25	7.40	1.85
6.50	9.00	7.20	1.80
6.75	8.76	7.01	1.75
7.00	8.53	6.83	1.71
7.25	8.32	6.66	1.66
7.50	8.12	6.50	1.62
7.75	7.93	6.35	1.59
8.00	7.76	6.21	1.55
8.25	7.59	6.07	1.52
8.50	7.43	5.94	1.49
8.75	7.28	5.82	1.45
9.00	7.13	5.70	1.43
9.25	6.99	5.59	1.40
9.50	6.86	5.49	1.37
9.75	6.73	5.39	1.35
10.00	6.61	5.29	1.32
10.25	6.50	5.20	1.30
10.50	6.39	5.11	1.28
10.75	6.28	5.02	1.26
11.00	6.18	4.94	1.24
11.25	6.08	4.86	1.22
11.50	5.99	4.79	1.20
11.75	5.89	4.72	1.18
12.00	5.81	4.65	1.16

NOTES:

1. Gutter cross slope is 0.5 inch/foot.
2. Pavement cross slopes is 0.0256 ft/ft.
3. Manning's n = 0.016.
4. Stormwater spread is 10 feet.
5. See the standard drawings for details on curb and gutter and residential street sections.

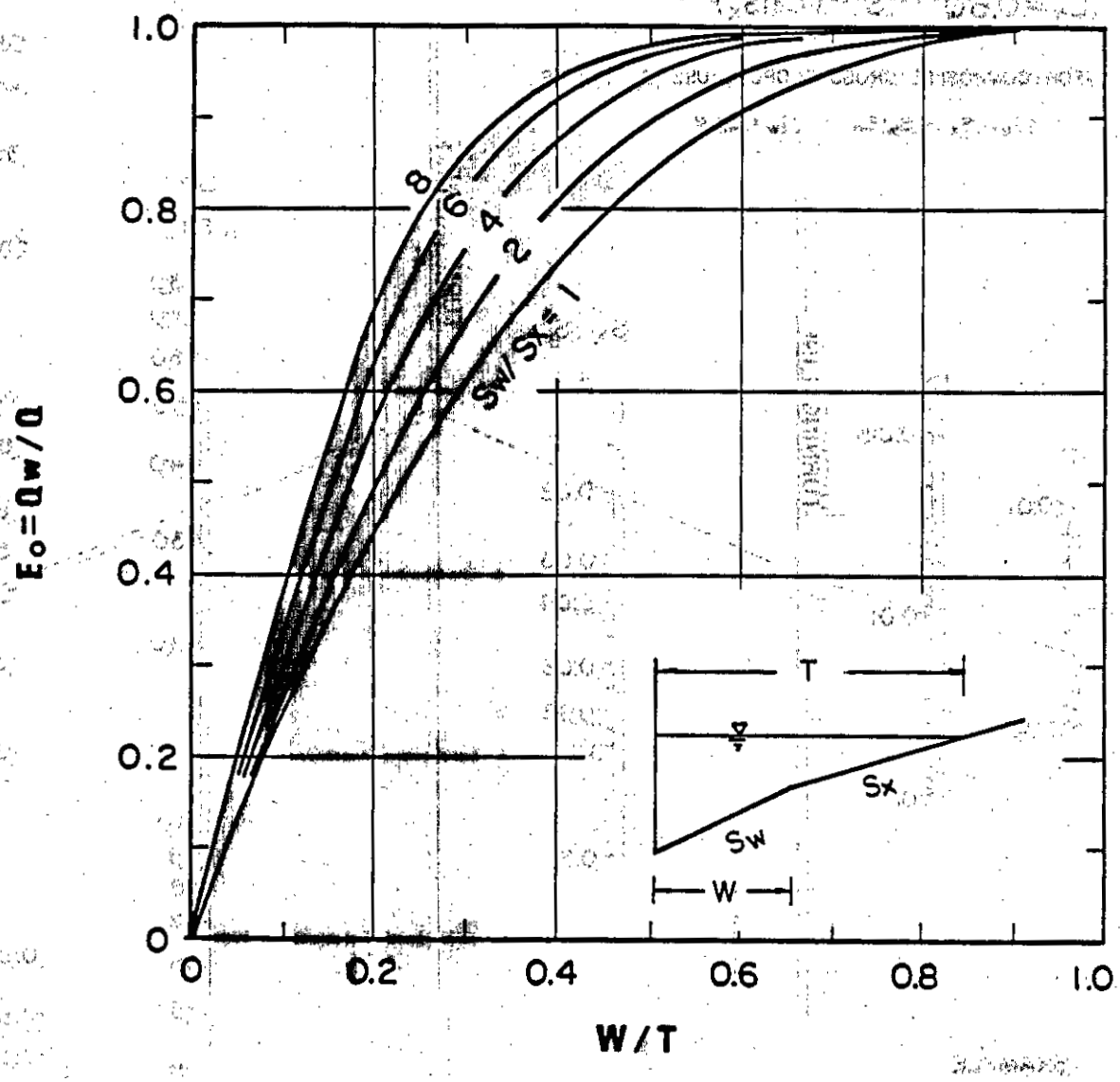


1) For V-Shape, use the nomograph with $S_x = S_{x1} S_{x2} / (S_{x1} + S_{x2})$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use Figure 7-4 to find E_o. The total discharge is $Q = Q_s / (1 - E_o)$, and $Q_w = Q - Q_s$.

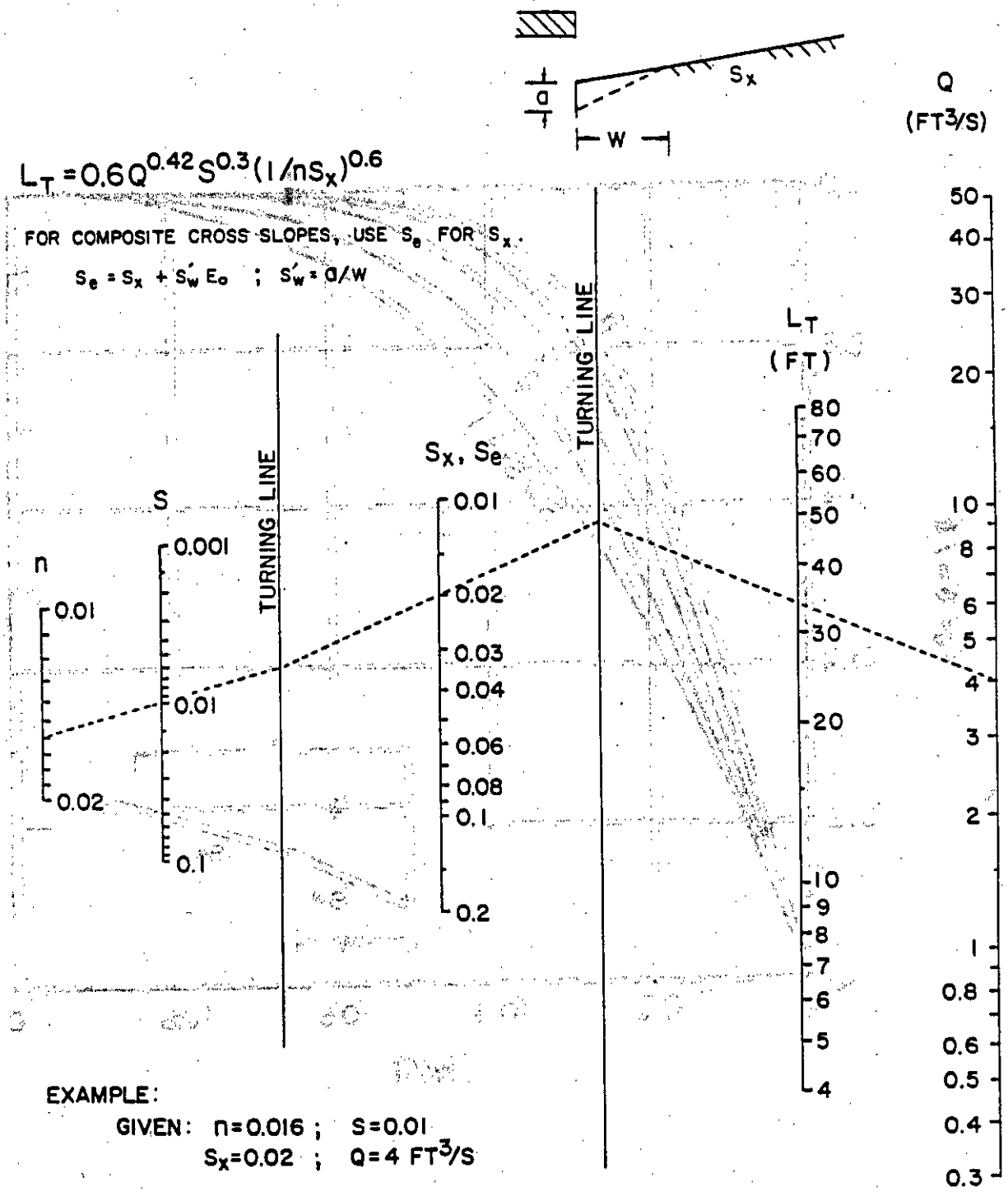
Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 7-1
Nomograph for Flow in Triangular Gutter Sections



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

FIGURE 7-2
Ratio of Frontal Flow to Total Gutter Flow



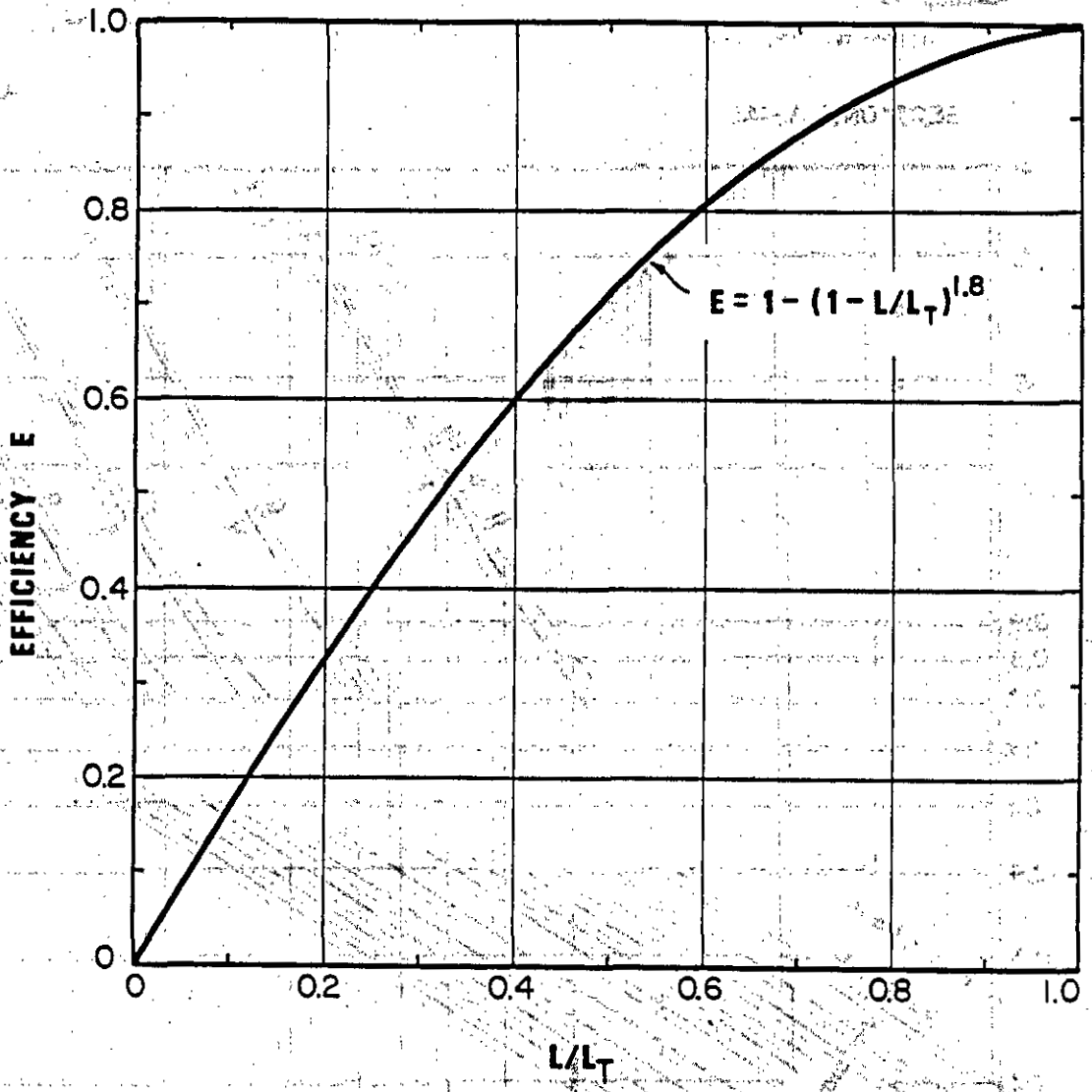
EXAMPLE:

GIVEN: $n = 0.016$; $S = 0.01$
$S_x = 0.02$; $Q = 4\ FT^3/S$

FIND: $L_T = 34$ FT

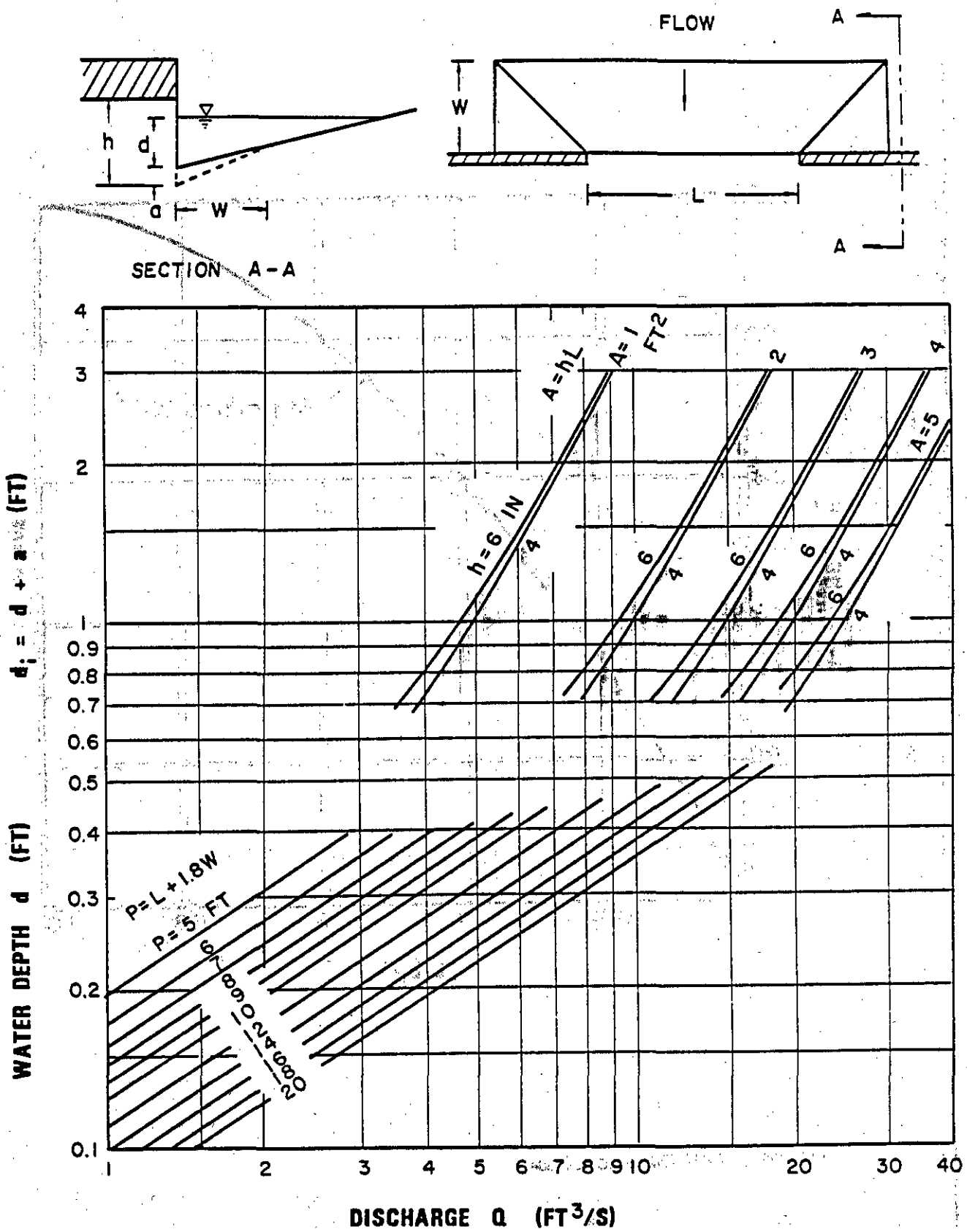
Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 7-3
Curb-Opening and Slotted Pipe Inlet Length for Total Interception



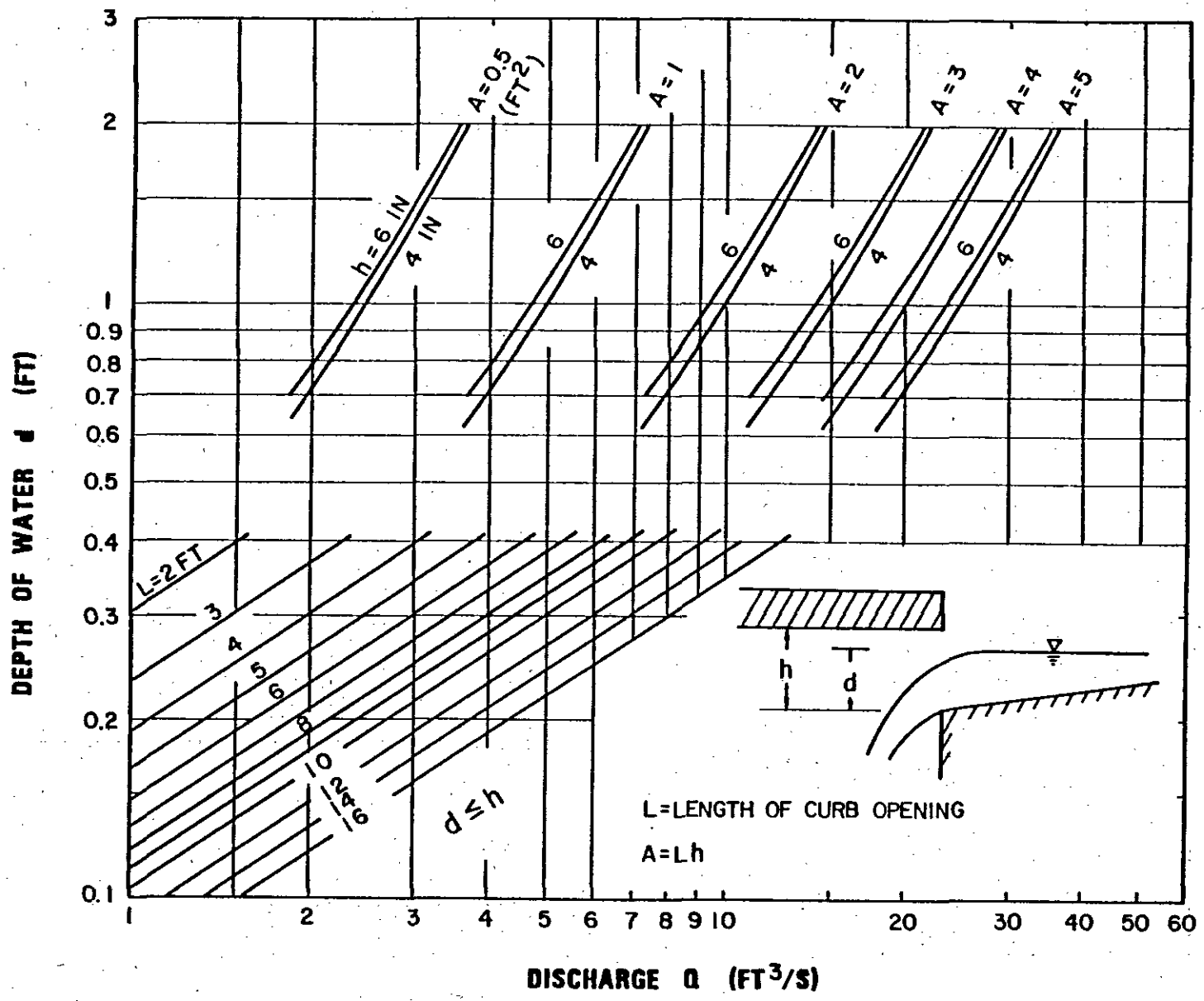
Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 7-4
Curb-Opening and Slotted Pipe Inlet Interception Efficiency



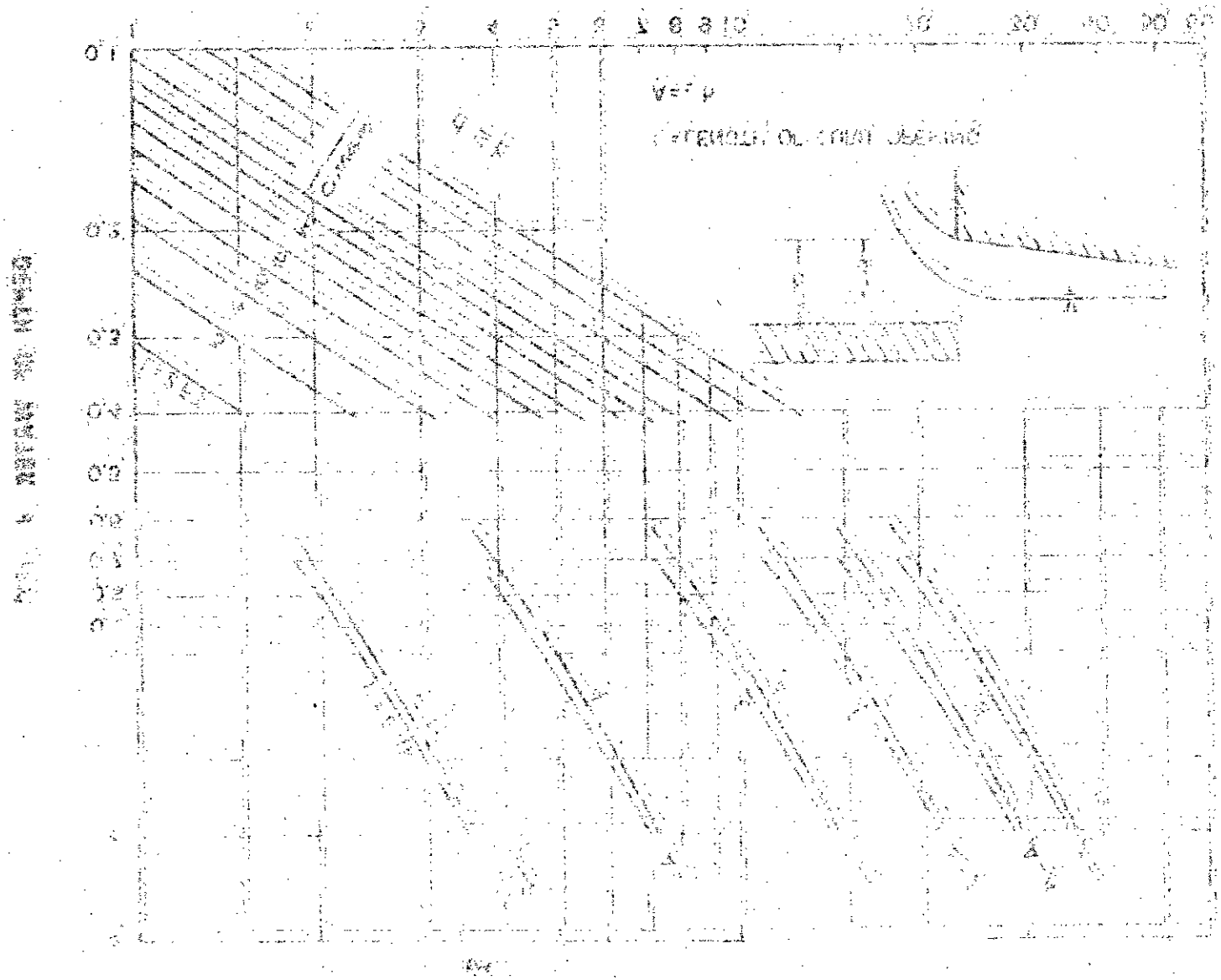
Reference: USDOT, FHWA, HEC-12 (1984).

FIGURE 7-5
Depressed Curb-Opening Inlet Capacity in Sump Locations



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

FIGURE 7-6
Undepressed Curb-Opening Inlet Capacity in Sump Locations



CHAPTER 8
Storm Sewer Hydraulics

CHAPTER 8
Storm Sewer Hydraulics

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Chapter 8
STORM SEWER HYDRAULICS

8.1 DESIGN CRITERIA

8.1.1 RETURN PERIODS

The following return periods shall be used for storm sewer design:

Street Classification	Return Period
Major and Minor Arterials	25-year
Major Collectors	25-year
Minor Collectors	10-year
Residential and Local	10-year

The impact of a 100-year return period flood shall be analyzed as directed by the Engineer of Public Works.

8.1.2 MANNING'S n VALUES

All storm sewer pipe shall be reinforced concrete unless alternate materials of equal or greater effectiveness and strength are approved by the Engineer of Public Works. Values for Manning's roughness coefficient for concrete pipe, concrete box culvert, and corrugated metal pipe (CMP) are given below:

Concrete pipe	n = 0.012
Box culvert (cast-in-place)	n = 0.013
CMP (non-spiral flow, annular corrugations)	n = 0.024
CMP (full pipe spiral flow, helical corrugations)	
Sizes 15-24"	n = 0.017
Sizes 30-54"	n = 0.021
Sizes 60-96"+	n = 0.024

Full spiral flow occurs only for pipes with a length 20 times diameter, operating under full flow, and free of sediment buildup. Conditions where full spiral flow would be appropriate are down drains, retention outlet pipes, and free outlet or gravity storm sewer systems with a design velocity above 4 feet per second.

8.1.3 SLOPES AND HYDRAULIC GRADIENT

The maximum and minimum slopes for storm sewers shall conform to the following criteria:

1. The maximum hydraulic gradient shall not produce a velocity that exceeds 20 feet per second. Higher velocities require approval from the Engineer of Public Works.

2. The minimum desirable physical slope shall be that which will produce a velocity of 2.5 feet per second when the storm sewer is flowing full.

When hydraulic calculations do not consider minor energy losses, the elevation of the hydraulic gradient for design flood conditions shall be at least 1 foot below the gutter elevation. Minor losses shall be con-sidered when the velocity exceeds 6 feet per second. If all minor energy losses are calculated, it is accept-able for the hydraulic gradient to reach the gutter elevation.

8.1.4 PIPE SIZE AND LENGTH

The minimum pipe size shall be 15 inches when access spacing is 50 feet or less. When access spacing exceeds 50 feet, the minimum pipe size shall be 18 inches. Standard pipe size increments of 6 inches shall be used for pipes larger than 18 inches.

The minimum box culvert size shall be 3 by 3 feet for precast units and 4 by 4 feet for cast-in-place units. Increments of 1 foot in the height or width shall be used above this minimum. The span by height format is used for reporting box culvert dimensions, e.g., in the dimension 10 by 7 feet, the span is 10 feet and the height is 7 feet.

Access spacing shall not exceed 400 feet for conduits less than 54 inches in diameter and shall not exceed 800 feet under any circumstances.

8.1.5 MINIMUM CLEARANCES

Minimum clearances for storm sewer pipe shall comply with the following criteria:

1. A minimum of 1 foot is required between the bottom of the road base material and the outside crown of the storm sewer, with 15 to 18 inches preferred.

2. For utility conflicts that involve crossing a storm sewer alignment, the recommended minimum design clearance between the outside of the pipe and the outside of any conflicting utility should be 0.5 foot if the utility has been accurately located at the point of conflict. If the utility has been approximately located, the minimum design clear-ance should be 1 foot. Electrical transmission lines or gas mains should never come into direct contact with the storm sewer.

3. Storm sewer systems should not be placed parallel to or below existing utilities in a manner that could cause utility support problems. Recommended clearance is 2 feet extending from each side of the storm sewer and 1:1 side slopes from the trench bottom. Where right-of-way limitations are inadequate, shoring of trenches is acceptable.

4. When a sanitary line or other utility must pass through a manhole, a minimum 1-foot clearance should be maintained between the bottom of the utility and the flow line of the storm main, and greater clearance is recommended. Flow will be less obstructed when the utility is placed above or as close as possible to the crown of the pipe. The head loss caused by an obstruction should be accounted for. (Note: Gas mains shall not pass through inlet and manhole structures.)

8.1.6 INLET LOCATION AND SPACING

Curb inlets shall be located to facilitate the entrance of water from gutters into the storm sewer system. Consideration shall also be given to the movement of vehicles to and from adjacent property on turnouts and to the maintenance of safe pedestrian walkways. Curb inlets shall not be located within handicap ramps and shall be avoided within a 35-foot radius of turns.

The location and spacing of inlets shall be based on inlet capacity and width of spread calculations. Surface storm water shall not be allowed to drain across street intersections. Procedures in Chapter 7 shall be used to establish inlet spacing such that a maximum of 20 percent bypass is allowed. The last inlet shall have no bypass.

8.2 GENERAL PROCEDURE

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 8.1.

2. Hydrologic Calculations: Determination of design flow rates and volumes (see Section 8.3).

3. Hydraulic Calculations: Determination of pipe sizes required to carry design flow rates and volumes, as discussed in Section 8.4.

4. Outfall Design: Outlet protection or detention/retention may be required because of downstream constraints; see Chapter 9 for detention/retention or Chapter 10 for outlet protection.

8.3 HYDROLOGIC CALCULATIONS

The hydrologic determination of a peak runoff rate for sizing a storm sewer system shall be made using procedures presented in Chapter 4. In general, storm sewer systems are sized to carry storm-water intercepted by appropriate inlet facilities. However, if the intercepted runoff is transported through an exten-sive pipe network, channel storage within the storm sewers can modify the peak rate of the runoff as it travels along the system. The peak flow

modification can be evaluated with hydrologic channel routing procedures (see Chapter 4).

For many projects, the Rational Method is well suited to performing hydrologic calculations for storm sewer systems. In general, as the time of concentration, drainage area, and variability in land use increase, more complex procedures are warranted. A rule-of-thumb is that flood hydrograph procedures should be considered when the time of concentration goes above 30 to 45 minutes. In addition, the size and complexity of the storm sewer system should be considered. (See Chapter 4 for additional guidance on selecting hydrologic methods.)

To apply the Rational Method at each design point, the following data are required:

1. Tributary area
2. Time of concentration
3. Rainfall intensity
4. Runoff coefficient

The time of concentration is the sum of the inlet travel time and the storm sewer travel time and must be calculated for each design point considered. Rainfall intensity is obtained from an IDF curve (see Chapter 4), based on the time of concentration and design return period. The runoff coefficient should be the composite factor based on tributary land use and soil conditions. The tabular data presented in Chapter 4 can provide a good basis for selecting the runoff coefficient, but if possible, a comparison of historical performance with the results of design calculations should be made.

The Rational Method as presented in Chapter 4 implicitly assumes that all runoff from the tributary area is intercepted by the storm sewer system. Therefore, because inlet efficiencies of 100 percent are unrealistic, bypass must be accounted for by adjusting the tributary drainage area.

The proper selection of the time of concentration indirectly accounts for situations in which peak runoff arrives at individual inlets at different points in time. However, this approach does not explicitly account for channel

storage that can become important in the upstream portions of a large storm sewer system.

In some cases, only a portion of the area draining to a particular location in a storm sewer system will cause the peak discharge. In systems where a small subbasin of the total area has an unusually long travel time, another larger subbasin of the contributing area with a shorter time of concentration (and, therefore, a higher average rainfall intensity) can cause a higher peak discharge.

8.4 HYDRAULIC CALCULATIONS

Hydraulic calculations are used to size pipes to handle the design flows determined from hydrologic calculations (Section 8.3). The hydraulic capacity of a storm sewer pipe can be calculated for the two types of conditions typically referred to as open channel and pressure flow. Results should provide a balanced system in which all segments will be used to their full capacity consistent with the flood protection criteria for the project site.

Hydraulic procedures 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), the American Iron and Steel Institute (1980), and Marsalek (1985). These publications should be consulted if additional details are required.

8.4.1 PRESSURE VERSUS OPEN CHANNEL FLOW

Open channel flow occurs when a free water surface is exposed to the atmosphere as a boundary (see Figure 8-1). When the conduit is flowing full, the pipe is considered to be flowing under pressure (see Figure 8-2).

Guidance is presented in Figure 8-3 for determining whether pressure or open channel flow conditions occur in a storm sewer system. In general, if the hydraulic grade line is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the hydraulic grade line is below the crown of the pipe, open channel flow calculations are appropriate.

A special concern with storm sewers designed to operate under pressure flow conditions is that inlet surcharging and possible manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel 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.

As hydraulic calculations are performed, existence of the desired flow condition should be verified. Storm sewer systems can alternate between pressure and open channel flow conditions from one section to another.

The discharge point of the storm sewer system usually establishes a starting point for evaluating flow conditions. If the discharge is submerged, as when the tailwater established by the receiving waterbody is above the crown of the storm sewer, the exit loss should be added to the tailwater and calculations for head loss in the storm sewer system started from this point, as depicted in the submerged discharge conditions of Figure 8-3. 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 open channel flow calculations should be used at the upstream manhole.

When the discharge point is not submerged, a flow depth should be determined at some control section to allow calculations to proceed upstream. As shown in Figure 8-3, the hydraulic grade line is then projected 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 illustrated in Figure 8-3 is not entirely correct, because backwater and drawdown conditions can exist, but is generally reasonable for typical pipe sizes. 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 will have a very significant effect on design, backwater and drawdown curves should be developed.

8.4.2 ENERGY LOSSES

The following energy losses shall 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:

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

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

Head losses due to pipe form conditions are generally calculated as:

$$H_L = K\left(\frac{v^2}{2g}\right) \qquad (8\text{-}2)$$

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 system being considered. Values for the entrance loss coefficient are the same as those developed for culverts (see Chapter 6). The exit loss coefficient is generally assigned a value of 1. Expansion and contraction loss coefficients for circular pipes can be selected based on the data from Brater and King (1976) presented in Tables 8-1 and 8-2.

The bend loss coefficient for storm sewer systems can be evaluated using Figure 8-4, 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 of 2 and greater than or equal to 6 (r = radius of bend and D = pipe diameter).

Junction and Manhole Losses

If losses associated with junctions and manholes are evaluated, procedures presented in a report by the University of Missouri (1958) or Marsalek (1985) shall be used. Although details of the procedures are not duplicated, the implications of laboratory test results are discussed below and head loss coefficients for typical manholes and junctions are presented in Table 8-3.

For straight flow-through conditions, the 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 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, because the jets could impinge upon each other. If directly opposing laterals are necessary, the installation of a deflector (as shown in Figure 8-5) will significantly reduce losses. The research conducted on this type of deflector is limited to the ratios of outlet pipe to lateral pipe diameters equal to 1.25. In addition, lateral pipes should be located such that their centerlines are separated laterally by at least the sum of the two lateral pipe diameters.

Figure 8-5 depicts several types of deflectors that can be efficient in reducing losses at junctions and bends for full flow conditions. As a contrast, several inefficient manhole shapes are shown in Figure 8-6. Several of these inefficient devices would appear to be improvements, indicating that special shapings deviating from those in Figure 8-5 should be used with caution.

8.4.3 OPEN CHANNEL FLOW

Under non-pressure conditions, the capacity of a closed conduit can be analyzed by applying Manning's Equation to evaluate frictional losses for uniform flow. As shown in Figure 8-1, the hydraulic grade line is the free water surface elevation and is parallel to the energy grade line under uniform flow conditions. For circular conduits flowing full, Manning's Equation can be expressed as:

$$v = \frac{0.592}{n} d_s^{2/3} S_o^{1/2} \qquad (8\text{-}3)$$

or

$$Q = \frac{0.465}{n} d_s^{8/3} S_o^{1/2} \qquad (8\text{-}4)$$

where:

v = Full flow velocity, in feet/second

Q = Design discharge, in cfs

n = Manning's roughness coefficient

d_s = Diameter of the circular conduit, in feet

S_o = Pipe slope, in feet/foot

Non-circular and non-full flow conditions can be evaluated using the standard form of Manning's Equation discussed in Chapter 5.

Given the appropriate peak runoff rate for the design point in question, the conduit is sized to carry this peak rate as an open channel, using Manning's Equation. For a condition in which pressure is allowed to develop in storm sewers with a design based on open channel flow conditions, the design capacity of the system will be greater than that determined using Equation 8-4, and can be evaluated as discussed below.

8.4.4 PRESSURE FLOW

If the hydraulic grade line, as illustrated in Figure 8-2, can be increased above the crown of the pipe, pressure flow occurs. The capacity of storm sewers designed to operate under pressure flow conditions can be sized using inlet and outlet control nomographs developed for the evaluation of culverts (see Chapter 6). A more general procedure involves the application of the energy and continuity equations, which can be developed to consider unsteady flow conditions.

The energy equation between upstream and downstream locations can be evaluated by considering velocity head, pipe form, and friction losses, expressed as:

$$H = H_v + H_L + H_f \qquad (8\text{-}5)$$

or

$$H = \left[1 + K_L + \frac{29\ n^2\ L}{R^{1.33}}\right] \frac{v^2}{2g} \qquad (8\text{-}6)$$

where:

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, in feet

H_v = Velocity head, in feet

H_L = Head loss due to pipe form conditions, in feet

H_f = Head loss due to friction, in feet

K_L = Loss coefficient for pipe form losses

n = Manning's roughness coefficient

L = Length of storm sewer segment, in feet

R = Hydraulic radius, in feet

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second²

If H can be determined, the storm sewer capacity is calculated by rearranging Equation 8-6 as follows:

$$v = \left[2g\,H \div \left(1 + K_L + \frac{29\,n^2\,L}{R^{1.33}}\right)\right]^{1/2} \qquad (8\text{-}7)$$

or

$$Q = A\left[2g\,H \div \left(1 + K_L + \frac{29\,n^2\,L}{R^{1.33}}\right)\right]^{1/2} \qquad (8\text{-}8)$$

where:

v = Average velocity of flow, in feet/second

Q = Storm sewer capacity, in cfs

g = Acceleration due to gravity, 32.2 feet/second²

H = Head, determined as the difference between the hydraulic grade line at the downstream pipe and the energy grade line at the upstream pipe, in feet

K_L = Loss coefficient for pipe form losses

n = Manning's roughness coefficient

L = Length of storm sewer segment, in feet

R = Hydraulic radius, in feet

The determination of H will generally involve an evaluation of energy losses, to establish the hydraulic and energy gradients. Because the velocity is a required input to energy loss calculations, an iterative trial and error procedure is generally required.

Table 8-1
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

v_1 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_1, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Reference: Brater and King (1976).

Table 8-2
VALUES OF K_3 FOR DETERMINING LOSS OF HEAD DUE TO SUDDEN CONTRACTION IN PIPES, FROM THE FORMULA

$$H_3 = K_3(v_2^2/2g)$$

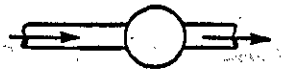
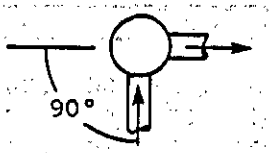
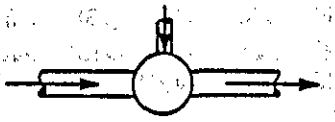
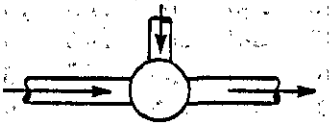
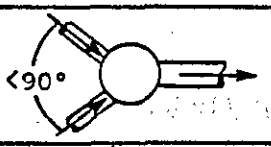
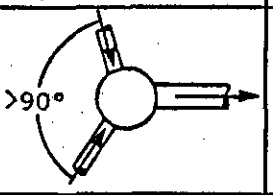
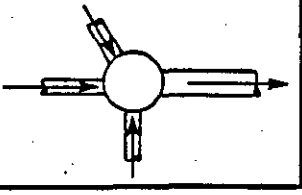
d_2/d_1 = Ratio of larger to smaller diameter

v_2 = Velocity in smaller pipe

$\frac{d_2}{d_1}$	Velocity, v_2, in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04.	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41.	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Reference: Brater and King (1976).

Table 8-3
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

$$hj(ft) = K \frac{[v(ft/sec)]^2}{64.4}$$

with v = larger velocity in main entrance or exit line of junction/manhole.

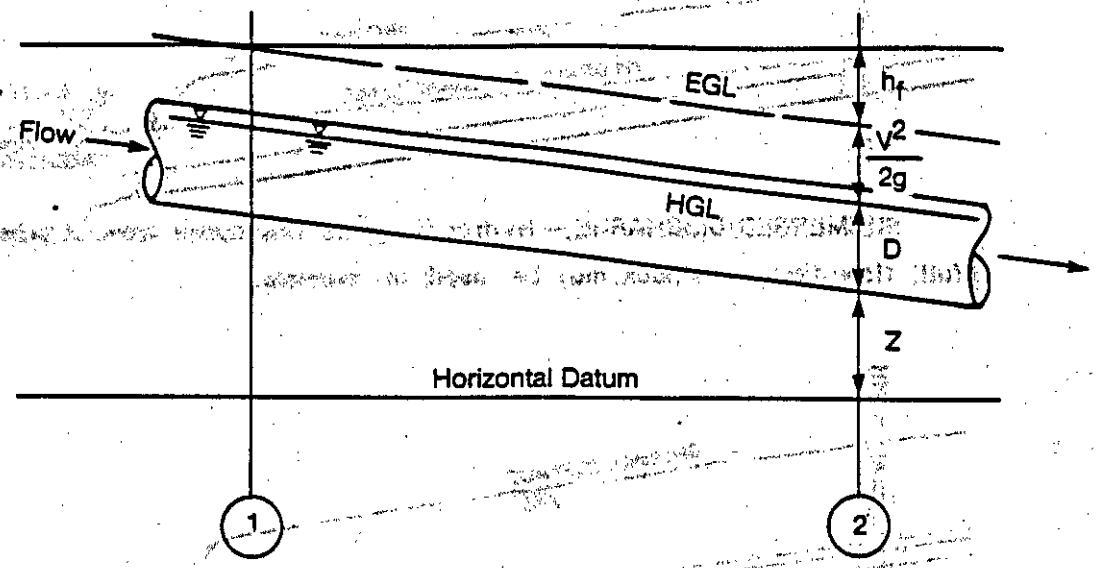


FIGURE 8-1
Open Channel Flow in a Closed Conduit

Z = Distance above horizontal datum
D = Depth of flow
P = Pressure head
$\frac{V^2}{2g}$ = Velocity Head
h_f = Friction loss between Section 1 and Section 2
EGL = Energy Grade Line
HGL = Hydraulic Grade Line

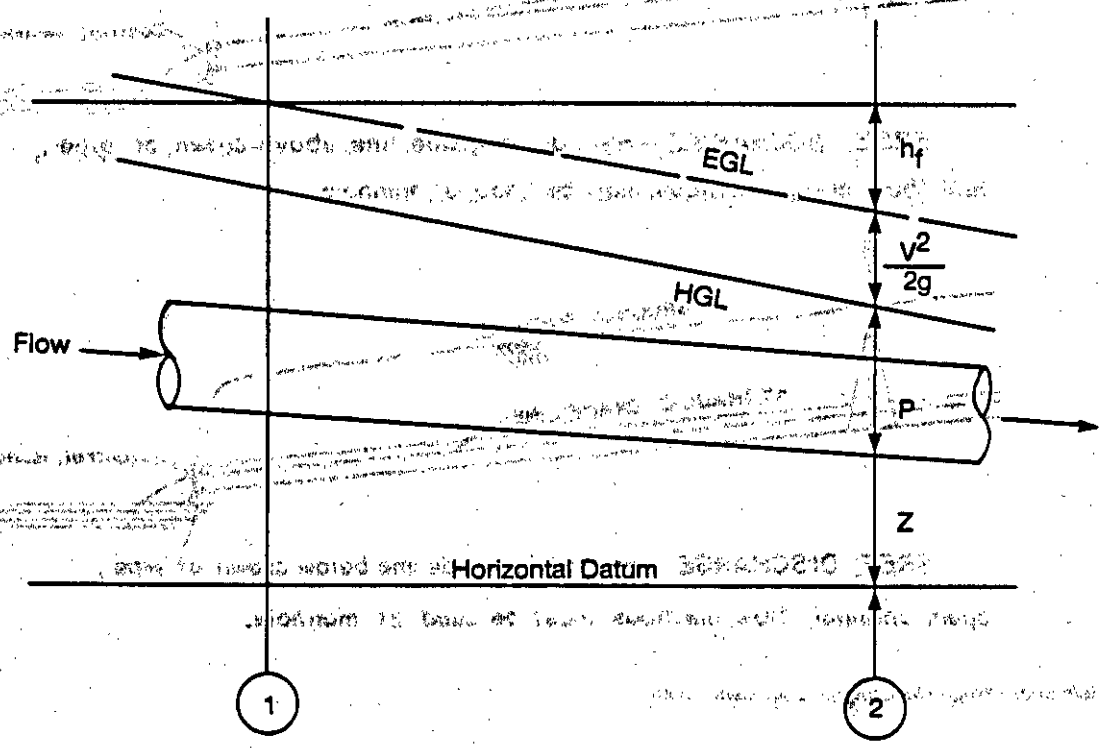
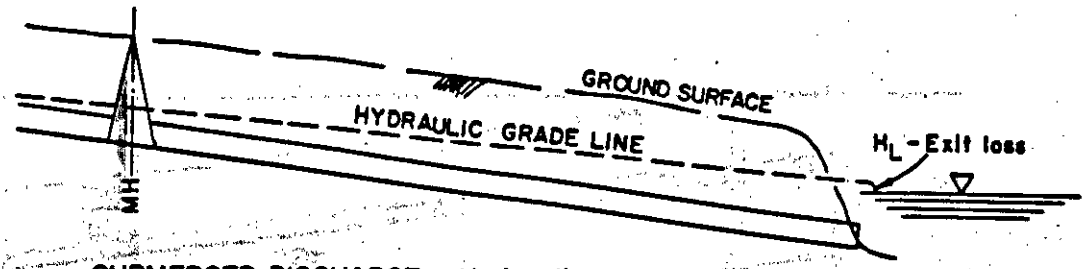
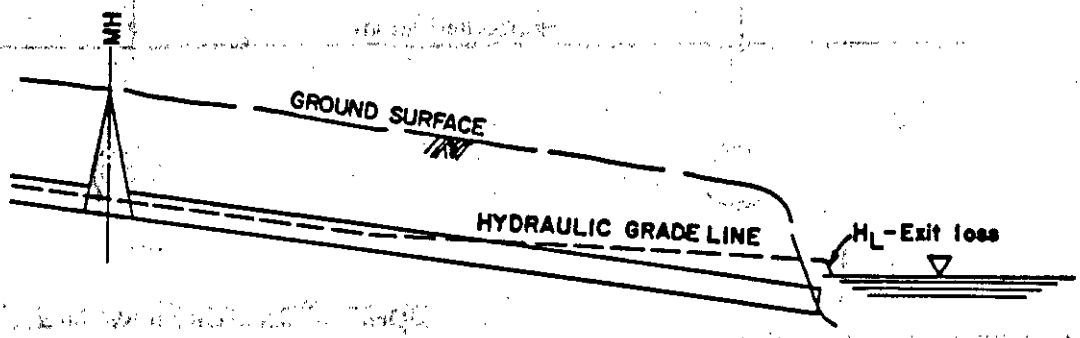


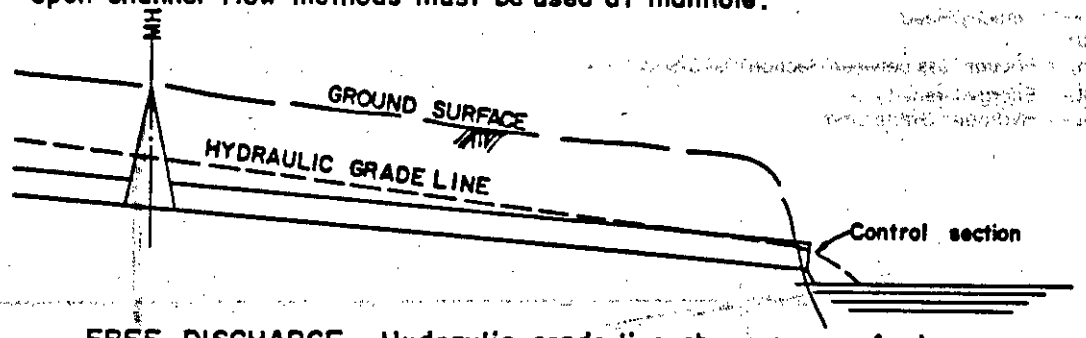
FIGURE 8-2
Pressure Flow in a Closed Conduit



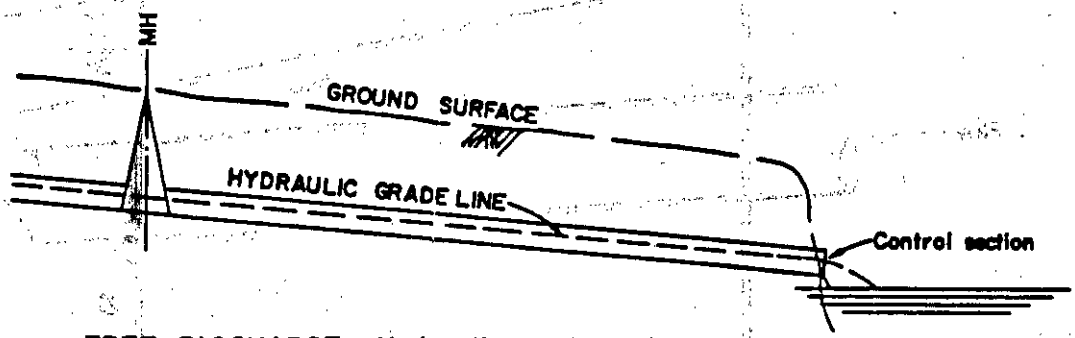
SUBMERGED DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



SUBMERGED DISCHARGED – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.



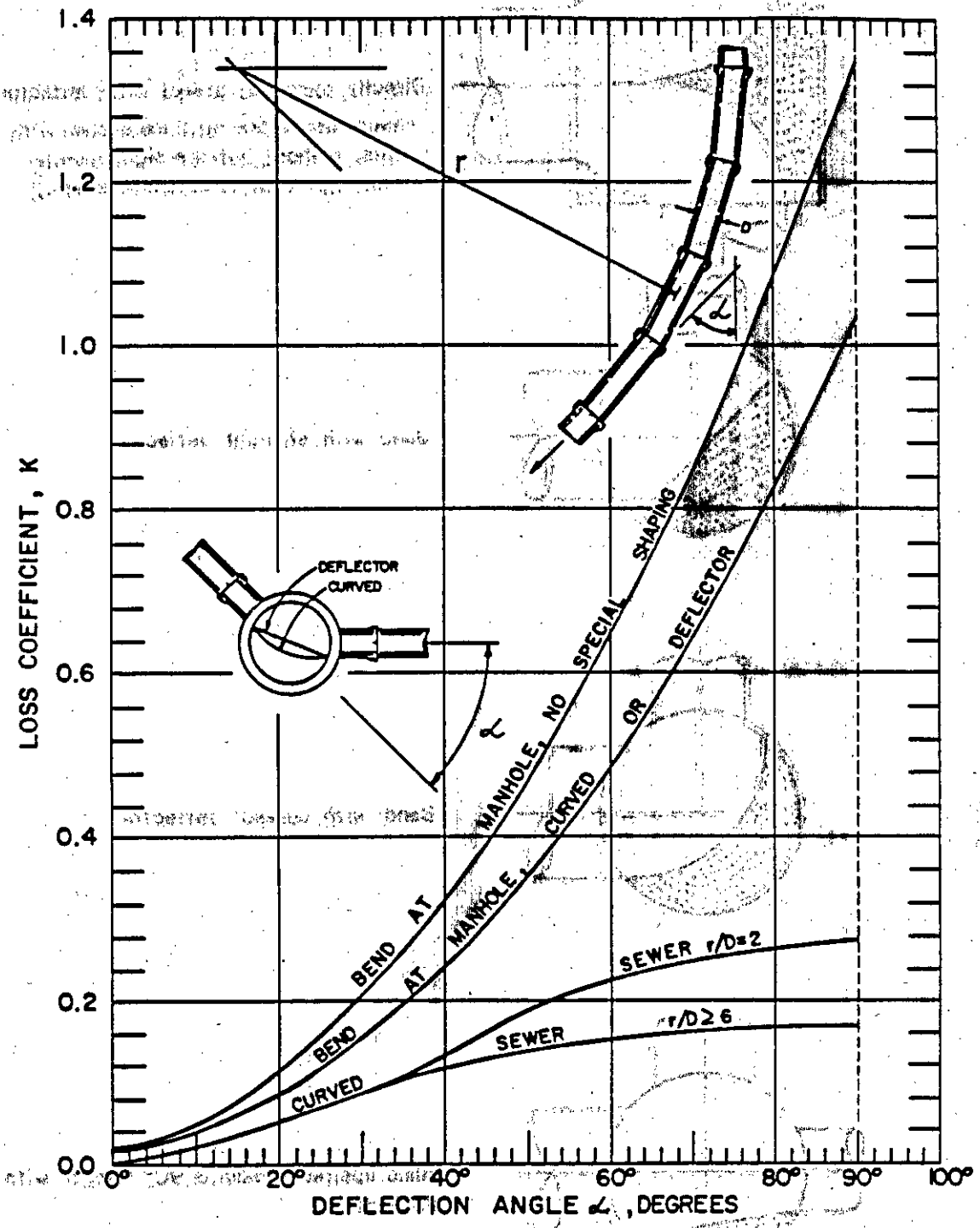
FREE DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



FREE DISCHARGE – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

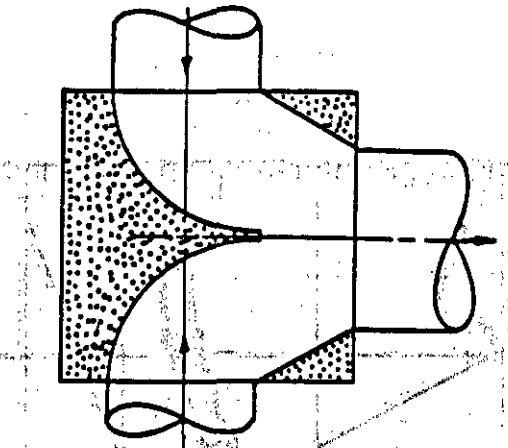
Reference: Wright-McLaughlin Engineers (1969).

FIGURE 8-3
Determination of Pressure vs. Open Channel Flow Conditions in Storm Sewer Systems

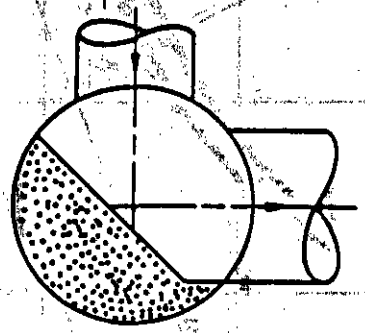


Reference: Wright-McLaughlin Engineers (1969).

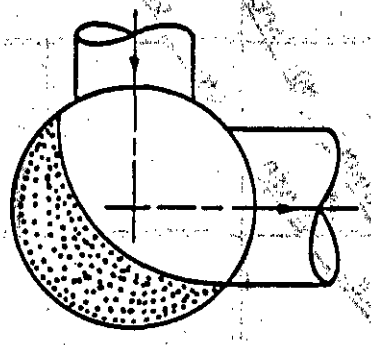
FIGURE 8-4
Storm Sewer Bend Loss Coefficient



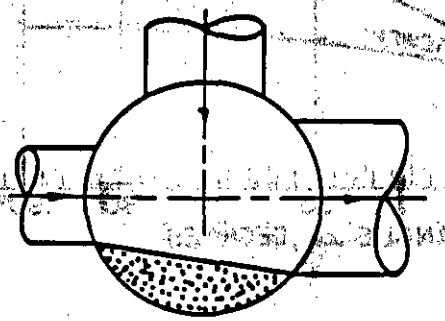
Directly opposed lateral with deflector
(head losses are still excessive with this method, but are significantly less than when no deflector exists.)



Bend with straight deflector



Bend with curved deflector

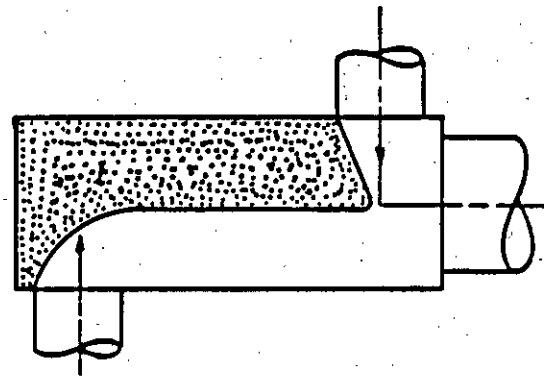


Inline upstream main & 90° lateral with deflector

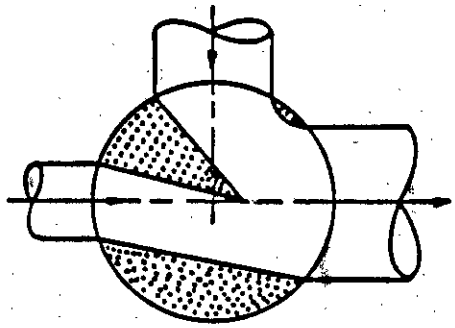
These methods of shaping the interior of a manhole were found efficient in University of Missouri (1958) tests.

Reference: Wright-McLaughlin Engineers (1969).

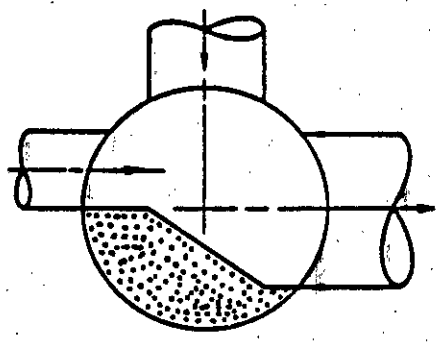
FIGURE 8-5
Efficient Manhole Shaping



Offset lateral with deflector



Inline upstream main & 90° lateral with divider

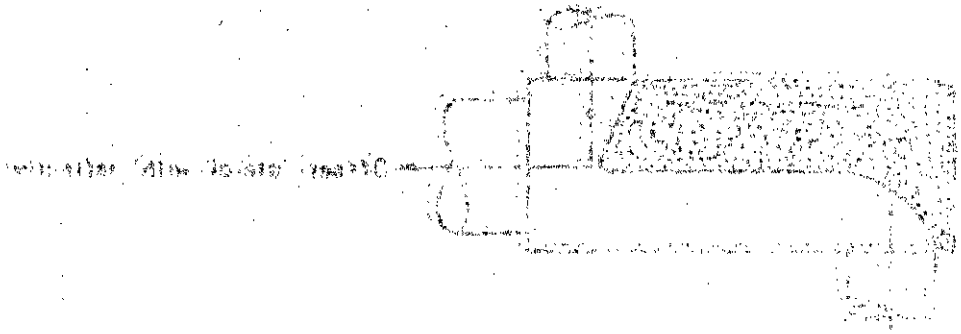


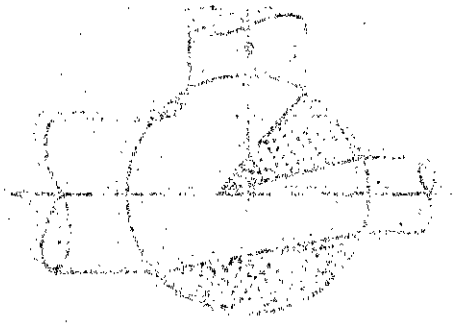
Inline upstream main & 90° lateral with deflector

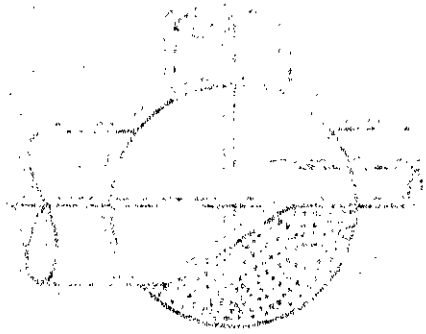
These methods of shaping the interior of a manhole were found efficient in University of Missouri (1958) tests, either due to increased head loss or tendency to plug with trash.

Reference: Wright-McLaughlin Engineers (1969).

FIGURE 8-6
Inefficient Manhole Shaping







CHAPTER 9
Storage System Hydraulics

CHAPTER 9
Storage System Hydraulics

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Chapter 9
STORAGE SYSTEM HYDRAULICS

9.1 TERMINOLOGY

For stormwater management purposes, retention refers to storage without access to a positive outlet, while detention facilities offer temporary storage accompanied by controlled release of the stored water. Wet detention typically has a pool of water below the outlet elevation; dry detention is typically placed with the basin bottom above the seasonal high water table. Retention and detention can be used separately or together in storage basins as site conditions and management objectives require.

9.2 DESIGN CRITERIA

The standard for determining the requirement for retention and/or detention facilities shall be that the calculated peak rate of stormwater runoff shall be no greater after development of a site than before development of a site. (See Sections 2.1.1 and 3.2.5). Each development plan will be reviewed by the Engineer of Public Works to determine if detention and/or retention will be required. When required, design criteria for detention/retention facilities shall include the following items:

1. Release rate
2. Detention volume
3. Grading and depth requirements
4. Outlet works

9.2.1 RELEASE RATE

Release rates for managing stormwater quantity by detention shall be based on limiting peak runoff rates, to match the following value:

The post development and pre-development calculated peak rate of stormwater runoff resulting from a ten-year return period, twenty-four (24) hour duration, type II storm distribution (as defined by the Soil Conservation Service, U.S. Department of Agriculture) shall be equal.

9.2.2 DETENTION VOLUME

Detention volume shall be adequate to provide attenuation of the post-development peak discharge rates to the allowable release rates set according to provisions in Section 9.2.1.

Routing calculations shall be consistent with procedures in Section 9.6. If siltation during construction causes loss of detention volume, design dimensions shall be restored prior to submitting as-built certification. Detention volume shall be drained within 72 hours.

9.2.3 GRADING AND DEPTH

To obtain sufficient storage volume, the construction of dry detention/retention facilities usually requires either excavation or the placement of earthen embankments. If vegetated embankments are used, they shall be less than 10 feet in height and shall be placed with side slopes no steeper than 4:1 (horizontal:vertical). Slopes for rip-rapped earthen embankments should be no steeper than 3:1. A slope stability evaluation is generally appropriate for embankment configurations with impoundment depths greater than 3 feet.
The maximum depth of stormwater detention/retention facilities should be consistent with safety and aesthetic considerations for the system. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage the growth of attached weeds (without creating undue potential for anaerobic bottom water) should be considered. A depth of from 6 to 8 feet is generally reasonable.

Other considerations when setting detention/retention facility depths include flood elevation requirements, public safety, land availability, land values, present and future land use, water table fluctuations, soil profile characteristics, maintenance, and freeboard. Aesthetically pleasing features are also an important consideration and can often

be provided with little extra cost or loss of capacity. A minimum freeboard of 1 foot above the high water elevation of the design flood shall be provided, with higher values considered when addressing special concerns. A six (6) foot high security type chain link fence will be required around all retention/detention ponds. The fence shall include three (3) strands of barbed wire at the top and a minimum of one (1) 12 foot wide double locking gate.

9.2.4 OUTLET WORKS

Outlet works shall be designed to prevent overtopping of the embankment based on a 100-year discharge rate considering total watershed development. Outlet works can take the following forms:

1. Drop inlets with pipes
2. Weirs
3. Orifices
4. Curb openings for parking lot storage

The principal spillway shall convey the 10-year discharge rate for total watershed development without allowing flow to enter the emergency outlet. A smooth and stable transition to downstream facilities shall be provided. The sizing of a particular outlet work shall be based on results of hydrologic routing calcula-tions (see Section 9.6), consistent with criteria in Sections 9.2.1 and 9.2.2.

9.2.5 WET PONDS

The bottom of a detention pond intended to have a permanent pool of water shall be sealed according to the following criteria:

1. Compact the subgrade to 95 percent (Standard Proctor Test) with an optimum moisture content of 2 percent.

2. Place two 6-inch layers of clay (plasticity index greater than 20) above the compacted subgrade. Each 6-inch layer shall be compacted to 95 percent (Standard Proctor Test) with an optimum moisture content of 2 percent.

Side slopes for wet ponds shall be no greater than 3:1 and

riprap shall be installed to protect slopes from wave attack. Riprap shall be at least 2 feet below the control elevation and up to the 100-year flood elevation plus 1 foot of freeboard.

9.3 GENERAL PROCEDURE

The following three relationships shall be considered when sizing a stormwater detention facility:

1. Inflow hydrograph for an appropriate design storm (see Chapter 4)

2. Stage-storage curve for basin (see Figure 9-1 for an example)

3. Stage-discharge curve for basin outlet control structure (see Figure 9-2 for an example)

In most cases, a trial and error design procedure is required, because only the inflow hydrograph is known. A general procedure for evaluating these variables is presented below:

1. Compute inflow hydrographs for appropriate design storms using procedures from Chapter 4. In most cases, both pre- and post-development hydrographs are required.

2. Perform preliminary calculations to select a reasonable detention storage configuration (see Section 9.5) to handle the hydrographs from Step 1.

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The estimated volume is the maximum storage requirement calculated from Step 2.

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. With inflow hydro-graphs from Step 1, perform rout-

ing calculations to check the design using the Storage Indication Method (see Section 9.6). If the routed peak discharges exceed the allowable peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

6. If appropriate, evaluate the downstream effects of detention out-flow to ensure that the recession limb of the outflow hydrograph does not cause downstream flooding problems.

7. Evaluate the control structure outlet velocity and provide stabilization if necessary.

Because this procedure can involve a significant number of reservoir routing calculations, a computer procedure is useful for conducting final routing computations. Additional information on retention/detention basin design computations can be found in articles by Mason and Rhomberg (1982, 1983), McKinnon (1984), Mein (1980), and Rossmiller (1982).

9.4 OUTLET HYDRAULICS

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, and orifices. If culverts are used as outlet works, procedures presented in Chapter 6 should be used to develop stage-discharge data.

9.4.1 SHARP-CRESTED WEIRS--NO END CONTRACTIONS

A sharp-crested weir with no end contractions is illustrated in Part A of Figure 9-3. The discharge equation for this configuration (Chow, 1959) is expressed as:

$$Q = \left(3.27 + 0.4 \frac{H}{H_c} \right) L H^{1.5} \qquad (9\text{-}1)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 9-3, Part C)

H_c = Height of weir crest above channel bottom, in feet (see Figure 9-3, Part C)

L = Horizontal weir length, in feet

9.4.2 SHARP-CRESTED WEIRS--TWO END CONTRACTIONS

A sharp-crested weir with two end contractions is illustrated in Part B of Figure 9-3. The discharge equation for this configuration (Chow, 1959) is expressed as:

$$Q = \left(3.27 + 0.4 \frac{H}{H_c}\right)\left(L - 0.2H\right) H^{1.5} \qquad (9\text{-}2)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 9-3, Part C)

H_c = Height of weir crest above channel bottom, in feet (see Figure 9-3, Part C)

L = Horizontal weir length, in feet

9.4.3 SHARP-CRESTED WEIRS--SUBMERGED DISCHARGE

The effect of submergence on a sharp-crested weir should be considered when applying Equations 9-1 and 9-2. 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 Equation 9-1 or 9-2 should be modified using the equation:

$$Q_s = Q_f \left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \qquad (9\text{-}3)$$

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

9.4.4 BROAD-CRESTED WEIRS

The general form of the broad-crested weir equation (Brater and King, 1976) is expressed as:

$$Q = C\ L\ H^{1.5} \qquad (9\text{-}4)$$

where:

Q = Discharge, in cfs

C = Broad-crested weir coefficient

L = Broad-crested weir length, in feet

H = Head above weir crest (measured at least 2.5H upstream of the weir), 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 data from Brater and King (1976) on selecting C values for broad-crested weirs are presented in Table 9-1. The effect of crest roughness and slope on weir discharge can be computed by applying Manning's Equation (see Chapter 5).

9.4.5 V-NOTCH WEIRS

The discharge through a v-notch weir can be evaluated using the following equation (Brater and King, 1976):

$$Q = 2.5 \tan \frac{r}{2} H^{2.5} \qquad (9\text{-}5)$$

where:

Q = Discharge, in cfs

r = Angle of v-notch, in degrees

H = Head on vortex of notch, in feet

9.4.6 ORIFICES

The discharge through an orifice can be evaluated using the following equation:

$$Q = CA(2gH)^{0.5} \qquad (9\text{-}6)$$

where:

Q = Discharge, in cfs

C = Orifice coefficient (a value of 0.6 is usually appropriate; see Brater and King [1976] if additional information is desired)

A = Area of orifice, in square feet

g = Acceleration due to gravity, 32.2 feet/second

H = Head above orifice centroid, in feet

9.5 PRELIMINARY DETENTION CALCULATIONS

9.5.1 STORAGE VOLUME

The following steps provide a graphical procedure for developing a preliminary estimate of the storage volume required to meet allowable discharge requirements:

1. Develop an inflow hydrograph and an allowable peak outflow rate, Q_o. Plot the hydrograph on graph paper.

2. On the graph from Step 1, draw an estimated outflow hydrograph with an allowable peak outflow of Q_o occurring on the recession limb of the inflow hydrograph. A simple estimate can be made by drawing a straight line from the point at which discharge begins to Q_o on the recession limb of the inflow hydrograph.

3. Calculate the area bounded by the line from Step 2 and the inflow hydrograph. This area represents a preliminary estimate of the required storage volume. Assuming triangular inflow and outflow hydrographs, a preliminary estimate of the storage volume can be made using the following equation:

$$S = 0.5b\,(Q_i - Q_o) \qquad (9\text{-}7)$$

where:

S = Volume of storage, in cubic feet

Q_i = Inflow peak flow, in cfs

Q_o = Outflow peak flow, in cfs

b = Duration of runoff, in seconds

4. Confirm that the preliminary estimate is adequate, using the Storage Indication Method (see Section 9.6.1).

9.5.2 PEAK FLOW REDUCTION

A preliminary estimate of the potential peak flow reduction for a selected detention volume can be obtained as follows:

1. Determine input data, including the peak flow rate of the inflow hydrograph, Q_i, the storage volume, S, and the duration of runoff, b.

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation:

$$Q_o = bQ_i - S \qquad (9\text{-}8)$$

where:

Q_o = Outflow peak flow, in cfs

b = Duration of runoff, in seconds

Q_i = Inflow peak flow, in cfs

S = Volume of storage, in cubic feet

3. Confirm that the preliminary estimate is adequate, using the Storage Indication Method (see Section 9.6.1).

9.6 ROUTING CALCULATIONS

The Storage Indication Method is recommended for performing reservoir routing calculations for final design of detention facilities.

9.6.1 STORAGE INDICATION METHOD

The following procedure is used to perform a reservoir routing by the Storage Indication Method:

1. Develop an inflow hydrograph, a stage-discharge curve, and a stage-storage curve for the proposed detention facilities. Example stage-storage and stage-discharge curves are presented in Figures 9-1 and 9-2, respectively.

2. Select a routing time period, Wt, to provide at least five points on the rising limb of the inflow hydrograph.

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of S OWt/2 versus stage. An example tabulation of storage-characteristics curve data is presented in Table 9-2 and Figure 9-4.

4. For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 - O_1 Wt/2$ can be determined from the appropriate storage characteristics curve (e.g., Figure 9-4).

5. Determine the value of $S_2 + O_2 Wt/2$ from the following relationship:

$$S_2 + \frac{O_2}{2} Wt = \left[S_1 - \frac{O_1}{2} Wt \right] + \left[\frac{I_1 + I_2}{2} Wt \right] \qquad (9\text{-}9)$$

where:

S_2 = Storage volume at time 2, in cubic feet

O_2 = Outflow rate at time 2, in cfs

Wt = Routing time period, in seconds

S_1 = Storage volume at time 1, in cubic feet

O_1 = Outflow rate at time 1, in cfs

I_1 = Inflow rate at time 1, in cfs

I_2 = Inflow rate at time 2, in cfs

6. Enter the appropriate storage characteristics curve (e.g., Figure 9-4) at the value of $S_2 + O_2 Wt/2$ deter-mined in Step 5 and read off a new depth of water, H_2.

7. Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage--discharge curve (e.g., Figure 9-2).

8. Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1, and H_1 equal to the previous I_2, O_2, S_2, and H_2, and using a new I_2 value. This process is continued until the entire inflow hydro-

graph has been routed through the storage basin.

9.6.2 EXAMPLE PROBLEM

Example 9-1. Reservoir Routing Using Storage Indication Method

An example application of the Storage Indication Method using data presented in Figures 9-1, 9-2, and 9-4 is presented in Table 9-3. The inflow hydrograph is given in columns 1 and 2 of Table 9-3. The objective is to find the outflow using the Storage Indication Method. A step-by-step discussion of the calculations summarized in Table 9-3 is presented below.

1. Using the data tabulated in Column 2 of Table 9-3, calculate:

$$\frac{I_1 + I_2}{2} Wt$$

and tabulate these values in Column 3 of Table 9-3.

2. Given that $S_1 - O_1Wt/2 = 0.05$ acre-foot for $H_1 = 0$ foot, find $S_2 + O_2Wt/2$ by adding 0.05 + 0.01 (Column 5 value plus Column 3 value) and tabulate 0.06 acre-foot in Column 6 of Table 9-3.

3. Enter the S + OWt/2 storage characteristics curve in Figure 9-4 and read the stage at the value of 0.06 acre-foot. This value is 100.10 feet and is tabulated as stage H_2 in Column 7 of Table 9-3.

4. Using the stage of 100.10 feet found in Step 4, enter the stage-discharge curve (Figure 9-2) and find the discharge corresponding to that stage. In this case, O is approximately 1 cfs and is tabulated in Column 8 of Table 9-3.

5. Assign the value of H_2 to H_1, find a new value of $S_1 - O_1Wt/2$ from Figure 9-4, and repeat the calculations for Steps 2, 3, and 4. Continue repeating these calculations until the entire inflow hydrograph has been routed through the storage basin.

6. The Storage Indication Method calculations give a peak outflow of 220 cfs. The inflow hydrograph has a peak rate of 360 cfs, so a reduction of approximately 40 percent is calculated.

9.7 PROTECTIVE TREATMENT

Protective treatment may be required to prevent entry to facilities that present a public safety hazard. Fences shall be provided for detention areas where one or more of the following conditions exist:

1. Rapid stage changes that would make escape practically impossible for small children

2. Water depths that either exceed 2.5 feet for more than 24 hours or are permanently wet and have side slopes steeper than 4:1 (horizontal:vertical)

3. Passage through the detention area of a low-flow watercourse or ditch with a depth greater than 5 feet or a flow velocity greater than 5 feet per second

4. Side slopes that equal or exceed 1.5:1 (horizontal:vertical)

Guards or grates may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

Fencing should be considered for dry retention areas with design depths in excess of 2.5 feet for 24 hours, unless the area is within a fenced, limited access facility.

Table 9-1
BROAD-CRESTED WEIR COEFFICIENT C VALUES
AS A FUNCTION OF WEIR CREST BREADTH AND HEAD

Measured Head, H[a] (ft)	Weir Crest Breadth (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

Reference: Brater and King (1976).

[a]Measured at least 2.5H upstream of the weir.

Table 9-2
EXAMPLE TABULATION OF STORAGE CHARACTERISTICS CURVES

Stage (ft above NGVD)	Storage[a] (acre-ft)	Discharge[b] (cfs)	Discharge[b] (acre-ft/hr)[d]	$S - \frac{O}{2}\Delta t$[c] (acre-ft)	$S + \frac{O}{2}\Delta t$[c] (acre-ft)
100	0.05	0	0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[a] Obtained from Figure 9-1.

[b] Obtained from Figure 9-2.

[c] Δt = 10 min = 0.167 hour.

[d] 1 cfs = 0.0826 acre-ft/hr.

Table 9-3
STORAGE INDICATION METHOD--EXAMPLE CALCULATIONS

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (min)	Inflow (cfs)	$\frac{(I_1 + I_2)\Delta t}{2}$ (acre-ft)	H_1 (ft)	$S_1 - \frac{O_1}{2}\Delta t$ (acre-ft)	$S_2 + \frac{O_2}{2}\Delta t$ (acre-ft)	H_2 (ft)	Outflow, O (cfs)
0	0						
10	2	0.01	0.0	0.05	0.06	100.10	1
20	27	0.20	100.10	0.06	0.26	101.10	16
30	130	1.08	101.10	0.21	1.29	102.20	41
40	300	2.96	102.20	0.61	3.57	104.10	100
50	360	4.55	104.10	2.20	6.75	105.60	175
60	289	4.47	105.60	4.40	8.87	106.25	217
70	194	3.33	106.25	5.80	9.13	106.30	220
80	133	2.25	106.30	5.90	8.15	106.05	205
90	91	1.54	106.05	5.30	6.84	105.65	177
100	61	1.05	105.65	4.50	5.55	105.10	147
110	37	0.67	105.10	3.60	4.27	104.50	116
120	20	0.39	104.50	2.70	3.09	103.80	87
130	11	0.21	103.80	1.90	2.11	103.05	64
140	5	0.11	103.05	1.18	1.30	102.25	43
150	1	0.04	102.25	0.63	0.67	101.40	22
160	0	0.0	101.40	0.35	0.35	100.70	10

Note:

$$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \quad \text{Equation 9-9}$$

(column 6) = (column 5) + (column 3)

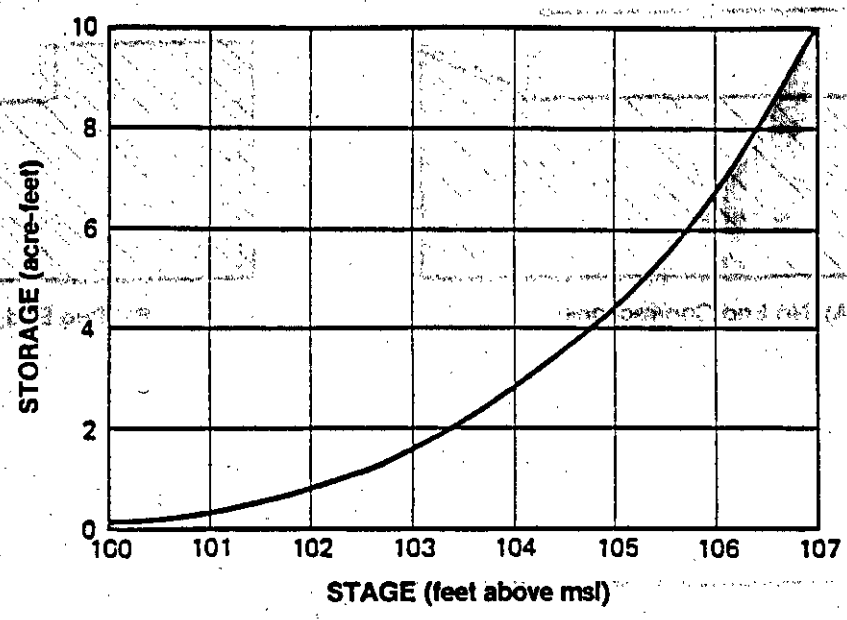


FIGURE 9-1
Example Stage-Storage Curve

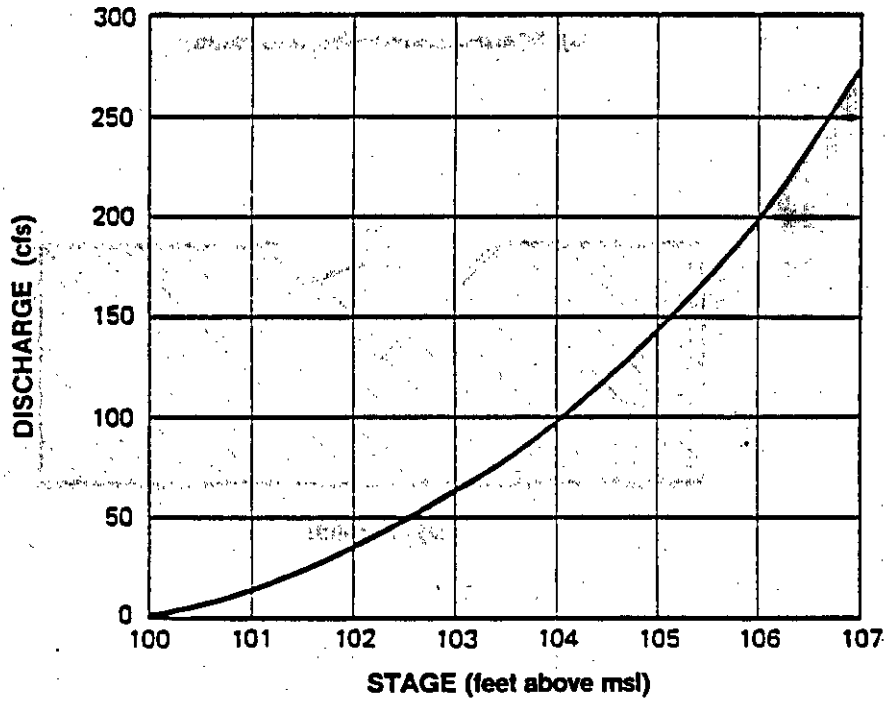


FIGURE 9-2
Example Stage-Discharge Curve

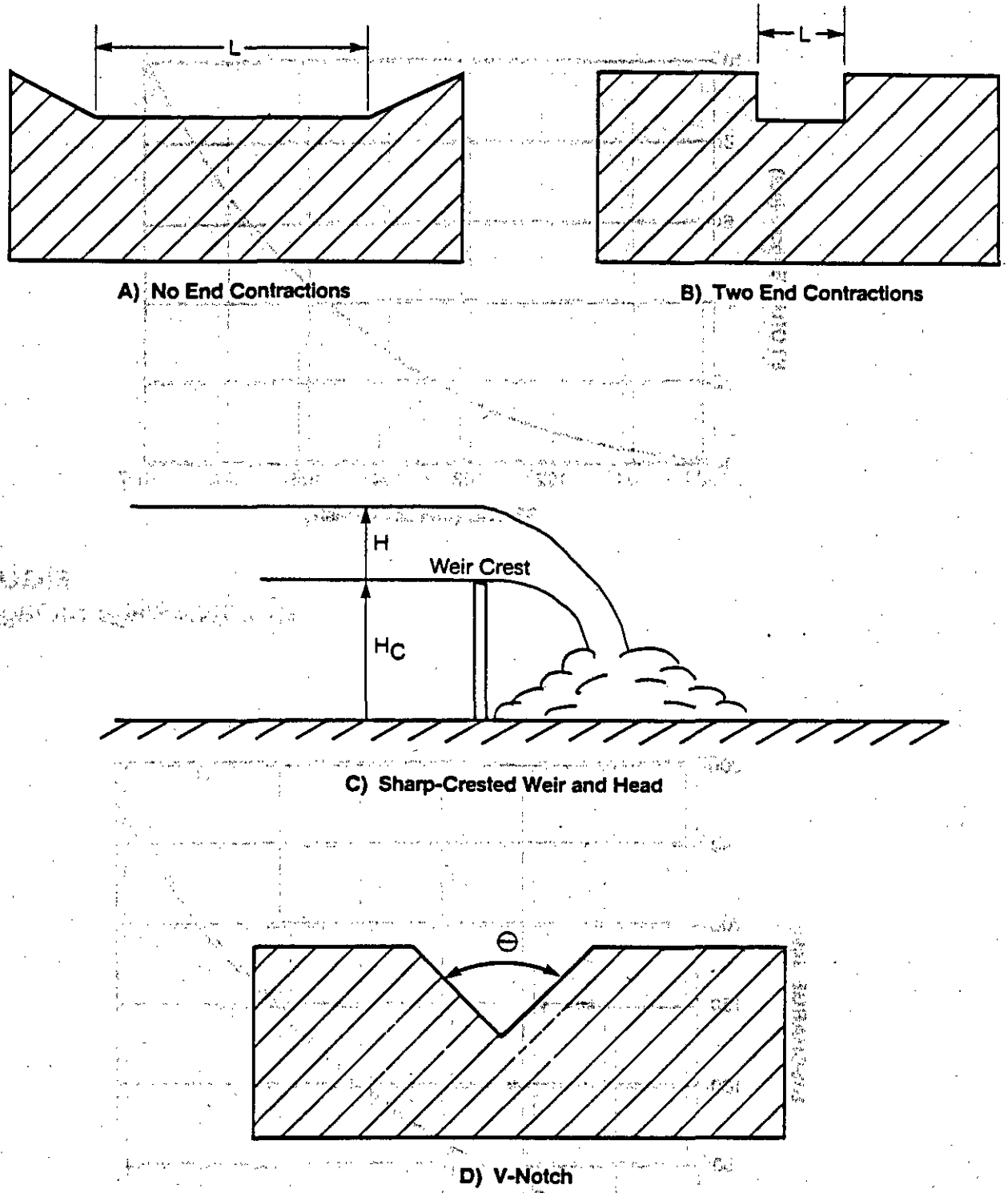
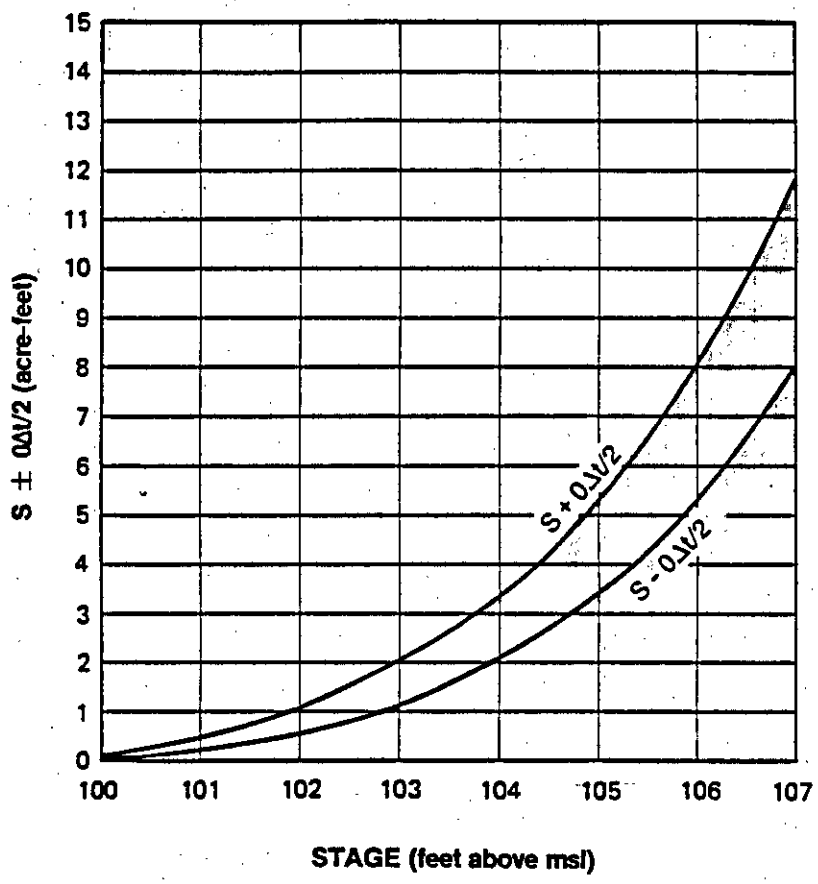
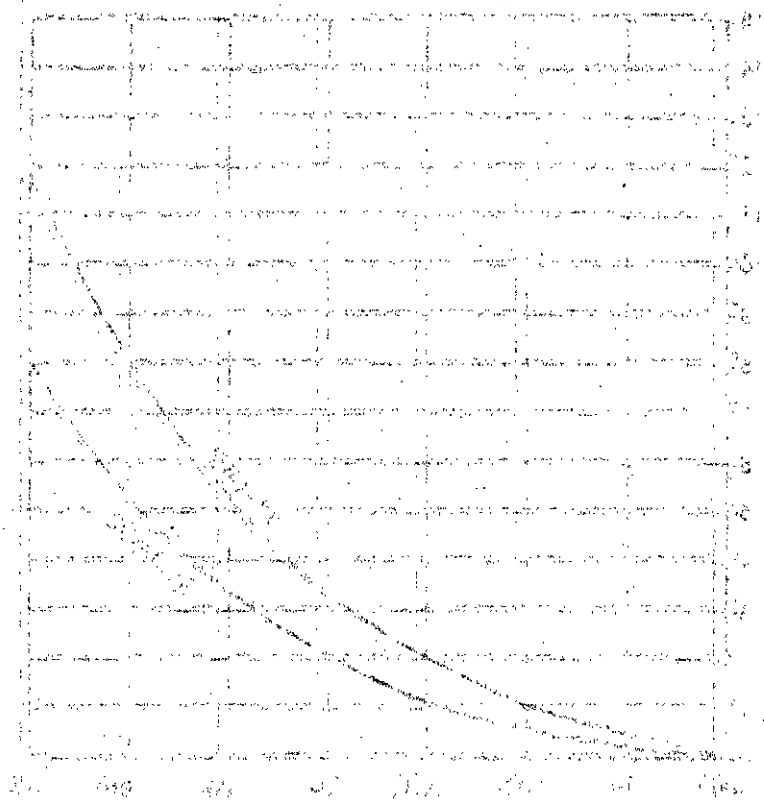


FIGURE 9-3
Illustrations of Weir Flow Control Structures



NOTE: Data presented in Table 9-2.

FIGURE 9-4
Storage Characteristics Curves



CHAPTER 10
Erosion and Sediment Control

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Chapter 10
EROSION AND SEDIMENT CONTROL

10.1 EROSION AND SEDIMENT CONTROL PLANS

10.1.1 SUBMITTAL REQUIREMENTS

The Engineer of Public Works shall require the following information on the erosion and/or sediment control plans:

1. A complete plan of the proposed development at a scale no less than 1 inch = 100 feet. This plan is to include existing and proposed contours at intervals no greater than 2 feet with at least one benchmark required.

2. Existing and proposed buildings, impervious surfaces, and drainage structures, including inlets, catch basins, manholes, junction boxes, driveway pipes, culverts, cross drains, headwalls, outlet facilities, and sanitary sewers.

3. Any proposed drainage improvements, such as channels, ditches, drainage pipes, and structures.

4. All fill areas indicated as such, with the limits and elevations indicated.

5. Drainage arrows indicating the existing and proposed direction of runoff throughout the plan.

6. Temporary erosion and sediment control measures to be implemented during construction, including a time sequence schedule for the construction and erosion control plan.

7. Measures the builder or contractor will use to keep construction materials, including dirt, mud, and debris, off the streets.

10.1.2 BASIC PRINCIPLES

The design of erosion and sediment control systems involves the application of common sense planning, scheduling, and

control actions that will minimize the adverse impacts or soil erosion, transport, and deposition. Five basic principles govern the development and implementation of a sound erosion and sediment control plan:

1. Plan the project to take advantage of the topography, soils, waterways, and natural vegetation at the site.

2. Expose the smallest practical area for the shortest possible time.

3. Apply onsite erosion control measures to reduce the gross erosion from the site.

4. Place sediment control measures to trap eroded material and prevent offsite damage.

5. Implement a continuous maintenance and inspection program.

The practices considered in this chapter are classified as either erosion control or sediment control. In general, erosion control practices are designed to prevent soil particles from being detached, whereas sediment control practices prevent the detached particles from leaving the site or from entering a receiving water. Sediment control measures are generally structural in nature; erosion control measures can be either structural (such as diversions) or non-structural (e.g., mulches). Additional information, including construction details, can be obtained from the Manual for Erosion and Sediment Control in Georgia (Georgia Soil and Water Conservation Committee, 1980).

10.2 EROSION CONTROL

The erosion control plan shall effectively control and reduce soil erosion from disturbed and filled land. The erosion control plan shall include appropriate measures from the following categories:

1. Surface stabilization
   a. Mulches
   b. Seeding and vegetation

c. Chemical binders and tacks
d. Synthetic fabrics and riprap

2. Runoff control
a. Diversions
b. Slope and down drains
c. Level spreaders
d. Check dams
e. Outlet protection

3. Exposure scheduling

Appropriate control measures for each classification are defined in enough detail to identify applicable conditions for employing each measure. Quantitative performance data and additional information related to design considerations are available in other references, such as USDA, SCS (1975), Wischmeier and Smith (1978), Goldman et al. (1986), or the Georgia Soil and Water Conservation Committee (1980).

10.2.1 TEMPORARY AND PERMANENT CONSIDERATIONS

Construction erosion control measures are usually temporary, intended to function for only the duration of construction. To the maximum extent possible, surface stabilization measures shall provide permanent protection once construction is complete. In addition, the layout for temporary runoff control measures shall be consistent with the layout of permanent drainage facilities.

10.2.2 *SURFACE STABILIZATION*

Surface stabilization measures will be required when land is cleared and existing conditions of the soil surface are altered. Surface stabilization control measures include mulches, seeding and vegetation, chemical binders or tacks, and synthetic fabrics and riprap.

Mulches (Temporary)

A mulch is a layer of material applied to the soil surface for temporary stabilization and to help establish plant cover by holding in moisture and preventing seed loss. The major types of mulching material include straw or hay and

wood chips. Mulches are practical for 6 months or less on graded or cleared areas when the growing season is not suitable to produce an erosion-resistant cover from seedings. Final grading is not required before mulching, but mulch may be applied after final grade is reached. Any structural erosion control features should be installed prior to mulching.

Seeding and Vegetation (Temporary and Permanent)

Surface stabilization by vegetation includes temporary seeding, permanent seeding, sod, vines, shrubs, and trees. Sod is preferred in situations where immediate protection is required, such as in swales or ditches. The types of seeding and vegetation, application rates, site preparation, and fertilizer and water requirements should be consistent with the growing season and duration of protection required. Mulching should be employed along with all seeding operations. Guidelines for selecting and placing vegetation are presented in Table 10-1. To establish permanent vegetation, soil testing results should be evaluated by an agronomist to determine soil treatment requirements for parameters such as pH, nitrogen, phosphorus, potassium, or other factors. Vegetative cover must be irrigated during dry periods.

Chemical Binders and Tacks (Temporary)

Chemical binders and tacks are sprayed on bare soils or mulches to bind soil particles or mulch material, reduce moisture loss, and enhance plant growth. A chemical binder or tack is a temporary erosion control measure and may be applied with seed, lime, and fertilizers. Chemical binders and tacks provide a viable alternative to seeding if construction occurs at a time when seeding is not feasible.

Synthetic Fabrics and Riprap

Other stabilization materials include nettings and plastic filter sheets for temporary control and dumped riprap for permanent control. Nettings of fiberglass, plastic, and paper yarn can be used to anchor straw, hay, wood chips, or grass and sod in drainageways and in other areas subject to concentrated runoff. Plastic filter sheets, which consist of porous fabric woven from polypropylene monofilament

yarns, are lightweight, porous, strong, and abrasion resistant. Dumped riprap consists of stone or broken concrete dumped in place on a filter blanket or prepared slope to form a well-graded mass with a minimum of voids.

10.2.3 RUNOFF CONTROL

Runoff control measures will be required whenever natural and/or existing land conditions result in an increase and/or change in runoff. This will include when land is cleared and/or vegetation removed to install utility systems, such as storm and sanitary sewers, gas, water, electric, and telephone. Runoff control measures include diversions, slope and down drains, level spreaders, check dams, and outlet protection.

Diversions (Temporary or Permanent)

Any structure that slows or diverts runoff away from exposed areas can reduce erosion. Types of diversion structures include dikes, swales, and channels, any of which can function as temporary or permanent facilities.

A diversion dike is a ridge of compacted soil placed above, below, or around a disturbed area to intercept runoff and divert it to a stable area. Generally, a diversion dike is the least durable diversion structure and is best used to provide protection for short periods and relatively small amounts of runoff. A dike is often appropriate above a newly constructed cut and fill slope to prevent excessive erosion until more permanent control features are established. Where the ground slope is not steep, a dike is also appropriate above graded slopes to divert sediment-laden runoff into sediment traps or basins. Once the slope is stabilized, the diversion dike should be removed.

A diversion swale is an excavated, temporary drainageway used above and below disturbed areas to intercept runoff and divert it to a safe disposal area. A diversion swale can be constructed at the perimeter of a disturbed area to transport sediment-laden water to a sediment trap or sediment basin. The swale should be left in place until the disturbed area is permanently stabilized. A diversion swale can be constructed in conjunction with a dike to prevent

stormwater from entering a disturbed area. Although generally a temporary feature, a carefully planned swale could also become part of the permanent drainage system.

A diversion channel is a permanent or temporary drainageway constructed by excavating a ditch along a hillside and often building a dike with the excavated soil along the downhill edge of the ditch. In other words, it can be a combination of a ditch and a dike. Although diversion channels can be used in place of temporary structures like dikes and swales, their primary application is to provide permanent runoff control on long slopes subject to heavy flow concentrations. In addition to usage on or above long spoil slopes, diversion channels are practical for intercepting runoff upgradient of a roadway to prevent offsite flow from entering gutter and inlet facilities.

Slope and Down Drains (Temporary or Permanent)

Down drain structures (pipes) and slope drains (paved or sodded channels) can be used as temporary or permanent structures to conduct concentrated runoff safely down a slope. Such structures are often used to help dispose of water collected by diversion structures.

A paved slope drain is a channel placed to extend from the top to the bottom of a slope and lined with bituminous concrete, portland cement concrete, or comparable nonerodible material (such as grouted riprap). This structure is generally used to convey a concentrated flow of surface runoff down a slope without causing erosion. When flow is supercritical, a pipe down drain should be used to prevent flow from leaving the channel.

A down drain generally consists of either corrugated metal pipe or flexible tubing, together with a prefabricated entrance section, and is temporarily placed to extend from the top to the bottom of a slope.

Level Spreaders (Permanent)

A level spreader is generally a permanent outlet constructed at zero percent grade across the slope to convert a concentrated flow of sediment-free runoff (e.g., from

diversion outlets) into sheet flow and to discharge it at nonerosive velocities onto undisturbed areas stabilized by existing vegetation. The level spreader should be used only where the following conditions are found:

1. The spreader can be constructed on undisturbed soil.

2. The area directly below the level lip is stabilized by existing vegetation.

3. The drainage area above the spreader is stabilized by existing vegetation.

4. The materials used are rigid and nonerodible, such as concrete or asphalt, and a fixed grade can be maintained.

5. The water will not be reconcentrated immediately below the point of discharge.

Check Dams (Permanent)

A check dam is generally a permanent structure used to maintain subcritical flow and thus stabilize the grade or control head cutting in natural or artificial channels. Check dams minimize or prevent excessive erosion by reducing velocities in watercourses or by providing partially lined channel sections or structures that can withstand high flow velocities. Check dams are used where the following conditions occur:

1. The capacity of earth or vegetative measures to handle water safely at permissible velocities is exceeded.

2. The grade or overall conditions are excessive.

3. Water must be lowered from one elevation to another.

Outlet Protection (Permanent)

Outlet protection entails providing de-energizing devices and erosion-resistant channel sections between drainage

outlets and stable downstream channels. The channel sections may be rock-lined, vegetated, paved with concrete, or otherwise erosion-resistant. The purpose of outlet protection is to convert pipe flow to channel flow and reduce the velocity of the water consistent with the channel lining. The flow of water can then be conveyed to a stable existing downstream channel without causing erosion.

This practice is applicable to storm sewer outlets, road culverts, and paved channel outlets discharging into natural or constructed channels that, in turn, discharge into existing streams or drainage systems. The appropriate treatment should be provided along the entire length of the flow path from the end of the conduit, channel, or structure to the point of entry into an existing stream or publicly maintained drainage system. Detailed information on energy dissipation is presented in HEC-14 (USDOT, FHWA, 1983).

10.2.4 EXPOSURE SCHEDULING

The season and length of time cleared land is exposed to the erosive forces of rainfall can have a significant effect on gross soil loss. In general, the goal is to select the season and duration of exposure so vegetation can be established before heavy rainfall. Historically, the highest monthly rainfalls in Huntsville occur during March, April, May, and December (see Table 4-2). A quantitative evaluation of exposure scheduling on the gross soil loss from a site can be performed using the monthly distribution of the rainfall factor, R, which can be obtained from data presented in Wischmeier and Smith (1978). This evaluation involves determining the appropriate surface stabilization factors for the sequence of land covers being considered.

10.3 SEDIMENT CONTROL

Sediment control measures will be required to trap eroded material from exposed areas and to prevent offsite damage. The sediment control plan shall include appropriate measures from the following categories:

1. Straw bale barriers and silt fences
2. Traps

3. Basins
4. Construction entrance stabilization

Sediment basins shall not be constructed under the following conditions:

1. Failure would result in loss of life, damage to buildings, or interruption of service from public roads or utilities.

2. The tributary drainage area is more than 150 acres.

3. The basin will become a maintenance problem for the City of Huntsville.

10.3.1 STRAW BALE BARRIERS AND SILT FENCES (TEMPORARY)

Straw bale barriers and silt fences are temporary construction measures with a life expectancy of 3 months or less. They can be placed across or at the toe of an exposed area to intercept and detain small amounts of sediment from unprotected areas of limited extent. Straw bale barriers and silt fences are stop-gap measures that should be applied only when other alternatives prove infeasible and in the following situations:

1. Water is not concentrated in a channel or other drainageway above the barrier.

2. The type of erosion expected is sheet and rill.

3. Contributing drainage area is less than one-half acre and the length of slope above the dike is less than 100 feet.

Straw bales shall be placed in a row with ends tightly abutting adjacent bales. Each bale shall be embedded a minimum of 4 inches into the soil and securely anchored in place by stakes or rebars. Silt fences shall be installed consistent with manufacturer specifications. Sketches of straw bale barriers and silt fences are presented in Figures 10-1 and 10-2, respectively.

10.3.2 TRAPS (TEMPORARY OR PERMANENT)

A sediment trap is a temporary basin formed by an excavation and/or an embankment to intercept sediment-laden runoff and trap and retain the sediment. In so doing, drainageways, properties, and rights-of-way below the trap are protected from sedimentation. If maintained by periodic cleaning, traps can be used as permanent facilities.

An earth outlet sediment trap consists of a basin formed by excavation and/or an embankment. The trap has a discharge point over or cut into natural ground. A pipe outlet sediment trap consists of a basin formed by an embankment or a combination of an embankment and excavation. The outlet for the trap is through a perforated riser and a pipe through the embankment.

A trap can also be formed at storm sewer inlets by placing a raised barrier around the inlet. In all cases, the tributary area to a sediment trap shall be limited to 1 acre. Sketches of temporary traps at storm sewer inlets are presented in Figure 10-3.

10.3.3 BASINS (TEMPORARY OR PERMANENT)

A sediment basin is usually a temporary facility constructed along a flow path to capture sediment carried by runoff from a cleared construction site. The basin is formed by placing an earthen dam across the watercourse, by excavating a depression, or by a combination of the two. Basins act as a last line of defense against offsite sediment damage and, if maintained by periodic cleaning, can sometimes be used as permanent facilities.

Components of a typical sediment basin are illustrated in Figure 10-4, including the following of storage volume:

1. Sediment storage volume for retaining trapped sediment

2. Detention storage volume to hold the design storm for a duration sufficient to remove sediment

3. Flood storage volume to prevent overtopping and dam failure during rare storm events

Sediment basins require scheduled inspection and maintenance to function properly and shall be located for easy access. In addition to routine maintenance, sediment basins shall be inspected after all major storm events to ensure that spillway structures are not clogged and that the sediment storage volume has not been exceeded.

A sediment basin shall be cleaned out when it has reached 50 percent of its sediment storage capacity. A red pole shall be placed in the sediment storage area to establish the depth at 50 percent of the storage volume. When the red pole disappears, the sediment shall be removed. The method of sediment disposal shall be productive and shall be clearly identified on the basin plans prior to construction.

10.3.4 APPROXIMATE BASIN SIZING

An approximate method for sizing a sediment basin based on order-of-magnitude settling velocities for various sizes of sediment particles uses the following equations:

$$A = 60\ Q\ S \qquad (10\text{-}1)$$

$$d_{min} = Q/vW \qquad (10\text{-}2)$$

where:

A = Surface area of sediment basin, in square feet

Q = Design discharge, in cfs

S = Settling velocity of sediment, in minutes/foot (from Table 10-2)

d_{min} = Minimum depth of basin, in feet

v = Velocity of flow through the basin, in feet/second

W = Width of basin, in feet

Settling velocities for various sediment particle sizes are presented in Table 10-2. After the required surface area has been calculated using Equation 10-2, the following guidelines provide a basis for sizing the basin:

1. Provide volume for storing settled solids.

2. Keep flow-through velocities at or below 0.5 foot per second.

3. Avoid unnecessary basin width (rule-of-thumb, length $\geq$3 times width). In wide basins, installation of a baffle is suggested to avoid short circuiting.

4. It is generally impractical to trap sediment finer than silt-size particles (or in some cases fine sand).

An example application of Equation 10-1, using Q = 57 cfs and S = 33 minutes/foot (for silt), gives A = 112,860 square feet. Assuming a width of 50 feet, a minimum depth to maintain a velocity of 0.5 foot/second is calculated using Equation 10-2 and gives a d_{min} of 2.3 feet. To provide sediment storage, an additional 2.7 feet of depth are added, resulting in the following dimensions:

Depth = 5 feet
Width = 50 feet
Length = 2,300 feet
Area = 2.6 acres

10.3.5 CONSTRUCTION ENTRANCES (TEMPORARY)

All points of access to a construction site shall be stabilized to reduce or eliminate the tracking or flowing of sediment onto public rights-of-way. A stabilized pad of crushed stones is to be placed at entrances of construction sites for this purpose. Maintenance activities shall include periodic top dressing with additional stone as conditions demand or cleanout of any facilities used to trap sediment.

In some cases, wheels of construction vehicles should be cleaned prior to leaving the construction site. When appropriate, a stabilized area (one with crushed stone) that drains into a sediment trap or basin shall be provided. Details for treatment of a typical construction entrance are shown in Figure 10-5.

Table 10-1
GUIDELINES FOR SELECTING VEGETATIVE COVER

Plant or Plant Mixture	Application Rate Per Acre (lb)[a]	Planting Dates[b]
TEMPORARY PLANT MIXTURES		
1. Abruzzi Rye	70	September-December
Kentucky 31 Fescue	30	September-December
Weeping Lovegrass	5	September-December
2. Kentucky 31 Fescue	30	January-April 15
Reseeding Crimson Clover	30	January-April 15
Weeping Lovegrass	5	January-April 15
3. Brown Top Millet[c]	30	April-August
Weeping Lovegrass	5	April-August
Kentucky 31 Fescue[d]	30	April-August
PERMANENT PLANT MIXTURES		
Backslopes Steeper than 3:1		
1. Bermudagrass (Hulled)	20	April-June
Kentucky 31 or Alfa Fescue	30	April-June
Sericea Lespedeza	30	April-June
2. Bermudagrass (Hulled)	25	July-August
Pensacola Bahiagrass	30	July-August
Sericea Lespedeza	35	July-August
3. Kentucky 31 or Alfa Fescue	35	September-March
Sericea Lespedeza	35	September-March
Reseeding Crimson Clover	30	September-March
Backslopes Flatter than 3:1		
1. Bermudagrass (Hulled)	20	April-June
Kentucky 31 or Alfa Fescue	30	April-June
Kobe Lespedeza	30	April-June
2. Bermudagrass (Hulled)	25	July-August
Pensacola Bahiagrass	30	July-August
Reseeding Crimson Clover	35	July-August
3. Kentucky 31 or Alfa Fescue	25	September-March
Reseeding Crimson Clover	30	September-March
Bermudagrass (Unhulled)	30	September-March
Annual Ryegrass	25	September-March

Table 10-1
(continued)

Plant or Plant Mixture	Application Rate Per Acre (lb)[a]	Planting Dates[b]
Rocky Backslopes Steeper than 3:1		
1. Weeping Lovegrass	5	April-June
Pensacola Bahiagrass	30	April-June
Kentucky 31 Fescue	30	April-June
Crown-Vetch Corinilla Varia[e]	8	April-June
2. Weeping Lovegrass	5	July-August
Pensacola Bahiagrass	20	July-August
Annual Lespedeza (Kobe)	20	July-August
Reseeding Crimson Clover	20	July-August
Crown Vetch Corinilla Varia	8	July-August
3. Weeping Lovegrass	5	September-March
Reseeding Crimson Clover	30	September-March
Kentucky 31 Fescue	20	September-March
Pensacola Bahiagrass	20	September-March
Crown Vetch Corinilla Varia[e]	8	September-March

PERMANENT SPRIG PLANTS		
1. Riflawn Bermudagrass	30 Cubic Feet, Machine	Acceptable Dates
2. Centipede	Set; 50 Cubic Feet,	Should Be Confirmed
3. Myers Zoysia	Broadcast & Disked	with Local USDA
4. Zoysia Matrella		Extension Office

Reference: Alabama Highway Department (1985).

[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]Brown Top Millet (Panicum Ramosum) 97 percent min. purety - 80 percent min. germmable.

[d]Overseed in September or October.

[e]Variety--Emerald or Che Mung.

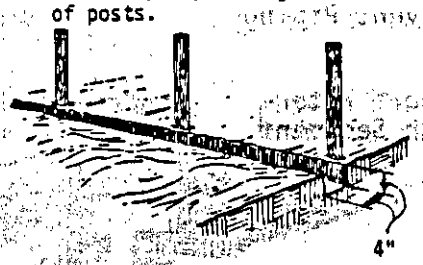
Table 10-2
APPROXIMATE SETTLING VELOCITIES FOR VARIOUS SIZE SEDIMENT PARTICLES

Diameter of Grain (mm)	Order of Magnitude	Time Required to Settle 1 Foot
10	Gravel	.3 sec (0.005 min)
1	Coarse Sand	3.0 sec (0.05 min)
0.1	Fine Sand	38.0 sec (0.633 min)
0.01	Silt	33.0 min
0.001	Size of Bacteria	55.0 hr
0.0001	Size of Clay Particles	230.0 days

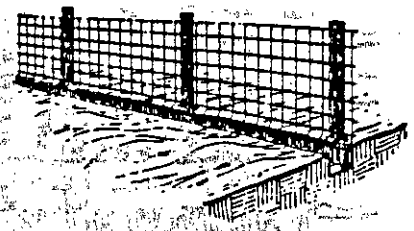
Reference: Urquhart (1959).

[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]

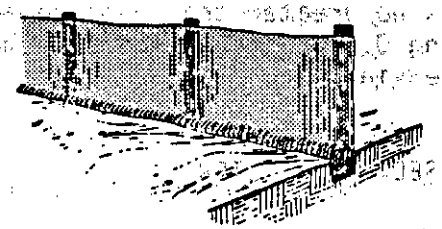
1. Set posts and excavate a 4"x4" trench upslope along the line of posts.



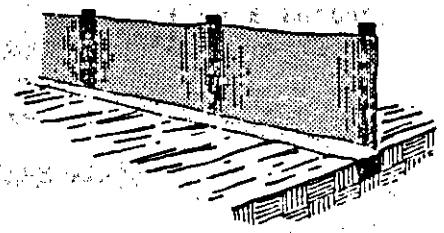
2. Staple wire fencing to the posts.



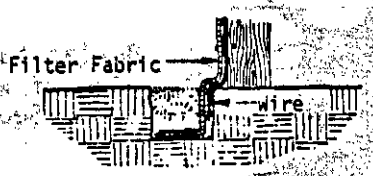
3. Attach the filter fabric to the wire fence and extend it into the trench.



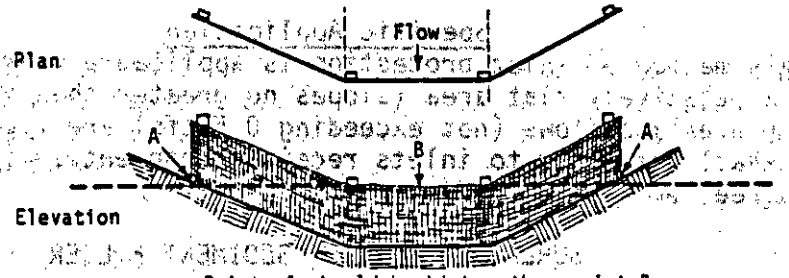
4. Backfill and compact the excavated soil.



Extension of fabric and wire into the trench.



CONSTRUCTION OF A SILT FENCE

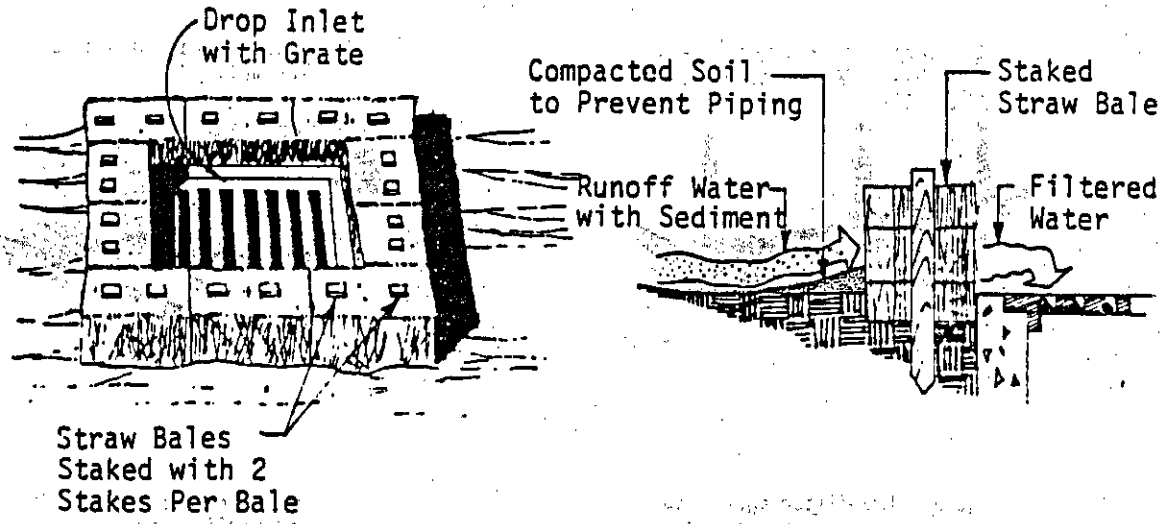


Points A should be higher than point B

PROPER PLACEMENT OF A FILTER BARRIER IN A DRAINAGE WAY

Reference: Virginia Soil and Water Conservation Commission (1980).

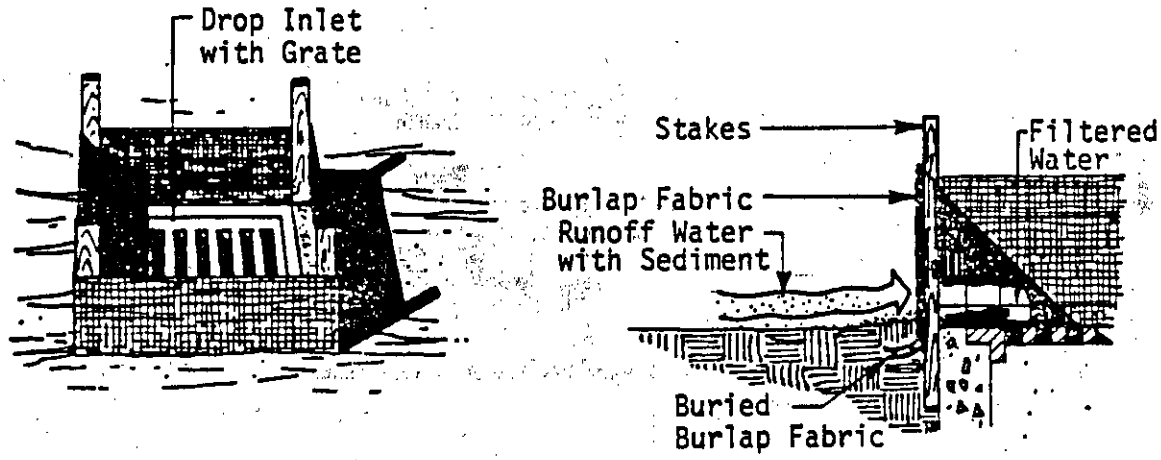
FIGURE 10-2
Details for Placing Silt Fences



Specific Application

This method of inlet protection is applicable where the inlet drains a relatively flat area (slopes no greater than 5 percent) where sheet or overland flows (not exceeding 0.5 cfs) are typical. The method shall not apply to inlets receiving concentrated flows, such as in street or highway medians.

STRAW BALE DROP INLET SEDIMENT FILTER



Specific Application

This method of inlet protection is applicable where the inlet drains a relatively flat area (slopes no greater than 5 percent) where sheet or overland flows (not exceeding 0.5 cfs) are typical. The method shall not apply to inlets receiving concentrated flows, such as in street or highway medians.

BURLAP DROP INLET SEDIMENT FILTER

Reference: Virginia Soil and Water Conservation Commission (1980).

FIGURE 10-3
Details for Placing Temporary Inlet Sediment Traps

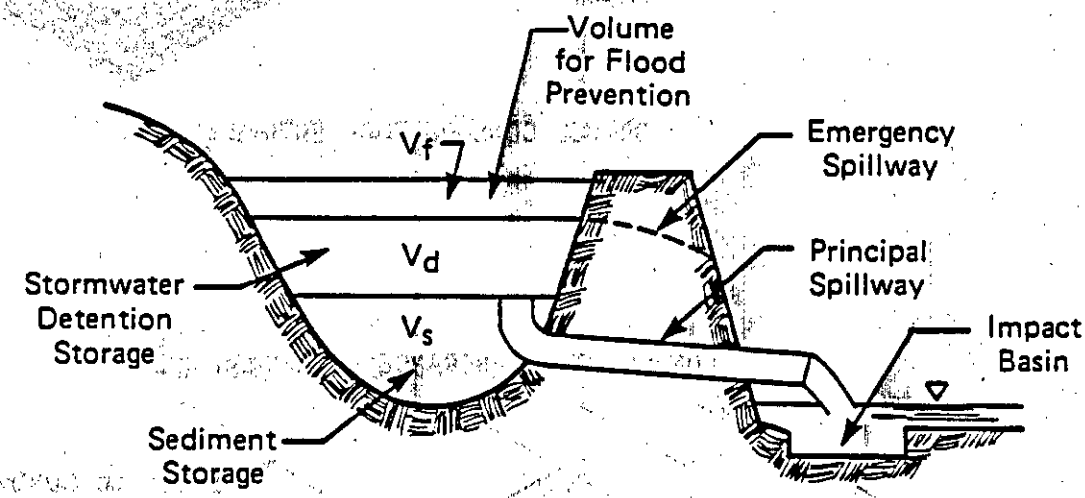
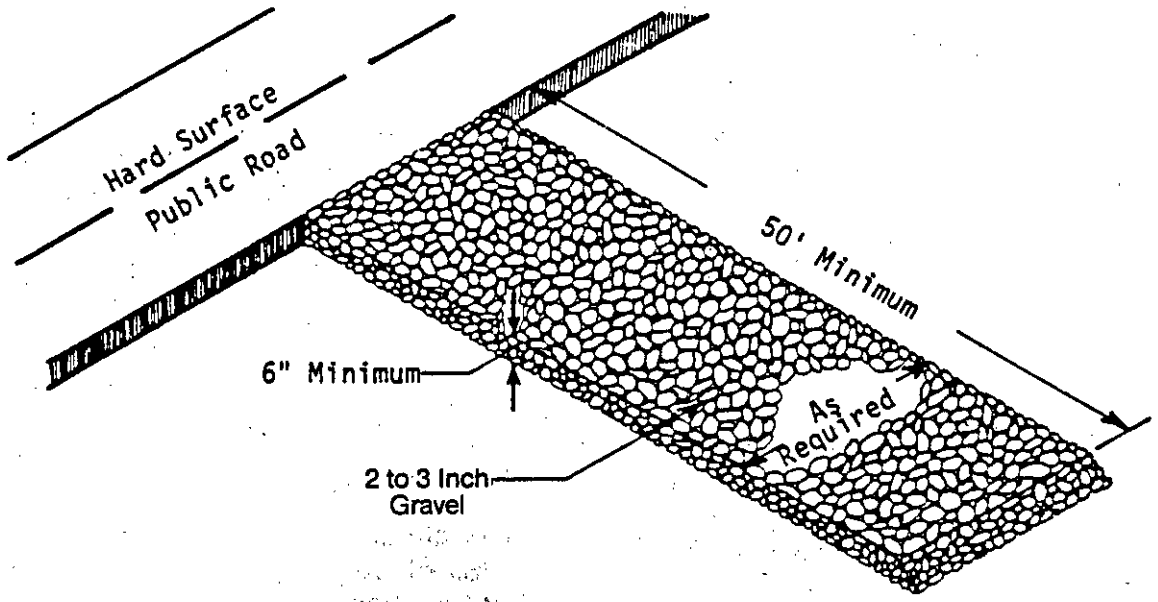
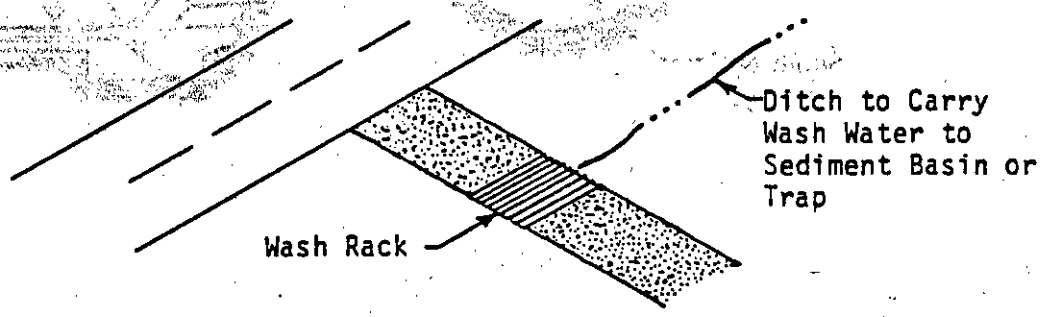


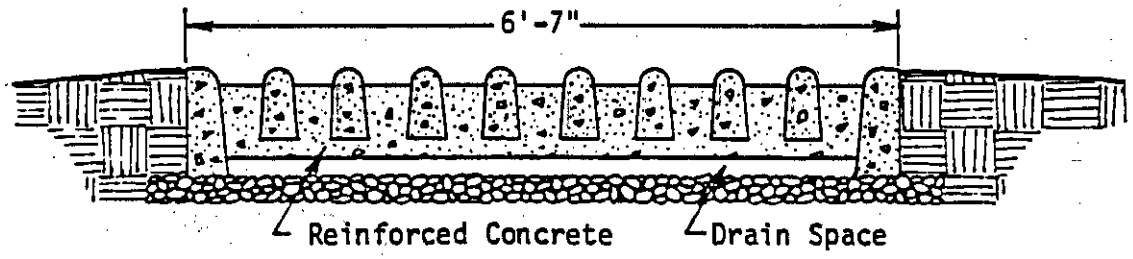
FIGURE 10-4
Conceptual Sketch of Components for Typical Sediment Basins



GRAVEL CONSTRUCTION ENTRANCE

CONSTRUCTION ENTRANCE WITH WASH RACK





Detail of Wash Rack

Reference: Virginia Soil and Water Conservation Commission (1980).

FIGURE 10-5
Details for Construction Entrance Treatment

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REFERENCES

REFERENCES

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APPENDIX A
Ordinance 87-269
Article XLV Floodway-Floodway Fringe Districts Regulations

APPENDIX A
Ordinance 87-259
Article XLV Floodway-Fringe Districts Regulations

Appendix A
ORDINANCE NO. 87-269

Article XLV - Floodway-Floodway Fringe Districts Regulations

Purpose in View

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ORDINANCE NO. 87-269

AN ORDINANCE TO AMEND THE ZONING ORDINANCES OF THE CITY OF HUNTSVILLE, ALABAMA

The public welfare requiring it, and under authority granted by Section 11-52-78 of the 1975 Code of Alabama, BE IT ORDAINED by the City Council of the City of Huntsville, Alabama, that the Zoning Ordinance of the City of Huntsville, Alabama, as adopted on the 21st day of March, 1963, as amended, is hereby further amended as follows:

Repeal ARTICLE XLV and replace with new ARTICLE XLV to read as follows:

ARTICLE XLV - FLOODWAY-FLOODWAY FRINGE DISTRICTS REGULATIONS

Purpose in View

It is the purpose of this ordinance to promote the public health, safety and general welfare and to minimize public and private losses due to flood conditions in specific areas by provisions designed to:

(1) restrict or prohibit uses which are dangerous to health, safety and property due to water or erosion or in flood heights or velocities;

(2) require that uses vulnerable to floods, including facilities which serve such uses, be protected against flood damage at the time of initial construction;

(3) control the alteration of natural flood plains, stream channels, and natural protective barriers which are involved in the accommodation of flood waters;

(4) control filling, grading, dredging and other development which may increase erosion or flood damage; and,

(5) prevent or regulate the construction of flood barriers which will unnaturally divert flood waters or which may increase flood hazards to other lands. (79-239)

45.1 - Provision for Floodway District and Floodway Fringe District

The boundaries and designations of the Floodway District and Floodway Fringe District provided for herein are hereby established as shown on the maps identified by the title "Flood Boundary and Floodway Maps, City of Huntsville, Alabama, Madison County," (Community-Panel Numbers 010153 0001-0050) composed of panels 10, 15, 20 and 30 issued by the U.S. Department of Housing and Urban Development, Federal Insurance Administration, bearing an effective date of November 1, 1979, and panels 25, 35, 40 and 50 issued by the Federal Emergency Management Agency, revised effective date of September 27, 1985. These maps as identified, and all explanatory matter thereof, are hereby adopted and made a part of this ordinance. Such maps and explanatory matter shall be filed in the office of the City Clerk-Treasurer of the City of Huntsville, Alabama. (79-239)(85-714)

45.2 - Definitions

Addition (to an existing building) - Any walled and roofed expansion to the perimeter of a building in which the addition is connected by a common load-bearing wall other than a fire wall. Any walled and roofed addition which is connected by a fire wall or is separated by independent perimeter load-bearing walls is new construction.

Area of shallow flooding - A designated AO Zone on a community's Flood Insurance Rate Map (FIRM) with base flood depths from one to three feet where a clearly defined channel does not exist, where the path of flooding is unpredictable and indeterminate, and where velocity flow may be evident.

Area of special flood hazard - The land in the flood plain within a community subject to a one percent or greater chance of flooding in any given year.

Base flood - The flood having a one percent chance of being equaled or exceeded in any given year.

Development - Any man-made change to improved or unimproved real estate, including, but not limited to, buildings or other structures, mining, dredging, filling, grading, paving, excavating, drilling operations, or permanent storage of materials.

Elevated building - A non-basement building built to have the lowest floor elevated above the ground level by means of fill, solid foundation perimeter walls, pilings, columns (posts and piers), shear walls, or breakaway walls.

Flood or flooding - A general and temporary condition of partial or complete inundation of normally dry land areas from:

(1) the overflow of inland waters;

(2) the unusual and rapid accumulation or runoff of surface waters from any source.

Flood Hazard Boundary Map (FHBM) - An official map of a community, issued by the Federal Emergency Management Agency, where the boundaries of the areas of special flood hazard have been defined as Zone A.

Flood Insurance Rate Map (FIRM) - An official map of a community on which the Federal Emergency Management Agency has delineated both the areas of special flood hazard and the risk premium zones applicable to the community.

Flood Insurance Study - The official report provided by the Federal Emergency Management Agency. The report contains flood profiles, as well as the Flood Boundary Floodway Map and the water surface elevation of the base flood.

Floodway - The channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than one foot.

Floor - The top surface of an enclosed area in a building (including basement), i.e., top of slab in concrete slab

construction or top of wood flooring in wood frame construction. The term does not include the floor of a garage used solely for parking vehicles.

Functionally dependent facility - A facility which cannot be used for its intended purpose unless it is located or carried out in close proximity to water, such as a docking or port facility necessary for the loading and unloading of cargo or passengers, shipbuilding, ship repair, or seafood processing facilities. The term may include long-term storage, sales, or service facilities in connection with a marine facility only with the approval of the Board of Adjustment.

Highest adjacent grade - The highest natural elevation of the ground surface, prior to construction, next to the proposed walls of a structure.

Mean Sea Level - The average height of the sea for all stages of the tide. It is used as a reference for establishing various elevations within the flood plain. For the purpose of this ordinance, the term is synonymous with National Geodetic Vertical Datum (NGVD).

Manufactured home - A structure, transportable in one or more sections, which is built on a permanent chassis and designed to be used with or without a permanent foundation when connected to the required utilities. The term also includes mobile homes, park trailers, travel trailers, and similar transportable structures placed on a site for 180 consecutive days or longer and intended to be improved property.

National Geodetic Vertical Datum (NGVD) - As corrected in 1929, NGVD is a vertical control used as a reference for establishing varying elevations within the flood plain.

New construction - Structures for which the "start of construction" commenced on or after the effective date of this ordinance.

Start of construction - The actual start means the first placement of permanent construction of a structure (including a manufactured home) on a site, such as the pouring of slabs or footings, installation of piles,

construction of columns, or any work beyond the stage of excavation or the placement of a manufactured home on a foundation. Permanent construction does not include land preparation, such as clearing, grading and filling; nor does it include the installation of streets and/or walkways; nor does it include excavation for a basement, footings, piers or foundations or the erection of temporary forms; nor does it include the installation on the property of accessory buildings, such as garages or sheds not occupied as dwelling units or not part of the main structure.

Substantial improvement - Any combination of repairs, reconstruction, alteration, or improvements to a structure in which the cumulative cost equals or exceeds fifty percent of its current replacement cost. The current replacement cost of the structure should be (1) the appraised value of the structure prior to the start of the initial repair or improvement, or (2) in the case of damage, the value of the structure prior to the damage occurring. For the purpose of this definition, "substantial improvement" is considered to occur when the first alteration of any wall, ceiling, floor, or other structural part of the building commences, whether or not that alteration affects the external dimensions of the structure. The term does not, however, include any project for improvement of a structure required to comply with existing health, sanitary, or safety code specifications which are solely necessary to assure safe living conditions.

45.3 - Provisions for All Flood Hazard Districts

The provisions of this section shall apply to all flood hazard districts.

45.3.1 All new construction and substantial improvements shall be constructed and/or anchored to prevent flotation, collapse or lateral movement of the structure.

45.3.2 All new construction and substantial improvements shall be constructed with materials and utility equipment resistant to flood damage.

45.3.3 All new construction and substantial improvements shall be constructed by methods and practices that minimize flood damage.

45.3.4 All new and replacement water supply systems shall be constructed to minimize or eliminate infiltration of flood waters into the system.

45.3.5 New and replacement sanitary sewer systems shall be constructed to minimize or eliminate infiltration of flood waters into the systems and exfiltration from the systems into flood waters.

45.3.6 On-site waste disposal systems shall be located to avoid impairment to them or contamination from them during flooding.

45.3.7 Any alteration, major structural repair, reconstruction, or improvement to a structure on which the start of construction was begun after the effective date of this ordinance, shall meet the requirements of "new construction" as contained in this ordinance.

45.3.8 Electrical, heating, ventilation, plumbing, air conditioning equipment, and other service facilities shall be designed and/or located so as to prevent water from entering or accumulating within the components during conditions of flooding.

45.4 - Interpretation of Floodway District and Floodway Fringe District

The Floodway District and Floodway Fringe District are those land areas as specified on the maps identified in Section 45.1 hereof. Any interpretation necessary shall be made by the Engineer of Public Works Department of the City of Huntsville, Alabama.

45.5 - Application of the Floodway District

To enable the district to operate in harmony with the Zoning Ordinance of the City of Huntsville, Alabama, the Floodway District is created as a special district to be superimposed on other existing zoning districts contained in the Zoning Ordinance of the City of Huntsville, Alabama. Except where in conflict with the specific requirements of the Floodway

District, permitted uses, accessory uses, signs, minimum lot requirements, minimum yard requirements, maximum height, and requirements for off-street parking and loading shall be determined by the requirements of the basic district regulations contained elsewhere in this ordinance.

45.6 - Uses Specifically Prohibited in Floodway District

45.6.1 Structures of any type that would obstruct or impede the flow of flood water or cause any increase in the base flood level are hereby specifically prohibited, except as hereinafter provided.

45.6.2 Fill of any type in the floodway.

45.6.3 Manufactured homes (mobile homes) except on existing improved lots in existing manufactured home (mobile home) parks or subdivisions licensed as of the effective date of this ordinance, and then only if such mobile homes are anchored to resist flotation, collapse, or lateral movement by use of over-the-top and frame ties to ground anchors and if the elevation standards as provided for in Section 45.7.2 are met. Any replacement manufactured home may be placed on a lot in an existing manufactured home park or subdivision provided the elevation standards as set forth in Section 45.7.2 are met and provided the replacement is anchored to resist flotation, collapse, or lateral movement by use of over-the-top and frame ties to ground anchors. These standards shall be in addition to and consistent with applicable state requirements for resisting wind forces.

45.7 - Application of the Floodway Fringe District

45.7.1 To enable the district to operate in harmony with the Zoning Ordinance of the City of Huntsville, Alabama, the Floodway Fringe District is created as a special district to be superimposed on other existing zoning districts contained in the Zoning Ordinance

of the City of Huntsville, Alabama. Except where in conflict with specific requirements of the Floodway Fringe District, permitted uses, accessory uses, signs, minimum lot requirements, minimum yard requirements, maximum height, and requirements for off-street parking and loading shall be determined by the requirements of the basic district regulations contained elsewhere in this ordinance.

45.7.2 Residential Construction - New construction or substantial improvement of any residential structure shall have the lowest floor, including basement, elevated to at least one foot above the 100-year flood elevation. When solid foundation perimeter walls are used to elevate a structure, openings sufficient to facilitate the unimpeded movement of flood water shall be provided in accordance with the standards of Section 45.7.4 and must be certified as such to the Building Inspector by a registered professional engineer or registered professional architect.

45.7.3 Non-Residential Construction - New construction or substantial improvement of any commercial, industrial or other non-residential structure shall either have the lowest floor, including basement, elevated to at least one foot above the 100-year flood elevation or, together with attendant utility and sanitary facilities, be flood-proofed so that below the 100-year flood level the structure is watertight with walls substantially impermeable to the passage of water and with structural components having the capability of resisting hydrostatic and hydrodynamic loads and effects of buoyancy. A registered professional engineer or registered architect shall certify to the Building Inspector that the standards of this subsection are satisfied.

45.7.4 Elevated Buildings - New construction of or substantial improvements to elevated buildings that include fully enclosed areas formed by foundation and other exterior walls below the base flood elevation shall be designed to preclude finished living space and to allow for the entry and exit of floodwaters to automatically equalize hydrostatic flood forces on exterior walls.

(1) Designs for complying with this requirement must either be certified by a registered engineer or architect or meet the following minimum criteria:

(a) Provide a minimum of two openings having a total net area of not less than one square inch for every square foot of enclosed area subject to flooding;

(b) The bottom of all such openings shall be no higher than one foot above grade; and

(c) Such openings may be equipped with screens, louvers, valves or other coverings or devices provided they permit the automatic flow of floodwaters in both directions.

(2) Electrical, plumbing, and other utility connections are prohibited below the base flood elevation;

(3) Access to the enclosed area shall be the minimum necessary to allow for parking of vehicles (garage door) or limited storage of maintenance equipment used in connection with the premises (standard exterior door) or entry to the living area (stairways or elevator); and

(4) The interior portion of such enclosed area shall not be partitioned or finished into separate rooms.

45.7.5 For new manufactured home parks and subdivisions; for expansions to existing manufactured home parks and subdivisions; for existing manufactured home parks and subdivisions where the repair, reconstruction or improvement of the streets, utilities and pads equals or exceeds fifty percent of value of the streets, utilities and pads before the repair, reconstruction, or improvement commenced; and for manufactured homes not placed in a manufactured park or subdivision, but which are permitted by the Board of Zoning Adjustment as a variance, the following are required:

(1) Stands or lots elevated on compacted fill so that the lowest floor of the manufactured home will be one foot above the 100-year flood level; and

(2) Each stand or lot shall have proper surface drainage and shall be accessible to the manufactured home and prime mover or hauler.

45.7.6 A registered land surveyor shall submit data to the Building Inspector certifying that the elevation of the lowest floor of a structure in a Floodway Fringe District is in compliance with this ordinance. This certification shall be submitted prior to the floor being placed.

45.8 - Permit Required

45.8.1 It shall be unlawful for any person, firm or corporation to carry out any mining, filling, grading, paving, excavation, or drilling operations in areas designated as Floodway District or Floodway Fringe District without having first obtained a permit for such

activity from the office of the Engineer of Public Works of the City of Huntsville. The Engineer of Public Works shall maintain an official file copy of the "Flood Boundary and Floodway Map" in his office, and shall review applications for permits for any mining, filling, grading, paving, excavation, or drilling operations. Any person failing to comply with the provisions of this section of this ordinance shall be guilty of a misdemeanor and upon conviction shall be fined according to ARTICLE IV. Where any violation is discovered, the Engineer of Public Works shall direct the person, firm, or corporation responsible to restore the land to its prior state before said violation. A separate offense shall be deemed committed on each day on which a violation occurs or continues. The Engineer of Public Works shall take whatever steps necessary to enforce this section of this ordinance.

45.8.2 No building permit will be issued where any part of the property lies within any flood hazard area until plans for such improvements have been reviewed and approved by the Engineer of Public Works.

45.8.3 In unnumbered "A" zones of the "Flood Hazard Boundary Maps" of September 17, 1976, where FIA has not yet provided base flood elevation data, the Engineer of Public Works will obtain, review, and reasonably utilize any other flood data available from a federal, state, or other source for all new construction and/or substantial improvements in such areas in order to set minimum first floor elevations, including basements.

45.9 - Standards for Areas of Shallow Flooding (AO Zones)

Located within the areas of special flood hazard districts established in Section 45.1 are areas designated as shallow flooding areas. These areas have special flood hazards associated with base flood depths of one to three feet where

a clearly defined channel does not exist and where the path of flooding is unpredictable and indeterminate; therefore, the following provisions apply:

45.9.1 All new construction and substantial improvements of residential structures shall have the lowest floor, including basement, elevated to the depth number specified on the Flood Insurance Rate Map, in feet, above the highest adjacent grade. If no depth number is specified, the lowest floor, including basement, shall be elevated at least two (2) feet above the highest adjacent grade.

45.9.2 All new construction and substantial improvements of non-residential structures shall:

(a) have the lowest floor, including basement, elevated to the depth number specified on the Flood Insurance Rate Map, in feet, above the highest adjacent grade. If no depth number is specified, the lowest floor, including basement, shall be elevated at least two (2) feet above the highest adjacent grade;

(b) together with attendant utility and sanitary facilities be completely flood-proofed to or above that level so that any space below that level is watertight with walls substantially impermeable to the passage of water and with structural components having the capability of resisting hydrostatic and hydrodynamic loads and effects of buoyancy.

45.10 - Standards for Streams Beyond the FIA Study Limits Without FIA Established Base Flood Elevations and/or Floodways

45.10.1 The following provisions shall apply to streams located beyond the FIA Study Limits established in Section 45.1, when streams

exist but where base flood data and floodway data are not available.

(1) A registered professional engineer shall submit sufficient engineering data to the Engineer of Public Works showing base flood discharges, base flood elevations, and floodway data along the stream channel. The Engineer of Public Works shall request as much information as deemed necessary to determine and set base flood elevations, floodway widths and regulatory minimum finish floor elevations along the stream channel.

(2) Minimum Finished Floor Requirements - New construction or substantial improvements of structures shall be elevated or flood-proofed to meet the requirements of Section 45.7 based on the established regulatory flood elevations set by the Engineer of Public Works as determined in paragraph 45.10.1(1) above.

(3) Floodway District - Section 45.6 shall apply to the established Floodway set by the Engineer of Public Works determined as described in paragraph 45.10.1(1).

45.10.2 When sufficient engineering data is currently available to the Engineer of Public Works to establish regulatory flood elevations and floodway data along the stream channel, the following provisions shall apply:

(1) The Engineer of Public Works shall provide the required minimum finished floor elevations and/or the location and width of the Floodway District from information and data from his records and files.

(2) Minimum Finished Floor Requirements - New construction or substantial improvements of structures shall be elevated or flood-proofed to meet the requirements of Section 45.7 based on the established regulatory flood elevations set by the Engineer of Public Works as determined in paragraph 45.10.2(1).

(3) Floodway District - Section 45.6 shall apply to the established Floodway set by the Engineer of Public Works determined as described in paragraph 45.10.2(1).

45.11 - Standards for Subdivision Proposals

45.11.1 All subdivision proposals shall be consistent with the need to minimize flood damage;

45.11.2 All subdivision proposals shall have public utilities and facilities such as sewer, gas, electrical and water systems located and constructed to minimize flood damage;

45.11.3 All subdivision proposals shall have adequate drainage provided to reduce exposure to flood hazards; and

45.11.4 Base flood elevation data shall be provided for subdivision proposals and other proposed developments to include manufactured home parks and subdivisions.

45.12 - Variances

45.12.1 The Engineer of Public Works shall review petitions for variances and report his findings to the Board of Zoning Adjustment.

45.12.2 Variances shall not be granted within any designated Floodway District if any increase in flood levels during the base flood discharge would result or if hazardous stream flow velocities would result, except under the following conditions.

45.12.3 Conditions for Variances

(1) Variances shall only be issued:

(a) Upon a determination that the variance is the minimum necessary, considering the flood hazard, to afford relief.

(b) Upon a showing of good and sufficient cause.

(c) Upon a determination that failure to grant the variance would result in unnecessary hardship unique and different from that shared or suffered by adjacent and similar property also in the designated floodway.

(d) Upon a determination that the granting of a variance will not result in increased flood heights, hazardous streamflow velocities, additional threats to public safety, extraordinary public expense, nor create nuisances, cause fraud on or victimization of the public, or conflict with existing local laws or ordinances with respect to the use to which the property is to be put.

(e) For any building or structure in existence prior to the effective date of these Floodway District Regulations that is hereafter destroyed or substantially damaged to 50 percent or more of its replacement cost at the time of destruction by any means, if the reconstruction does not exceed the volume and external dimensions of the original structure and does not offer any greater obstruction to

the flow of floodwaters than did the original structure.

(f) For the reconstruction, rehabilitation or restoration of structures listed on the National Register of Historic Places or the State Inventory of Historic Places without regard to the procedures set forth in this section provided the proposed reconstruction, rehabilitation, or restoration will not result in the structure losing its historical designation.

(g) For new construction on an isolated lot of one-half acre or less in size contiguous to and surrounded by lots with existing structures constructed below the base flood level in residential subdivisions recorded prior to the effective date of this ordinance.

45.12.3 Standards for Evaluating Variance Applications by the Board of Adjustment.

In passing upon such applications, the Board of Adjustment shall consider all technical evaluations, all relevant factors, all standards specified in other sections of this ordinance, and:

(1) the danger that materials may be swept onto other lands to the injury of others;

(2) the danger to life and property due to flooding or erosion damage;

(3) the susceptibility of the proposed facility and its contents to flood damage and the effect of such damage on the individual owner;

(4) the importance of the services provided by the proposed facility to the community;

(5) the necessity of the facility to a waterfront location in the case of a functionally dependent facility;

(6) the availability of alternative locations not subject to flooding or erosion damage for the proposed use;

(7) the compatibility of the proposed use with existing and anticipated development;

(8) the relationship of the proposed use to the comprehensive plan and flood plain management program for that area;

(9) the safety of access to the property in times of flood for ordinary and emergency vehicles;

(10) the expected heights, velocity, duration, rate of rise and sediment transport of the flood waters and the effects of wave action, if applicable, expected at the site; and

(11) the costs of providing governmental services during and after flood conditions including maintenance and repair of public utilities and facilities such as sewer, gas, electrical, and water systems, and streets and bridges.

45.12.4 Upon consideration of the factors listed above and the purposes of this ordinance, the Board of Zoning Adjustment may attach such conditions to the granting of variances as it deems necessary to further the purposes of this ordinance.

45.13.5 Interpret the exact location of boundaries of the areas of special flood hazard (for example, where there appears to be a conflict between a mapped boundary and actual field conditions). The person contesting the location of the boundary shall be given a reasonable opportunity to appeal the interpretation as provided in this article;

45.13.6 Obtain, review and reasonably utilize any base flood elevation and floodway data available from a federal, state or other source in order to administer the provisions of this ordinance when base flood elevation data or floodway data are not available on the official maps;

45.13.7 All permits specified in Section 45.8 shall be maintained in the office of the Engineer of Public Works and shall be open for public inspection.

45.14 - Duties and Responsibilities of the Building Inspector

45.14.1 When flood-proofing is utilized for a particular structure, the Building Inspector shall obtain certification from a registered professional engineer or architect;

45.14.2 Building permits pertaining to the provisions of this ordinance shall be maintained in the office of the Building Inspector and shall be open for public inspection.

45.14.3 Require a registered land surveyor to verify the actual elevation (in relation to mean sea level) of the lowest floor (including basement) of all new or substantially improved structures.

45.14.4 Require a registered land surveyor to verify the actual elevation (in relation to mean sea level) to which the new or substantially improved structures have been flood-proofed.

45.15 - Warning and Disclaimer of Liability

45.15.1 The degree of flood protection required by this ordinance is considered reasonable for regulatory purposes and is based on scientific and engineering considerations. Larger floods can and will occur on rare occasions. Flood heights may be increased by man-made or natural causes. This ordinance does not imply that land outside the areas of special flood hazard or uses permitted within such areas will be free from flooding or flood damages. This ordinance shall not create liability on the part of Huntsville, Alabama, or by any officer or employee thereof, for any flood damages that result from reliance on this ordinance or any administrative decision lawfully made thereunder.

45.15.2 The City Planning Department will submit an Annual Report to the Administrator of the Federal Insurance Administration regarding the Federal Flood Insurance Program. (79-239).

Adopted by the City Council and approved by the Mayor on June 30, 1987.

The original document is signed by the following:

John Glenn - President of the City Council, Huntsville, Alabama

Joe W. Davis - Mayor, City of Huntsville, Alabama

APPENDIX B
Stormwater Management Ordinance

ORDINANCE NO. _____

BE IT ORDAINED by the City Council of the City of Huntsville, Alabama as follows:

Section 1. This ordinance shall be known as the "Stormwater Management Ordinance" for the City of Huntsville, Alabama.

Section 2. Purpose

It is the purpose of this ordinance to control stormwater drainage facilities and land disturbance activities within the corporate limits of the City of Huntsville, Alabama, so as to promote the public health, safety and general welfare through provisions designed to:

1. Protect human life and health.

2. Reduce expenditure of public money for flood control projects.

3. Reduce the need for rescue and relief efforts associated with flooding.

4. Provide for the sound use and development of flood-prone areas so as to maximize beneficial use without increasing flood hazard potential.

5. Reduce damage to public facilities and utilities such as water and sewer lines; electric, telephone, and gas facilities; and streets and bridges located in flood plains.

6. Ensure a functional stormwater drainage system that will not result in excessive maintenance costs.

7. Encourage the improvement of existing flooding problems in conjunction with new development.

8. Encourage the use of natural and aesthetically pleasing design.

9. Reduce the impact on public and private property caused

by the accumulation of mud, dirt, water, debris and other construction materials.

Section 3. Definitions

Unless specifically defined below, words or phrases used in this ordinance and in the regulations and technical guidelines promulgated pursuant thereto shall have their common dictionary definitions or meanings they have in common usage.

Addition (to an existing building)

Any walled and roofed expansion to the perimeter of a building in which the addition is connected by a common load-bearing wall other than a fire wall. Any walled and roofed addition which is connected by a fire wall or is separated by independent perimeter load-bearing walls is new construction.

Area of Shallow Flooding

A designated AO Zone on the Flood Insurance Rate Map (FIRM) with base flood depths from one to three feet where a clearly defined channel does not exist, where the path of flooding is unpredictable and indeterminate and where velocity flow may be evident.

Area of Special Flood Hazard

The land in the flood plain subject to a one percent or greater chance of flooding in any given year.

Base Flood

The flood having a one percent chance of being equaled or exceeded in any given year.

Basement

That portion of a building having its floor subgrade (below ground level) on all sides.

Building

Any structure having a roof supported by columns or by walls.

Channel

A natural or artificial watercourse of perceptible extent, with definite bed and banks to confine and conduct continuously or periodically flowing water. Channel flow is that water which is flowing within the limits of the defined channel.

Cut

Portion of land surface or area from which earth has been removed or will be removed by excavation; the depth below original ground surface to the excavated surface.

Detention

The temporary delay of storm runoff prior to discharge into receiving waters.

Developer

Any individual, firm, corporation, association, partnership or trust involved in commencing proceedings to effect development of land for himself or others.

Development

Any man-made change to improved or unimproved real estate; including, but not limited to, buildings or other structures, mining, dredging, filling, grading, paving, excavating, drilling operations or permanent storage of materials.

Drainage Basin

A part of the surface of the earth that is occupied by and provides surface water runoff into a drainage system, which consists of a surface stream or a body of impounded surface water, together with all tributary surface streams and bodies of impounded surface water.

Engineer of Public Works

The Engineer of Public Works for the City of Huntsville, Alabama or his authorized representative.

Erosion

The disintegration or wearing of soil by the action of water.

Excavation

See cut.

Fill

Portion of land surface or area to which soil, rock or other materials have been or will be added; height above original ground surface after the material has been or will be added.

Flood or Flooding

A temporary condition of partial or complete inundation of normally dry land areas from the overflow of inland waters or the unusual and rapid accumulation or runoff of surface waters from any source.

Flood Insurance Study

The official report provided by the Federal Emergency Management Agency. The report contains flood profiles, as well as the Flood Boundary Floodway Map and the water surface elevation of the base flood.

Floodway

The channel of a river or other watercourse and the adjacent land area that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than one foot.

Floodway Fringe

That portion of the flood plain lying outside the floodway.

Floor

The top surface of an enclosed area in a building (including basement), i.e., top of slab in concrete slab construction or top of wood flooring in wood frame construction. The term does not include the floor of a garage used solely for parking vehicles.

Highest Adjacent Grade

The highest natural elevation of the ground surface, prior to construction, next to the proposed walls of a structure.

Grading

Any operation or occurrence by which the existing site elevations are changed; or where any ground cover, natural or man-made, is removed; or any watercourse or body of water, either natural or man-made is relocated on any site, thereby creating an unprotected area. This includes stripping, cutting, filling, stockpiling, or any combination thereof, and shall apply to the land in its cut or filled condition.

Impervious Surface

A term applied to any ground or structural surface that water cannot penetrate or through which water penetrates with great difficulty.

Lowest Floor

The lowest floor of the lowest enclosed area (including basement). An unfinished or flood resistant enclosure, usable solely for parking of vehicles, building access, or storage and in an area other than the basement area, is not considered a building's lowest floor, provided that such an enclosure is not built so as to render the structure in violation of the requirements of this ordinance or of the regulations and technical guidelines promulgated pursuant thereto.

Retention

The prevention of storm runoff from direct discharge into receiving waters. Examples include systems which discharge through percolation, exfiltration, filtered bleed-down and evaporation processes.

Sediment

Solid material, both mineral and organic, that is in suspension, is being transported, or has been moved from its site of origin by air, water or gravity as a product of erosion.

Site

All contiguous land and bodies of water in one ownership, graded or proposed for grading or development as a unit, although not necessarily at one time.

Slope

Degree of deviation of a surface from the horizontal, usually expressed in percent or ratio.

Start of Construction

The actual start means the first placement of permanent construction of a structure (including a manufactured home) on a site, such as the pouring of slabs or footings, installation of piles, construction of columns, or any work beyond the stage of excavation or the placement of a manufactured home on a foundation. Permanent construction does not include land preparation; such as clearing, grading and filling; nor does it include the installation of streets and/or walkways; nor does it include excavation for a basement, footings, piers or foundations or the erection of temporary forms; nor does it include the installation on the property of accessory buildings, such as garages or sheds, not occupied as dwelling units or not part of the main structure.

Stripping

Any activity that removes or significantly disturbs the vegetative surface cover, including clearing and grubbing operations.

Structure

Any combination of materials; including buildings constructed or erected, the use of which requires location on or in the ground or attachment to anything having location on the ground, including, but not limited to, buildings, towers, carports, signs, billboards, and fences, but not including telephone poles and overhead wires, fences less than three (3) feet high, retaining walls or terraces.

Substantial Improvement

Any combination of repairs, reconstruction, alteration, or

improvements to a structure, taking place during the life of a structure, in which the cumulative cost equals or exceeds fifty percent of the market value of the structure. The market value of the structure should be (1) the appraised value of the structure prior to the start of the initial repair or improvement, or (2) in the case of damage, the value of the structure prior to the damage occurring. For the purposes of this definition, "substantial improvement" is considered to occur when the first alteration of any wall, ceiling, floor, or other structural part of the building commences, whether or not that alteration affects the external dimensions of the structure. The term does not, however, include any project for improvement of a structure required to comply with existing health, sanitary, or safety code specifications which are solely necessary to assure safe living conditions.

Watercourse

A channel, natural depression, slough, gulch, stream, creek, pond, reservoir, or lake in which storm runoff and floodwater flows either regularly or infrequently. This includes major drainageways for carrying urban storm runoff.

Section 4. Authority of Engineer of Public Works

a. The Engineer of Public Works, with the approval of the City Council following a public hearing, shall establish written regulations and technical guidelines (and any amendments thereto) as may be necessary to enforce the terms of this ordinance. These regulations and guidelines, and all appendices thereto, shall be filed in the City Clerk's office of the City of Huntsville, Alabama.

b. The Engineer of Public Works shall have the authority to issue grading permits as required under the regulations promulgated pursuant to paragraph (a) of this section. Any grading permit exemption authorized by the Engineer of Public Works under such regulations shall not be construed as relieving the party responsible for the identified activity from making onsite drainage improvements that may be required or from compliance with the Zoning Ordinance of the City of Huntsville or any other applicable laws or regulations.

c. The Engineer of Public Works shall have the authority to review all building permit applications prior to their issuance by the City of Huntsville Inspection Department to insure compliance with the provisions of this ordinance and the regulations and technical guidelines adopted pursuant hereto. Building permit applications are exempted from review by the Engineer of Public Works if the structure which is the subject of such application is to be remodeled within its existing walls or is a residential dwelling on a single lot within an approved subdivision. No building permit, unless exempted as indicated herein, shall be issued by the City of Huntsville Inspection Department until an application for such building permit has been approved by the Engineer of Public Works. The exemptions listed herein shall not be construed as relieving the party responsible for the identified activity from onsite drainage improvements that may be required in accordance with adopted building and construction codes or from compliance with the Zoning Ordinance of the City of Huntsville or any other applicable laws or regulations.

d. The Engineer of Public Works is authorized to enter upon the premises of any land within the City of Huntsville for which a grading or building permit application has been filed to inspect the site before, during, and upon conclusion of any land disturbance activity to determine compliance with permit conditions and the requirements of the regulations and technical guidelines promulgated pursuant to this ordinance.

e. The Engineer of Public Works, or an employee of his department designated by him, is authorized to issue a citation to any person found by him to be in violation of this ordinance or the regulations and technical guidelines promulgated pursuant hereto. The citation shall order the person to appear in the Municipal Court of the City of Huntsville, Alabama, at a date and time certain to answer the charge (s) against him.

f. The Engineer of Public Works shall have the authority to require a performance bond equal to the work to be performed to insure proper construction of any stormwater drainage facility or proper performance of any other land disturbance activity governed by the provisions of this ordinance.

g. The Engineer of Public Works is authorized to issue a "Stop Work Order" with regard to any land disturbance activity that is being done contrary to the provisions of this ordinance, the regulations and technical guidelines promulgated pursuant hereto, or in a dangerous or unsafe manner. Such notice shall be in writing, shall be delivered to the owner of the property, his agent, or the person doing the work, and shall state the conditions under which work may be resumed.

Section 5. Standards

The Engineer of Public Works shall apply the following standards in the enforcement of any regulations and technical guidelines promulgated pursuant to this ordinance:

a. The calculated peak rate of stormwater runoff resulting from a ten-year return period, twenty-four (24) hour duration, type II storm distribution (as defined by the Soil Conservation Service, U.S. Department of Agriculture) shall be no greater after development of a site than before development of a site.

b. No construction, whether by private or public action, shall be performed in such a manner as to materially increase the degree of flooding in its vicinity or in other areas whether by flow restrictions, increased runoff, or by diminishing channel or overbank storage capacity.

Section 6. Stormwater Management Board

a. Creation, number, appointment and terms of office of members, qualifications, removal compensation.

There is hereby created the "City of Huntsville Stormwater management Board" which shall consist of five members appointed by the Mayor and approved by a majority vote of the City Council. Each member shall be appointed for a term of three years, except that in the first instance, one member shall be appointed for a term of three years, two for a term of two years and two for a term of one year and thereafter each member appointed shall serve for a term of three years or until a successor is duly appointed. In addition to

the five regular members provided for in this subsection, two alternate members shall be appointed, in like fashion as the regular members to serve on such board at the call of the chairperson only in the absence of regular members, and while so serving, shall have and exercise the power and authority of regular members. Alternate members shall be appointed to serve for three year terms. All members shall be eligible for reappointment. Two of the regular members and one alternate shall be registered civil engineers. One of the regular members shall be a hydrologist. The remaining two regular members and one alternate shall be appointed from the community at large. All members shall be residents of the City of Huntsville. No member shall hold an elective office, nor shall any member be an employee of Madison County or the City of Huntsville, Alabama. All members shall serve as such without compensation. No member shall exercise the powers and duties delegated to such member by this ordinance in connection with any matter pending before the Board in which such member has a pecuniary or direct interest or would be financially affected by the final result. Members may be removed by the Mayor for continued absence from meetings of the Board or for other just cause.

b. Organization

Within ten days after its appointment, the Stormwater Management Board shall elect one of its members as chairperson and another as vice chairperson. The Engineer of Public Works shall appoint the secretary to the board who shall be the custodian of the minutes and records of the proceedings of the board.

c. Duties and Procedure

The Stormwater Management Board shall adopt such rules and regulations as it may deem necessary to conduct its business. The board shall hear all appeals as provided in this section. The board shall meet at regular, monthly intervals, with the day and time to be determined by the chairperson. In the event no appeals have been filed and there is no business pending, the chairperson may cancel the meeting at any time after the seventh day before its scheduled date.

d. Quorum, Voting and Conflict of Interest

Four members of the Stormwater Management Board shall constitute a quorum. The concurring vote of four members of the board shall be required to reverse any order, requirement, decision or determination of the Engineer of Public Works or to decide in favor of the applicant on any matter upon which it is required to pass under or to effect any variation in this ordinance or the regulations and technical guidelines adopted pursuant thereto. No member of the board shall vote on any matter in which he has a direct or indirect pecuniary or personal interest. The alternate member, if available, shall replace any member who has a conflict of interest.

e. Appeals

(1) Appeals to the Stormwater Management Board may be taken by any person aggrieved by a decision of the Engineer of Public Works arising out of the regulations and technical guidelines promulgated pursuant to this ordinance. A notice of appeal must be filed with the Engineer of Public Works and the Stormwater Management Board within 15 days after such decision of the Engineer of Public Works, specifying the grounds thereof. The Engineer of Public Works shall transmit to the board all papers constituting the record upon which the decision appealed from was taken. The board shall give public notice of the hearing of the appeal, as well as due notice to the parties in interest, and shall decide the same within a reasonable time.

(2) A decision of the Stormwater Management Board (a) reversing or varying any order, requirement, decision or determination of the Engineer of Public Works or (b) in favor of the applicant on any matter upon which it is required to pass shall be in writing and shall specify in what manner such variations shall be made, the conditions upon which they are to be made and the reasons therefor.

(3) Every decision of the Stormwater Management Board shall be final, subject however, to such remedy as any aggrieved party may have at law or in equity. Every decision shall indicate the vote of each member of the board upon the decision and shall be entered into the

board's minutes and filed in the office of the Engineer of Public Works. A copy of each decision shall be delivered by mail or otherwise to the appellant.

f. Powers

The Stormwater Management Board shall have the following powers:

(1) To hear and decide appeals where it is alleged there is error in any order, requirement, decision or determination made by the Engineer of Public Works in the enforcement of this ordinance or any regulation or technical guideline promulgated pursuant thereto;

(2) To authorize, upon appeal in specific cases, such variance from the terms of this ordinance, or any regulation or technical guideline promulgated pursuant thereto, as will not be contrary to the public interest, where owing to special conditions, a literal enforcement of the provisions of this ordinance, or of any regulation or technical guideline promulgated pursuant thereto, will result in unnecessary hardship.

In deciding appeals under paragraph (2) of this subsection, the Stormwater Management Board shall consider all relevant factors; including, but not limited to, all technical evaluations, all standards specified in the regulations and technical guidelines promulgated pursuant to this ordinance, and:

a. The danger that materials may be swept onto other lands to the injury of others.

b. The danger to life and property due to flooding or erosion damage.

c. The susceptibility of the proposed facility and its contents to flood damage and the effect of such damage on the individual owner.

d. The importance of the services provided by the proposed facility to the community.

e. The necessity of the facility to a waterfront location, in the case of a functionally dependent facility

f. The availability of alternative locations, not subject to flooding or erosion damage, for the proposed use.

g. The compatibility of the proposed use with existing and anticipated development.

h. The safety of access to the property in times of flood for ordinary and emergency vehicles.

i. The expected heights, velocity, duration, rate of rise, and sediment transport of the floodwaters and the effects of wave action, if applicable, expected at the site.

j. The costs of providing governmental services during and after flood conditions including maintenance and repair of public utilities and facilities; such as sewer, gas, electrical and water systems and streets and bridges.

Upon consideration of the factors listed above, and the objectives of this ordinance, the Stormwater Management Board may attach such conditions to the granting of a variance as it deems necessary to further such objectives.

Section 7. Penalties and Injunctions

a. A violation of this ordinance shall be punishable by a fine of not more than Five Hundred and No/100 Dollars ($500.00) for each and every violation. Every day that a violation continues shall constitute a separate offense.

b. In addition to all other remedies provided by law, the city of Huntsville shall have the right to injunctive relief for any violation of this ordinance.

Section 8. Severability

The provision of this ordinance are severable. If any part of this ordinance is declared to be invalid or unconstitutional, such declaration shall not affect the part which remains.

ADOPTED THIS THE _____ DAY OF _____, 1991.

_____
President of the City Council of
the City of Huntsville, Alabama

APPROVED THIS THE _____ DAY OF _____, 1991.

_____
Mayor of the City of
Huntsville, Alabama