WEST VIRGINIA DEPARTMENT OF TRANSPORTATION

Division of Highways

1900 Kanawha Boulevard East • Building Five • Room 110
Charleston, West Virginia 25305-0430 • 304/558-3505

Joe Manchin III
Governor

February 2008

MEMORANDUM

TO: ALL HOLDERS OF WVDOH DRAINAGE MANUAL

FROM: GREGORY L. BAILEY, P. E., DIRECTOR
ENGINEERING DIVISION

Gregory Bailey
2-19-08

SUBJECT: 2007 Drainage Manual – 3rd Edition

Attached is the copy of the 3rd Edition of the WVDOH Drainage Manual which replaces the 1984 Edition. The content of this publication was finalized in the year of 2007. The date of implementation for the content of this publication shall be determined by one of the following:

- **This manual shall apply to any project for which the drainage analysis and design has yet to begin.**

- **The entire manual or parts of the manual shall apply to any project which the engineering and design is ongoing at the discretion of the project manager. A note shall be provided on the contract documents "General Notes" sheet and in the design documentation as to the point and manner of the manuals use.**

Addendum one is already under development. It will include a detailed description and instructions for the process of designing a storm water detention basin with a design example. Example problems for hydrology, inlet spacing, storm sewer design, ditch design, and culvert design are either planned or are currently in development.

An electronic version of this manual is available on the engineering and publications web site at www.wvdot.com. Revisions to this manual will be distributed by mail and will be posted on the web site. This web site contains the publications used for engineering and construction and is updated frequently.

Any comments, complaints, or suggestions regarding the 3rd Edition or the future addendums to the manual shall be sent to drainage.manual@dot.state.wv.us. Questions about the content or use of the 3rd Edition should be directed to the Hydraulics and Drainage Unit at (304) 558-9756 or (304) 558-9696.

GLB:Fc

Attachment

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION
Division of Highways
1900 Kanawha Boulevard East • Building Five • Room 110
Charleston, West Virginia 25305-0430 • (304) 558-3505

Earl Ray Tomblin
Governor

Paul A. Mattox, Jr., P. E.
Secretary of Transportation/
Commissioner of Highways

May 2, 2012

MEMORANDUM

TO: All Holders of West Virginia Division of Highways Drainage Manual, 3rd Edition

FROM: Gregory L. Bailey, P. E. Gregory Bailey
Director
Engineering Division

SUBJECT: Revisions to the West Virginia Division of Highways Drainage Manual, 3rd Edition

Attached for your use are revisions to the West Virginia Division of Highways Drainage Manual, 3rd Edition, published in 2007. Please remove and destroy the earlier versions. This manual is also available on the West Virginia Division of Highways website at http://www.transportation. wv.gov/highways/Engineering/Pages/default.aspx. The date of implementation for the revisions shall be determined by one of the following:

1. The content shall apply to any project for which the drainage analysis and design has yet to begin.

2. The content shall apply to any project which the engineering and design is ongoing at the discretion of the project manager. A note shall be provided on the contract documents "General Notes" sheet and within the design documentation as the point and manner of the contents use.

The major revisions are described as follows:

Chapter 4 – Provided the update to the USGS Regression equations, the addition of the 1 year return period to the Rational Method IDF curves, and the isopluvial map for the 1 year 24 hour rainfall depth for the TR-55 method.

Chapter 5 – Correction to the narrative above Table 5-1. The allowable flow spread from the curb for a design speed less than 40 mph shall be the shoulder width plus 3 feet into the traveled way.

Chapter 8 – Provided a simplified table for the inlet control constraints, inserted nomographs for HDPEPP pipe, and removed Chart 8-19 for Structural Plate Pipe Arch with 10" corner radius.

Chapter 9 – Provided the update to the West Virginia Code article which denotes the Dam Control Act.

Chapter 10 – Added detail to the causeway material requirements, flow analysis requirements, crest elevation, and pipe size.

Should you need any additional information, please contact Mr. Darrin Holmes at (304) 558-9696, or Mr. Douglas Kirk at (304) 558-9756, both of the Hydraulic and Drainage Unit of the Engineering Division.

GLB:Fjd

Attachments

WEST VIRGINIA DEPARTMENT OF TRANSPORTATION

Division of Highways

1900 Kanawha Boulevard East • Building Five • Room 110
Charleston, West Virginia 25305-0430 • (304) 558-3505

Earl Ray Tomblin
Governor

Paul A. Mattox, Jr., P. E.
Secretary of Transportation/
Commissioner of Highways

March 18, 2016

MEMORANDUM

TO: ALL HOLDERS OF WVDOH DRAINAGE MANUAL

FROM: R. J. Scites, P. E. RJ Scites
Director
Engineering Division

SUBJECT: Drainage Manual Revisions Addendum 2

Please know that the WVDOH Drainage Manual, dated December 2007, has been revised with Addendum 2. The revised sheets replace all previous copies and contain the following revisions as noted below:

- Section 1.4 – Table 1-2 change roughness coefficients for sheet flow to coincide with HDS-2.
- Section 4.2.8.3 – Methods to account for Karst, removed manipulation of initial abstraction values. The curve number tables, within the TR-55 method, use the la=0.2S relationship to relate CN to la.
- Section 4.4.2.5 – Table 4-5 changed roughness coefficients for sheet flow to coincide with HDS-2.
- Section 4.4.3.5 – Corrected subtext for shallow concentrated flow, as it was running together or squished.
- Section 4.4.5.1 – Added new section describing WMS and an applicable runoff curve number table for use with the NLCD data.
- Section 4.4.5.2 – Placed a limit of 20 acres when using the Modified Rational method hydrograph for routing through a basin. Limited use for sizing the storage area for a basin to 200 acres.

- **Section 5.3.3 – Removed reference in text to the 5 foot parameter in the allowable design spread. Table is correct.**

- **Section 8.4.5 – Corrected the last term in Form 1 of the Unsubmerged and Submerged equation for inlet control. The slope term should not be squared. FHWA Hydraulic Design Series 5 corrected this error in 2012.**

- **Section 8.4.6 – Changed subscript for HW above the outlet invert to avoid confusion with HW, due to outlet control. Changed to Hwoi.**

The Drainage Manual is available on the West Virginia Division of Highways website at http://www.transportation.wv.gov/highways/engineering/pages/publications.aspx.

Should you require any additional information, please contact Mr. Todd West via e-mail at todd.g.west@wv.gov.

RJS:Lkc

cc: DDC(TGW, MDL), DD (via email), DD(MF)

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
ENGINEERING DIVISION

DRAINAGE MANUAL
3rd EDITION
DECEMBER, 2007

Documentation Page

Title:

WEST VIRGINIA DIVISION OF HIGHWAYS

DRAINAGE MANUAL 3rd Edition

December 2007

Addendum 1 Februrary 2012

Addendum 2 June 2015

Sponsoring Agency:	Consultant:
WVDOH, Engineering Division, Hydraulic & Drainage Unit 1900 Kanawha Boulevard, East Building 5, Room A-430 Charleston, WV 25305-0430	Michael Baker Jr., Inc. 5088 West Washington Street Charleston, WV 25313

Principal Authors:

West Virginia Division of Highways	Michael Baker Jr. Inc.
Darrin Holmes, PE	Ramesh Chintala, PE, CFM, D.WRE

Technical Assistance provided by:

West Virginia Division of Highways	Michael Baker Jr. Inc
Jason Ritter, EI Kyle Stollings, PE James Sloan, PE Jackie Fields, PE Walt Simmons, EI	Mohiuddin Shaik, PE, CFM Joe Waterman, PE Richard Leininger, PE Sam Hamdan, PE David Preusch, PE, CFM, D.WRE Rubina Tabassum, PE, CFM

Federal Highway Administration

Eric Brown, PhD | Dan Ghere, PE | Joe Krolak, PE

ACEC/WV Drainage Manual Committee
Chairman: David Dee, PE of PB

WVDOH Project Manager:
Doug Kirk, PE

This 3rd Edition of the West Virginia Division of Highways Drainage Manual provides the designer with the needed information and tools to perform drainage analysis and design for highway facilities. Although this is a completely new manual, it does retain many of the principles, policies, criteria and methods from the previous editions. Additional references are provided for drainage situations that require more detailed analysis.

Includes Addendum 1 dated October 2011.
Content included within the addendum is marked within the footer.

Order printed copies or CD's from:
WVDOH, Engineering Division,
Technical Section
1900 Kanawha Boulevard, East
Building 5, Room A-650
Charleston, WV 25305-0430

Cover by Darrin Holmes, adapted from 1963 WVDOH Drainage Manual & art from Goran Milenkovic

TABLE OF CHAPTERS

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 1: INTRODUCTION

TABLE OF CONTENTS

LIST OF TABLES

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CHAPTER 1: INTRODUCTION

1.1 INTRODUCTION

Drainage has long been recognized as one of the essentials of highway construction, operation and maintenance. Public safety and the cost involved in the adequate removal of surface and subsurface water justifies a careful and scientific approach for the design of drainage facilities. This 3rd Edition of the West Virginia Division of Highways Drainage Manual provides the designer with the needed information and tools to perform drainage analysis and design for highway facilities. Although this is a completely new manual, it does retain many of the principles, policies, criteria and methods from the previous editions. Additional references are provided for drainage situations that require more detailed analysis.

The information contained in this manual is based largely on previous publications, including but not limited to the following:

- AASHTO Model Drainage Manual, 1991, 2005
- AASHTO Highway Drainage Guidelines, 1999
- WVDOH Drainage Manual, 1963, 1984
- WVDOH Design Directives
- WVDOH Standard Specifications
- Various FHWA Hydraulic Engineering Circulars
- Various FHWA Hydraulic Design Series
- Virginia DOT Drainage Manual, 2002

1.2 COMPUTER SOFTWARE

This manual replaces previous editions which were issued in 1963 and 1984. Developed in an era when only NASA and a few research scientists had access to computers, the earlier manuals relied heavily on nomographs to expedite drainage calculations. While many of the nomographs have been retained in this edition, the equations that the nomographs were developed from are also included. This will allow designers to develop computer programs and spreadsheets to accurately and expeditiously complete drainage calculations. Computer programs available from government agencies and private businesses are also acceptable. When using computer programs, all input data and results must be presented in a format that is easily understood and acceptable to the Division of Highways. The following table lists acceptable computer programs. The Hydraulic and Drainage Unit maintains this list.

Table 1-1
List of Acceptable Computer Programs

Program Name	**Supplier**
HEC-RAS, HEC-HMS	US Army Corps of Engineers Hydrologic Engineering Center
TR-20, WinTR-55	Natural Resources Conservation Service
HY-8, Visual Urban (HY-22), Hydraulic Toolbox, WMS, HMS	Federal Highway Administration
Culvertmaster, Flowmaster, Civilstorm	Bentley/Haestead
HydroCAD	Applied Microcomputer Systems
RIVERMorph	RIVERMorph, LLC

1.3 GENERAL DRAINAGE CONSIDERATIONS

The following issues should be considered in the drainage design process:

- Degree of current and future development adjacent to the project and in the watershed
- Effect of the proposed project on the existing drainage pattern
- Potential impact of backwater caused by the highway project
- Impact of concentrated flows from the highway on vehicle and pedestrian safety
- Adverse effects to downstream property owners
- Need for permanent drainage easements
- Potential for damage to receiving streams
- Soil permeability
- Presence of karst topography
- Potential impact to social, cultural, environmental and archaeological resources
- Compliance with applicable laws and regulations
- Initial and long-term cost

1.4 MANNING'S ROUGHNESS VALUES

Several chapters in this manual reference Manning's Roughness values (n values). To make it convenient for the designer, n values of various materials were compiled into Table 1-2.

Table 1-2
Table of Manning's Roughness (n) Values

Manning's Roughness Coefficient for Surfaces			
Concrete Surface (smooth)	0.012		
Concrete (rough), Lined Channel or Overlaid Surface	0.013		
Asphalt Surface (smooth)	0.011		
Asphalt Surface (rough), Lined Channel or Overlaid Surface	0.016		
Gravel Surface	0.024		
	May Use Values Between Those Provided Accompany with a Description		
Broken soil, Clean, No Residual Vegetation			0.05
Broken Soil, with Vegetation Cover ≤ 20%			0.06
Sparse Coarse Grass, Shrubby Vegetation, Bare Earth Present, Drier Less Fertile Soil			0.13
Short Grass, No Trees, Drier Less Fertile Soil			0.15
Broken Soil, Vegetation Cover > 20%, Fertile Soil			0.17
Medium Height Grass, Coarse, Possibly Trees, Fertile Soil			0.24
Forest Area, Light Density of Coarse Grass, Some Shrubs, Bushes, Small Trees			0.40
Dense Lawn Grass with Fertile Soil, Typical for a Golf Course			0.41
Forest Area, Dense Amount of Coarse Grass, Shrubs, Bushes, Small Trees			0.80
Manning's Roughness Coefficient for Ditches			
Existing Vegetative Lining	Recommended	Value Range	
Nearly bare, light grass	0.03	0.030 - 0.035	
Grass, weeds, and light brush	0.04	0.030 - 0.050	
Thick grass, thick brush, small trees	0.075	0.050 - 0.100	
Planned DOH Vegetative Lining	Recommended	Value Range	
Type B Seed Mixture (mowed)	0.042	0.036 - 0.050	
Type C-1 Seed Mixture (mowed)	0.036	0.030 - 0.040	
Type C-2 Seed Mixture (mowed)	0.027	0.022 - 0.033	
Type B Seed Mixture (unmowed)	0.09	0.050 - 0.140	
Type C-1 Seed Mixture (unmowed)	0.08	0.050 - 0.120	
Type C-2 Seed Mixture (unmowed)	0.03	0.025 - 0.040	
	Based on Depth of Flow		
Non Vegetative Lining	0 - 0.5'	0.5 - 2.0'	> 2.0'
Concrete Lined Ditch or channel	0.015	0.013	0.013
Grouted Rock Lined Ditch or channel	0.04	0.03	0.028
Bare Soil with little or no vegetation	0.023	0.02	0.02
Bare Rock or Rock Cut Ditch	0.045	0.035	0.025
Rock Lined Ditch or channel D50 = 4 inches	0.09	0.058	0.035
Rock Lined Ditch or channel D50 = 6 inches	0.104	0.069	0.035
Rock Lined Ditch or channel D50 = 12 inches	-	0.078	0.04

Manning's Roughness Coefficient for Pipes			
Corrugated Metal			
		Recommended	Value Range
Annular Circular	2 2/3" x 1/2"	0.024	
Annular Arch	2 2/3" x 1/2"	0.026	
Annular Structural Plate	6" x 2"	0.033	0.028 - 0.033
Annular Structural Plate	9" x 2 1/2"	0.035	0.033 - 0.037
Helical 24" dia. or less	2 2/3" x 1/2"	0.015	0.012 - 0.015
Helical > 24" dia.	2 2/3" x 1/2"	0.023	0.015 - 0.023
Helical	3" x 1"	0.028	0.027 - 0.028
Helical	5" x 1"	0.025	0.024 - 0.026
Spiral Rib Metal	3/4" x 3/4" x 7 1/2"	0.012	0.011 - 0.012
Steel, Non-Galvanized	Smooth	0.015	
Concrete			
Round and Elliptical	Smooth	0.012	0.011 - 0.012
Cast in Place Box	Smooth	0.013	0.012 - 0.015
Pre-Cast Box	Smooth	0.013	0.012 - 0.015
Plastic			
HDPEPP Corrugated	Corrugated	0.023	0.018 - 0.025
PVCPP Corrugated	Smooth Liner	0.010	0.007 - 0.011
HDPEPP Corrugated	Smooth Liner Type F Trench	0.013	0.010 - 0.017
HDPEPP Corrugated	Smooth Liner	0.015	0.013 - 0.022
Other			
Cast Iron	Smooth	0.015	
Clay Sewer	Smooth	0.013	

1.5 CHAPTERS & ADDITIONAL RESOURCES

The manual is composed of the chapters listed below. Additional resources listed below are available from the Federal Highway Administration's website containing current Hydraulics Engineering Publications at: http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm.

Table 1-3
List of Recommended Publications

Chapter	Title	Additional Resources
1	Introduction	• HDS-4, Introduction to Highway Hydraulics, 2008
2	Design Policy	
3	Documentation	
4	Hydrology	• HDS-2, Highway Hydrology, 2002
5	Storm Drainage Systems	• HEC-22, Urban Drainage Design Manual, 2009 • Hydraulic Performance of Curb & Gutter Inlets, 1999
6	Ditches	• HDS-3, Design Charts for Open-Channel Flow, 1961 • HEC-15, Design of Roadside Channels with Flexible Linings, 2005
7	Channels	• HDS-6, River Engineering for Highway Encroachments, 2001
8	Culverts	• HDS-5, Hydraulic Design of Highway Culverts, 2005 • Design for Fish Passage at Roadway-Stream Crossings: Synthesis Report, 2007 • HEC-14, Hydraulic Design of Energy Dissipators for Culverts & Channels, 2006
9	Stormwater Management	• HEC-22, Urban Drainage Design Manual, 2009
10	Bridges	• HDS-6, River Engineering for Highway Encroachments, 2001 • HEC-9, Debris Control Structures Evaluation and Countermeasures, 2005 • HEC-18 Evaluating Scour at Bridges, 2001 • HEC-20, Stream Stability at Highway Structures, 2001 • HEC-21, Bridge Deck Drainage Systems, 1993 • HEC-23, Bridge Scour and Stream Instability Countermeasures, 2009 • Assessing Stream Channel Stability at Bridges in Physiographic Regions, 2006

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 2:
DESIGN POLICY

TABLE OF CONTENTS

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CHAPTER 2: DESIGN POLICY

2.1 INTRODUCTION

This chapter outlines policies that will help the designer give the appropriate level of consideration to the many different variables that influence drainage design. These policies were established to ensure safe, economical and consistent design of highway drainage structures. An adequately designed highway drainage structure is expected to meet the following broad policies:

- The design of the structure is consistent with the West Virginia Division of Highways' (WVDOH) accepted standard of engineering practice, and
- The design approach is one that a "reasonably competent and prudent designer" would follow under similar circumstances.

In this manual, the word "shall" refers to mandatory requirements. The word "should" refers to recommendations that are not mandatory, but are generally accepted as good engineering practice.

Refer to the appropriate chapter for more specific policy and criteria.

2.2 POLICY VS. CRITERIA

Policy and criteria statements are closely interrelated. Criteria are numeric standards that are derived from broad policy statements. Policy drives criteria. The following definitions of policy and criteria will be used in this manual:

Policy - Policy is an officially stated guiding principle that is intended to determine a definite course of action. A policy statement assists in making a judgment or decision pertaining to the design.

Criteria - Design criteria are the specific standards by which a policy is implemented or placed into action. Criteria are needed for design, policy statements are not.

The following is an example of a policy statement:

> *The designer will size drainage structures to accommodate a storm event compatible with the projected traffic volumes.*

The design criteria for designing the structure might be:

> *For projected traffic volumes less than or equal to 400 vehicles per day, drainage structures shall be designed for a 10-year flood (exceedence probability of 10 percent).*

2.3 GENERAL POLICIES

The following general policies shall apply to hydrologic and hydraulic (H&H) design of highway drainage facilities:

- Drainage facilities shall be designed to accommodate the discharge for the minimum specified design storm criteria. Table 4-2 presents the minimum criteria to protect roadways from flooding or damage based on the frequency, return period or the annual probability of occurrence.
- The level of detail of the H&H analysis should be commensurate with the risk associated with the project, roadway classification and traffic volume, scope of the project, proximity to structures or other development, flooding history and size of the stream. For example, an Interstate bridge over the Kanawha River in Charleston will require a HEC-RAS model, while a culvert replacement on a rural county route may require only a visual evaluation by the maintenance supervisor.
- The design storm as well as the check storm shall serve as criteria for evaluating the adequacy of the design. The “design storm” is the storm event with a recurrence interval for which the drainage structure is sized to assure that no traffic interruption or significant damage will result. This will usually be the 10-year (10% annual chance), 25-year (4% annual chance) or 50-year (2% annual chance) storm. The “check storm” is one that is used to review (check) a drainage facility designed to accommodate a lesser design storm in order to judge whether a significant flood hazard due to a storm larger than the proposed design discharge has been overlooked. The “check storm” for a bridge or culvert will generally be the “overtopping flood”, which is the smallest flood that will result in flow over the highway or other watershed boundary. The check storm is most likely to result in the greatest relative backwater at a stream crossing. The magnitude and recurrence interval of the check storm will be site specific. The design storm and the check storm may vary widely depending on the grade, alignment and classification of the road and the characteristics of the water course and floodplain. **It is important to evaluate the check storm to protect adjacent property from increased flood damage and the Division of Highways from liability.**
- The predicted elevation of the 100-year or base flood serves as the present engineering standard for evaluating flood hazards and as the basis for regulating floodplains under the National Flood Insurance Program (NFIP). Projects located within the designated 100-year floodplain shall conform to the NFIP regulations. Regulations governing highways in the floodplain environment are detailed in 23 CFR Part 650, Subpart A, 44 CFR Chapter 1, and 23 CFR Part 771.

2.4 ENGINEERING EVALUATION

The engineering evaluation process through preliminary and final design requires the consideration and balancing of a number of competing factors. These factors include, but are not limited to:

- Federal and State laws and regulations
- Legal considerations
- Highway safety
- Highway durability and maintenance needs
- Flood hazards
- Costs
- Environmental and social concerns
- Site-specific concerns

Policies specific to the above factors are discussed in the following section. Site-specific concerns require due consideration and could sometimes become overriding factors.

2.4.1 FEDERAL AND STATE LAWS AND REGULATIONS

Design and construct all drainage structures and facilities to comply with applicable federal and state laws and regulations that are in effect at the time of construction.

2.4.2 LEGAL CONSIDERATIONS

Avoid unnecessary liability by considering potential impacts to nearby property, structures and natural and cultural resources. In some cases this may require additional engineering analysis beyond the usual standard of care. For example, some culvert installations may require a HEC-RAS model for backwater analysis and some roadway projects may require stormwater detention analysis. (See Chapter 10 and Chapter 9 respectively, for criteria).

2.4.3 HIGHWAY SAFETY

Make a diligent effort to exclude hazardous conditions that may be caused by water in or near the roadway. Design criteria contained within the individual chapters of this manual shall be considered the accepted standard of care for highway safety as it pertains to highway drainage.

2.4.4 HIGHWAY DURABILITY AND MAINTENANCE NEEDS

Design drainage structures and select materials to provide adequate longevity and minimal maintenance.

2.4.5 FLOOD HAZARDS

Roadways, bridges and drainage structures should be designed to avoid adverse flooding impacts. While bridges and their approach roadway embankments are usually evaluated for flooding impacts, independent roadway fills within the floodplain must also be evaluated. Highway encroachments within FEMA designated "floodways" should be avoided if at all possible. Refer to Section 10.3.1 Backwater Increases and NFIP Requirements.

2.4.6 COSTS

With the limited funds available for the highway system, cost is always an important consideration in highway work, including hydraulics and drainage.

2.4.7 ENVIRONMENTAL AND SOCIAL CONCERNS

All drainage structures should be designed to minimize the impacts to the environment and cultural resources. Close coordination with the Environmental Section is essential in achieving this objective.

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 3: DOCUMENTATION

TABLE OF CONTENTS

LIST OF TABLES

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CHAPTER 3: DOCUMENTATION

3.1 INTRODUCTION

Maintaining clear and complete records of the hydrologic and hydraulic calculations are of paramount importance to the drainage design process. This chapter presents the documentation that shall be included in the design files and construction plans. For further information, refer to *CFR 650.117 Content of Design Studies*.

3.2 DEFINITION

Hydrologic and hydraulic documentation as used in this chapter is the compilation and preservation of records consisting of design calculations, related drawings and details. These documents contain information which provide evidence for the basis of design decisions. Such information may include:

- Drainage area and other maps
- Site plan
- Field survey information
- WVDOH Drainage Forms
- Source references
- Photographs, video recordings or field sketches
- Engineering calculations and analyses

3.3 PURPOSE

The purpose of providing good documentation is to record the design procedure that was used and show how the final design decisions were made. These documents record whether the design analyses were reasonable, prudent and based on the best available technology. Good documentation should provide the following:

- A description of the situation at the time of design;
- An explanation of the proposed situation and resultant expectations;

- Documentation that accepted procedures and analysis were used at the time of the design commensurate with the site importance and flood hazard;
- Protection from liability when claims are made against the Division;
- File data necessary to quickly evaluate future site problems that might occur during the facility's service life; and
- Expedient plan development through clearly stated reasons and rationale for specific design decisions.

3.4 RESPONSIBILITY

The designer is responsible for determining what hydrologic analyses, hydraulic design and related information shall be documented during the plan development process. The designer shall determine whether complete documentation has been achieved during the plan development process through final design. The information presented in various chapters of this manual provides the guidance for these determinations. In addition to the usual chain of command, the Hydraulic and Drainage Unit may also assist in the decision making process.

3.5 PROCEDURE

3.5.1 INTRODUCTION

Based on the type of design elements present in the project, drainage documentation can generally be of two kinds: Drainage Calculations for roadway design projects and Hydrologic & Hydraulic (H&H) Reports for bridges and roadway projects located within floodplains regulated by FEMA. Where practicable, both kinds of reports could include items such as:

- identification and location of the facility;
- photographs (ground and aerial);
- hydrologic analyses;
- hydraulic analyses;
- drainage area maps, vicinity maps, and topographic maps;
- contour maps;
- interviews (local residents, adjacent property owners, and maintenance crews);
- newspaper clippings;

- design notes and correspondence relating to design decisions;
- history of performance of existing structure(s); and
- design assumptions.

3.5.2 DRAINAGE CALCULATIONS

Drainage Calculations should contain data and calculations that influenced the design of the drainage facility. The following sections shall generally be included in the Drainage Calculations.

3.5.2.1 PROJECT DESCRIPTION

The following items pertaining to the hydrologic analysis should be included:

- contributing watershed area and identification of source (map name, etc.);
- design frequency and basis for selection;
- design discharge; and
- expected level of development in upstream watershed over the anticipated life of the facility (include sources and basis for these development projections).

3.5.2.2 CULVERTS

The following items should be included:

- allowable headwater elevation and basis for its selection;
- cross section(s) used in the design high-water determinations;
- roughness coefficient estimation (Manning’s “n” values);
- pH, resistivity and sulfate content of soil and water
- observed high water elevations, dates and discharges, if readily available;
- type of culvert entrance condition;
- culvert outlet appurtenances and energy dissipation calculations and designs;
- copies of all computer analyses and the standard computation form provided in the Culverts Chapter of this manual;
- roadway geometry (plan and profile);

- potential of flood hazard to adjacent properties; and
- durability considerations (corrosion and abrasion)
- debris accumulation potential
- fish passage or other applicable environmental considerations
- tailwater elevation calculations
- observed channel instabilities (lateral and vertical).

3.5.2.3 STORM DRAINS

The following items should be included:

- computations for inlets and storm drains
- computations of the hydraulic grade line, if required by the project manager;
- copies of the standard computation forms provided in the Storm Drainage Systems Chapter of this manual;
- complete drainage area map;
- design frequency;
- information concerning outfalls and existing storm drains; and
- schematic layout of the storm drain system.

3.5.2.4 ROADSIDE DITCHES

The following items should be included:

- description of underlying soil and rock material;
- drainage areas for each outlet location;
- ditch plan and profile layout;
- ditch cross-sections;
- roughness coefficient estimation (Manning's "n" values);
- flow capacity and ditch sizing calculations;
- ditch lining calculations;
- analysis of outlet points and downstream effects; and
- copies of standard computation sheets provided in the Ditches Chapter.

3.5.2.5 OPEN CHANNELS

If the Corps of Engineer's HEC-RAS computer program is used for open channel design, the outline included in Section 3.5.3 should be followed.

Otherwise, the following items should be included:

- cross section(s) used in the design water surface elevation determinations and their locations;
- roughness coefficient estimation (Manning's "n" values);
- information on the method used for design water surface determinations;
- observed high water elevations, dates and discharges, if readily available;
- channel velocity measurements or estimates and locations;
- water surface profiles for the design storm, bankfull event, and 100-year storm;
- design or analysis of channel protection lining materials proposed for the channel bed and banks;
- planting plan;
- in-stream structures;
- energy dissipation calculations and design; and
- copies of all computer analyses.

3.5.3 HYDROLOGIC & HYDRAULIC REPORTS FOR BRIDGES

The following outline should be followed for H&H Reports for Bridges. It is assumed that the HEC-RAS computer program will be used to analyze bridge hydraulics.

3.5.3.1 PROJECT DESCRIPTION

A. Narrative

1. State Project Number, Federal Project Number, Bridge Name and Design Number
2. Town and County
3. Stream
4. Watershed

5. Route Number
6. Purpose of Project
7. Flood impact or stream relocation impact mitigation commitments in the Record of Decision for EA or EIS
8. Existing Bridge Description

B. Location Maps
1. County Map
2. USGS Topographic Map
3. Situation Plan

C. Field Observations
1. Normal Water Surface Elevation
2. High Water Marks
3. Features relevant to the hydraulic analysis
4. Verification of Manning's 'n' values
5. Visual Indicators of Stream Stability

D. Photographs

3.5.3.2 SUMMARY OF RESULTS

A. Design Alternates being carried forward

B. Table of Water Surface Elevations, Including Duplicate Effective, Corrected Effective, Proposed Alternates (Include separate table for temporary construction condition)

C. Compliance with FEMA and WVDOH criteria

D. Scour depths for 100-year and 500-year storms (include with final report)
1. Input Data
2. Calculations in accordance with FHWA's HEC-18
3. Results based on calculations and rock depth
4. Countermeasures, if necessary

E. Recommendation
1. Proposed Bridge
2. Existing Bridge – (remove or remain)

   3. Compensatory excavation, auxiliary structures, etc.

F. Signature Block, Consultant or In-House Designers

   1. Preparer

   2. Reviewer

   3. Engineers Seal of Preparer or Reviewer on Final Report

   4. Date

3.5.3.3 AVAILABLE DATA

A. FEMA Flood Insurance Study

B. Bridge Inspection Reports (include relevant pages only)

C. Existing Hydrologic Data

D. Existing Hydraulic Model from FEMA, USACE, NRCS, others

3.5.3.4 HYDROLOGY

A. Design Discharge determination based on Chapter 4, Section 4.3.3 (formerly DD-504)

B. Explain the selected Boundary Conditions

C. Hydrologic Model, if used (TR-20, HEC-HMS)

D. Effects on calculated design discharge due to existing flood control reservoir in watershed

E. Determination of Normal Flow for temporary (construction) condition modeling

3.5.3.5 HYDRAULIC MODELING

A. Source of Model

B. Site Map with Cross Sections, tied to the project mapping

C. Justification of Model Extent

D. Explanation of Data and Methods

   1. Manning's Values

   2. Bridge Modeling Approach

   3. Ineffective Flow Areas

   4. Any Unusual Circumstances

   5. Table of HEC-RAS Plan, Geometry and Flow Files

E. HEC-RAS Generated Tables

   1. Standard Table 1: Existing and Temporary Condition During Construction for the following profiles:

      a. Normal Flow
      b. Bankfull Storm (if determined)
      c. 2-Year Storm
      d. 10-Year Storm
      e. 100-Year Storm

   2. Standard Table 1: Existing and Proposed Condition for the following profiles:

      a. Bankfull Storm (if determined)
      b. 2-Year Storm
      c. 10-Year Storm
      d. 25-Year Storm
      e. 50-Year Storm
      f. 100-Year Storm
      g. 500-Year Storm

   3. Bridge Output Table – 100-Year Storm

      a. Existing
      b. Proposed

F. HEC-RAS Generated Plots

   1. Profile
   2. Bridge Internal Cross Sections

G. CD with HEC-RAS files

3.5.3.6 DO NOT INCLUDE:

- HEC-RAS generated reports (providing the data files on CD makes this unnecessary)
- Recommendations regarding Conditional Letter of Map Revision (CLOMR) request to FEMA. This may be included in a separate cover letter to the Division of Highways.

3.5.4 COMPUTER FILES

The following electronic computer files should be included along with the Drainage and H&H Reports:

- input data
- output results for existing conditions and selected alternatives

3.5.5 STORAGE

Methods of data storage will continue to change as technology advances. Ease of access, durability, legibility, storage space, and cost are some of the factors that should be considered in selecting a storage method.

Final Drainage Calculations and Hydrology & Hydraulic Reports prepared by consultants shall be saved to a CD in PDF format. The CD shall also contain all electronic files for any computer analyses that were used. The project manager shall forward the CD to the Engineering Division, Administration Section.

Final Drainage Calculations and Hydrologic & Hydraulic Reports prepared by WVDOH in-house staff shall be saved to the WVDOT shares server under \\Wvdot-shares\HydraulicsPermitting\Hydraulics, in the appropriate District folder.

3.5.6 CONSTRUCTION PLANS

Key hydraulic design data shall be documented in the construction plans as follows. See also DD-706.II.C.8.

Drainage areas (DA) less than 50 acres and the discharges (Q) less than 50 cfs should be reported to the nearest tenths of the data value. Areas larger than 50 acres and discharges greater than 50 cfs should be rounded to the nearest whole number.

Velocity (V) and depth (D) should be reported to the nearest tenths. Headwater elevations should be reported to the nearest hundredths. The elevation of the flood of record should be reported to the nearest tenths.

The X to the left of the decimal point represents the data value. The Xs to the right of the decimal point represent the number of digits. The subscript ($_X$) represents the design frequency event number.

- Major Culverts (36" diameter and greater): Show contour line and label for "Q_X HW ELEV. = X.XX FT" on the plan sheet and record the following data on the culvert profile:

DA = X.X ACRES or X ACRES

Q_X = X.X CFS or X CFS

Q overtopping = X.X CFS or X CFS

V_X = X.X FPS

HW = X.XX FT

Q_X HW ELEV. = X.XX FT

FLOOD OF RECORD ELEVATION ((MONTH/DAY/YEAR)) = X.X FT

- Major Storm Drain Outlets: Show the following data on the storm drain profile near the outlet:

DA = X.X ACRES or X ACRES

Q_X = X.X CFS or X CFS

V_X = X.X FPS

- Channel Relocations: Show the following data on the channel relocation profile near the beginning of the relocation:

DA = X.X ACRES or X ACRES

Q_X = X.X CFS

V_X = X.X FPS

D = X.X FT

Bankfull channel data, if Natural Stream Channel Design (NSCD) is used

- Hydraulic Bridges: Show the following data on the profile in the situation plan and profile sheet:

DA = X.X ACRES (X.X SQ MI when DA exceeds 2000 acres)

Q_X = X.X CFS (source e.g. FEMA, CALCULATED, etc.)

Q overtopping = X.X CFS or X CFS

V_X = xx.x FPS (source)

Q_X ELEV = X.XX (source) (also show line for this and label on the profile

3.5.7 SUBMISSION SCHEDULE

In accordance with Design Directive-202 (DD-202), hydraulic and drainage information shall be included in the submissions for each progressive scheduled review.

Table 3-1

Submission Schedule

Design Report Field or Office Review	• Plan & Profile Sheets - Channel Change Requirements with NSCD Features shown • Major Drainage (DD-706)
Preliminary Field Review (PFR)	• Channel Change Requirements with NSCD Features shown • Major Drainage Pipe Profiles (DD-706) • Major Drainage Calculations
Slope Review	• Same as Design Report Review
Final Field Review (FFR)	• Channel Change Requirements with NSCD Features shown • All Drainage Including Pipes, Pipe Profiles, Ditches and Underdrains Shown (DD-706) • Complete Drainage Calculations
Final Office Review (FOR)	• Same as FFR plus • Drainage Data Noted on Plan & Profile Sheets
Span Arrangement	• Hydraulic Data Plotted on Profile • Waterway Opening, and Appropriate Storm Frequency Elevations • Preliminary Hydraulic Study • Freeboard Documented • Navigational Clearance Requirements • Listing of Proposed Computer Software • Listing of Deck Drainage Requirements
Type, Size & Location (TS&L)	• Hydraulic Data Plotted on Profile • Scour Depths Shown • Final Hydraulic Study • Scour Analysis Including Completed DS-34 • Freeboard • Navigational Clearance Requirements

	• Conceptual Deck Drainage Design for Each Alternate • Deck Drainage Requirements
Bridge Rating	• Completed DS-34 "Scour Evaluation Summary"
Final Detail Bridge Plans	• Details Sheets - Deck Drainage Details Shown, Including Scuppers, Deck Inlets, Piping System, and Location and Details of Discharge Points • Plan & Profile Sheets – Stream Flow Direction, Low Water Surface Elevation, Hydraulic Data Plotted, Scour Depth Shown for 100 year Storm Event • Deck Slab – Deck Drain Locations Shown • Situation Plan – Hydraulic Data Shown
Final Tracings	• Final Completed Drainage Calculations

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 4:
HYDROLOGY

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

DRAINAGE COMPUTATION FORMS

DRAINAGE COMPUTATION WORKSHEETS

LIST OF MAPS

LIST OF CHARTS

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CHAPTER 4: HYDROLOGY

4.1 INTRODUCTION

Hydrology is defined as a science dealing with the properties, distribution, and circulation of water on and below the earth's surface and in the atmosphere. This chapter provides discussion of estimating the flow magnitude as a result of rainfall over a watershed.

Flow magnitude is usually expressed as the peak rate of runoff in cubic feet per second. The terms discharge, flow and runoff are used interchangeably, and refer to the rate or volume of water moving past a location per unit time.

Discharge calculations in drainage design are analogous to the design load calculation in structural design. Proper selection of the discharge is important for a safe and economical design. Errors in the estimates can result in a structure that is undersized thus causing additional drainage problems, or is oversized thus costing more than necessary.

It should be noted that any hydrologic analysis is an approximation because the rainfall-runoff relationship is complex and does not lend itself to exact solutions. The designer is frequently faced with exercising independent judgment and experience in order to deal with variable conditions. This is why hydrology is often referred to as an art as well as a science.

Hydrologic analysis always precedes hydraulic design, regardless of the probable size or cost of a drainage structure. The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage facilities. The terms “runoff” and “flow” are used interchangeably. A runoff hydrograph is required for structures (such as detention facilities) that are designed to control the volume of runoff. The design peak flow rate is generally used to determine the size of the drainage structure.

The designer should refer to the publications listed under Section 4.5 (References) for detailed information pertaining to the hydrologic methods presented in this chapter.

4.2 DESIGN CONCEPTS

4.2.1 *FLOOD HAZARDS*

A hydrologic analysis is a prerequisite to identifying flood hazard areas. The designer shall attempt to determine those locations at which construction and maintenance will be unusually expensive or hazardous due to the presence of flood hazards.

4.2.2 *FLOOD HISTORY*

The hydrologic analysis shall consider the flood history of the area and the effects of historical floods on existing and proposed structures.

4.2.3 *HYDROLOGIC STUDIES*

The type and sources of information available for the hydrologic analysis will vary from site to site. It is the responsibility of the designer to determine the type of information that is needed.

The discharge from a published hydrologic study conducted by a federal agency (such as the U.S. Army Corps of Engineers and the Federal Emergency Management Agency) shall be researched and used if available.

4.2.4 *PRELIMINARY STUDIES*

Preliminary hydrologic studies and surveys, including environmental and ecological impacts, shall be undertaken to determine if hydrologic considerations can significantly influence the selection of a highway corridor. The magnitude and complexity of these studies shall be commensurate with the importance of the project.

Typical data to be included in such studies or surveys include: topographic maps, aerial photographs, stream flow records, historical high-water elevations, flood discharges, and locations of reservoirs and regulatory floodplain areas.

4.2.5 *COORDINATION*

Many levels of government may be involved in planning, designing and constructing highway and water resource projects, thus making interagency coordination desirable and often necessary. The designer shall coordinate with local, state (e.g., Conservation Agency, DHS&EM, DNR) and federal (e.g., USACE, USGS, NRCS) agencies to assist in the completion of accurate hydrologic analyses.

4.2.6 *DOCUMENTATION*

The results of all hydrologic analyses shall be fully documented using the drainage computation forms in this chapter and the report format in Chapter 3. It is often necessary to refer to the analyses long after the project construction has been completed.

4.2.7 EVALUATION OF RUNOFF FACTORS

For all hydrologic analyses, the following factors shall be evaluated:

- Drainage basin characteristics including: size, shape, slope, land use, geology, soil type, antecedent moisture condition, surface infiltration and storage (ponds, wetlands).
- Meteorological characteristics such as precipitation amount and type (rain, snow, hail, or combinations thereof), storm cell size and distribution characteristics, storm direction and time rate of precipitation (rainfall hyetograph).
- Stream channel characteristics including: cross sectional geometry and plan form, slope, hydraulic resistance, natural and artificial flow controls, channel modification, aggradation, degradation, ice and debris streambed and bank materials, and visual stream instability indicators.
- Floodplain characteristics including: land use, geology, soil type and development potential.

4.2.8 INFLUENCE OF KARST TOPOGRAPHY

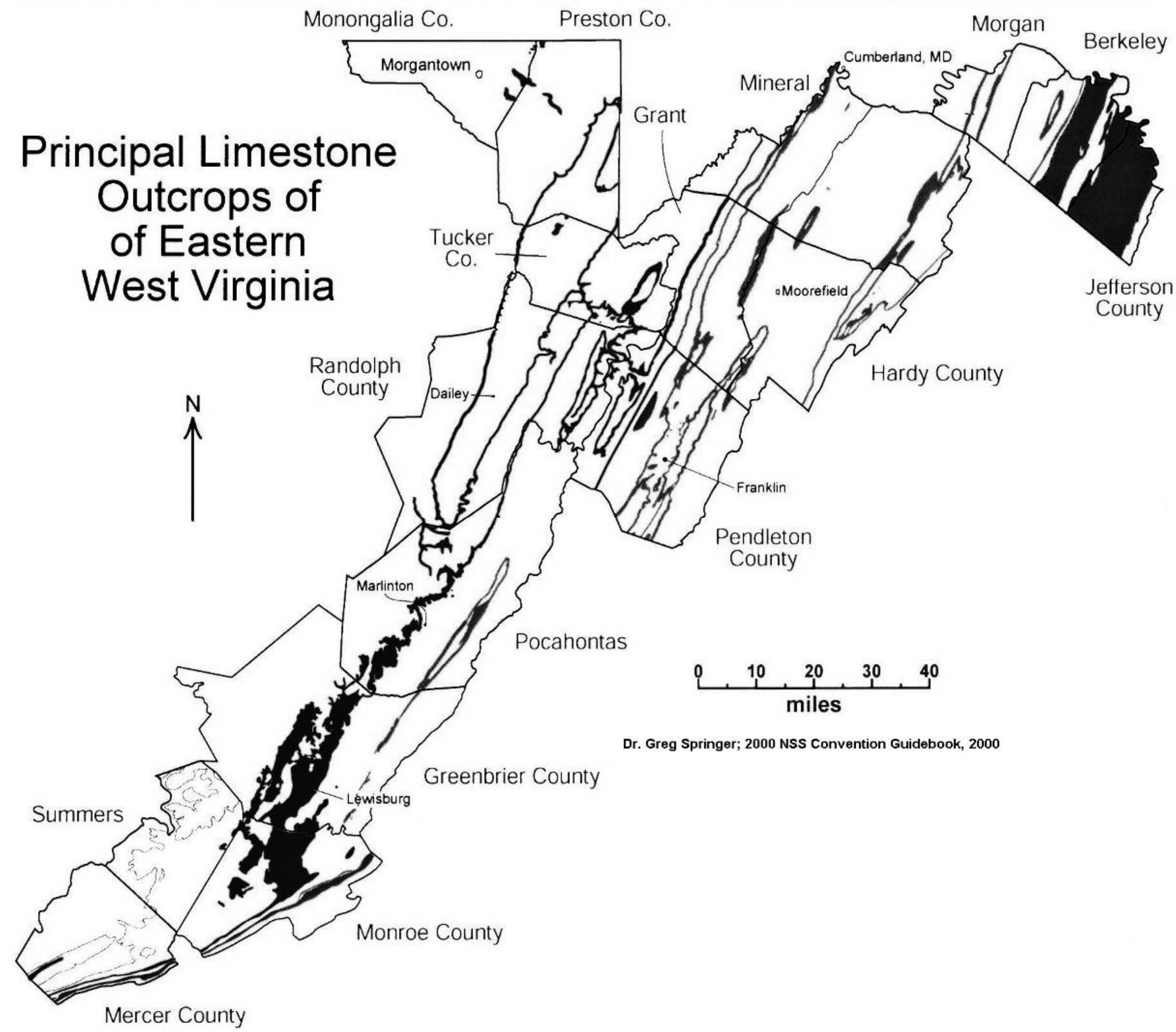
The word Karst is the German word for Kras, a region in Slovenia that sits on a limestone plateau. It was this location where the first scientific research of karst topography was made. Karst topography is a landscape of distinctive dissolution patterns marked by underground drainages. These are areas where the bedrock has a soluble layer of carbonate type rock such as limestone or dolomite. They are characterized by caves, crevices, cavities, fractured rock, disappearing streams, sinkholes, and ponds that appear to lack sufficient contributing drainage area. This topography can cause a poor estimation of runoff rates, over-designed conduits and under-designed stormwater management facilities. Standard hydrologic methods do not account for the infiltration losses in karst terrain, thus there needs to be some adjustment to account for the geologic influence on runoff.

4.2.8.1 IDENTIFYING KARST TOPOGRAPHY

Karst terrain can be found along the eastern boundary of the State in the counties of Mercer, Monroe, Summers, Greenbrier, Pocahontas, Randolph, Pendleton, Tucker, Grant, Preston, Monongalia, Mineral, Hardy, Hampshire, Morgan, Berkeley, and Jefferson (Map 4-1). Of these, Berkeley, Greenbrier, Jefferson and Monroe counties exhibit the most extreme karst features.

Map 4-1

Karst Areas for West Virginia



If a project is located in an area where the hydrology will be influenced by karst geology, the designer shall investigate the impact to the estimation of design discharge. Identification of karst terrain in a project area should be based on local geologic maps, soil maps, aerial photographs, and field observation of noticeable indicators. For some areas of the State, the USGS topographic maps show less than 50% of the karst features. These features could be detected with a simple site visit. Personnel within the Engineering Division's Geotechnical Section should always be consulted on the location of karst areas within the state.

Site evaluation for karst features should be performed in two phases, preliminary investigation done prior to a project design and a site specific investigation conducted once the project design is underway.

A preliminary investigation includes the review of maps, aerial photographs, and soil surveys along with a site visit. This visit should include someone with experience identifying karst features or local officials and residents familiar with the area. The purpose of this investigation is to identify areas of concern that may affect the project. The local office of the Natural Resource Conservation Service (NRCS) should be consulted about a sinkhole inventory in the area of a project. These inventories provide a list of documented dimensions and locations, however many were created in the early 1990's and they may not include recently developed features.

A site specific investigation includes collecting subsurface information for the areas of concern identified during the preliminary investigation. This information can be obtained through a boring contract using test pits, test borings, and geophysical instruments to evaluate the stability of soil and rock at locations of proposed construction activities. If unstable subsurface conditions are encountered, a decision can be made to proceed to remediate prior to construction or to modify the proposed layout to avoid problem areas. In critical locations, hydrological investigations (dye tracing) to define the subsurface flow paths of stormwater runoff entering the underground drainage network may be required.

4.2.8.2 SITE INVESTIGATION TOOLS

Geologic maps contain information on the physical characteristics and distribution of the bedrock and/or unconsolidated surficial deposits in an area. Geologic features such as the strike and dip of strata, joints, fractures, folds, and faults are usually depicted. The orientation of strata and geologic structures generally control the location and orientation of solution features in carbonate rock. The relationship between topography and the distribution of geologic units may reveal clues about the solubility of the specific rock units.

Aerial photographs are a simple and quick method of site reconnaissance. The inspection of photographs can reveal vegetation and moisture patterns that provide indirect evidence of the presence of cavernous bedrock. Piles of rock or small groups of brush or trees in otherwise open fields can indicate active sinkholes or rock pinnacles protruding above the ground surface. Circular and linear depressions associated with sinkholes and linear solution features and bedrock exposures are often visible when viewed in a stereo image.

During the site visit it is important to review drainage patterns, vegetation changes, depressions, and bedrock outcrops to look for evidence of ground subsidence. Sinkholes in subdued topography can often only be seen at close range. Disappearing streams are common in karst areas, and bedrock pinnacles that can be a problem in the subsurface will often protrude above the ground surface. A particularly simple and often overlooked part of the site visit is to interview the property owner. Often property owners can recount a history of problems with ground failure that may not be evident at the time of the site evaluation. The location of karst features should be noted on the site map for later reference. These can be compared to other information collected to assess the risk potential for karst-related problems.

Test pits or test probes are a simple and direct way to view the condition of soils that may reveal the potential for ground subsidence. An inspector should look for evidence of slumping soils, former topsoil horizons, cavities in soil or bedrock and fill from surface boulders or organic debris. The presence of organic soils at depth is an indicator of potentially active sinkholes sites. Leached or loose soils may also indicate areas of potential ground subsidence. These tools are not practicable for large areas and they can yield questionable data so their use should be site specific.

4.2.8.3 ACCOUNTING FOR KARST LOSS

The following procedure is recommended for estimating karst loss as part of a runoff estimate:

1. Define any areas within the apparent contributing drainage area where surface drainage has no means of escaping offsite other than through the karst strata. These areas can be assumed to contribute no surface runoff and can be subtracted from the contributing drainage area.
2. Areas on the mapping that show no defined streams or streams that disappear may also be subtracted from the contributing drainage area. These areas should be verified in the field.
3. Determine the remainder of the drainage area underlain by karst strata in percent.
4. Calculate the peak rate of runoff using the standard hydrologic methods presented in this manual and multiply that value by the karst loss coefficient (Table 4-1) based on the percent of area underlain by karst. The coefficient is intended to depict projected flow losses into bedrock.

Table 4-1

Karst Loss Coefficient

% Karst	Storm Return Period				
	2	10	25	50	100
100	0.33	0.43	0.44	0.46	0.50
90	0.35	0.46	0.48	0.50	0.56
80	0.38	0.51	0.53	0.56	0.62
70	0.47	0.58	0.60	0.62	0.68
60	0.55	0.66	0.67	0.70	0.74
50	0.64	0.73	0.74	0.76	0.80
40	0.73	0.80	0.81	0.82	0.85
30	0.82	0.86	0.87	0.87	0.89
20	0.91	0.92	0.92	0.92	0.93
10	1.00	0.98	0.98	0.98	0.97
0	1.00	1.00	1.00	1.00	1.00

Source: Adjusting Hydrology Models for Karst Geology, John Laughland P.E.

Other methods that can be utilized to account for karst loss include:

- Manipulating the runoff coefficient in the Rational method.
- Use of a Type I rainfall distribution within a Type II area, or manipulating the curve number values within the TR-55 method.

These parameters can be calibrated if some facts are known about the existing flow situation. That calibrated value can then be used to design a drainage structure. For example, say a project has a vertical alignment improvement by eliminating a dip in the roadway. There is a pipe crossing at this dip and area engineers and longtime local residents know that the roadway has never been overtopped by flow. They also know that there is only a small amount of water passing through the existing culvert during rainstorms. The TR-55 method leads to a proposed structure that is more than twice the size of the existing structure. The location of the project is in an area that is known for karst topography and a field visit yields some signs of karst strata below ground. A calibration of the curve number value can be done using the roadway profile elevation and its corresponding headwater for the existing structure. This new curve number value can then be used to calculate the flow for designing the proposed structure.

Manipulating these parameters has advantages and disadvantages in accurately representing the effects of karst topography. Calibrated values should be considered carefully and they must be defendable if questioned at a later time.

The USGS method should be avoided in a karst area if excessive runoff flows into a cavity (see Section 4.4.4.1). If the method is applied to a karst area there should be a field investigation to ensure the previous statement does not hold true. The equations are empirically based and the gage data used to derive the east region equations cover areas of karst topography where the data used to derive the south region equations cover less area influenced by karst topography (see Map 4-10).

4.2.8.4 KARST SURCHARGE

A rare event that may require consideration in areas of karst topography is the possibility of sinkhole surcharge. In this case, the opposite condition than what is expected occurs and water flows out of a depressed surface area during rainfall events. This occurs due to the connectivity of the underground conveyance network. These natural runoff detention areas may not be significant in the overall hydrology of an area but they could exert a significant impact by inundation during an extreme rainfall. The effect of this type of event may be considered similar to the effect of a check storm.

4.2.9 RETURN PERIOD OR PROBABILITY OF OCCURRENCE

The exceedance frequency is the relative number of times a flood of a given magnitude can be expected to occur on the average over a long period of time. It is usually expressed as a ratio or a percentage. By its definition, frequency is a probabilistic concept and is the probability that a flood of a given magnitude may be equaled or exceeded in a specified period of time, usually 1 year. Exceedance frequency is an important design parameter in that it identifies the level of risk during a specified time interval acceptable for the design of a highway structure.

The return period is a term commonly used in hydrology. It is the average time interval between the occurrence of storms or floods of a given magnitude. The exceedance probability (p) and the return period (T) are related by:

$$T = \frac{1}{p}$$

For example, a storm with an exceedance probability of 0.01 in any one year is referred to as the 100 year storm. The use of the term return period is sometimes discouraged because some people interpret it to mean that there will be exactly T years between occurrences of the event. Two 100 year floods can occur in successive years or they may occur 500 years apart. The return period is only the long term average number of years between occurrences.

4.2.10 RAINFALL FREQUENCY VERSUS STORM FREQUENCY

It is commonly assumed that there is a direct relationship between rainfall and storm frequency (i.e., the 10-year rainfall will produce the 10-year storm). The designer should recognize that this is not always true, depending on the antecedent soil moisture conditions and other hydrologic parameters.

Antecedent moisture conditions are the soil conditions at the beginning of the storm. These conditions affect the peak discharge only in the lower range of flood magnitudes (up to the 15-year event threshold). Antecedent moisture has a rapidly decreasing effect on runoff as floods become less probable.

4.2.11 RAINFALL FREQUENCY DATA

The National Weather Service's Hydrometeorological Design Studies Center (HDSC) completed a rainfall frequency update for the Ohio River Basin including West Virginia and its surrounding states in 2004. This update has improved rainfall frequency estimates and reflects additional rainfall data gathered since the last publication by the Weather Service. See Section 4.5 for reference to the (PFDS).

4.2.12 DISCHARGE DETERMINATION SITES

The most reliable method for calculating the magnitude and frequency of the expected peak discharge for a site with a given drainage area, is a long record of stream gage discharge data. On this basis, sites can be divided into two general categories:

- **Gaged sites –** The site is at or near a gaging station, and the stream flow record is of sufficient length to be used to provide estimates of peak discharges (see Section 4.4.1). The United States Geological Survey (USGS) operates and maintains stream flow gages of West Virginia streams and rivers.
- **Ungaged sites –** The site is not near a gaging station, and no stream flow record is available. This situation is very common for small watersheds.

The following sections will address hydrologic procedures that can be used for both categories of sites.

4.2.13 DISCHARGE DETERMINATION PROCEDURE

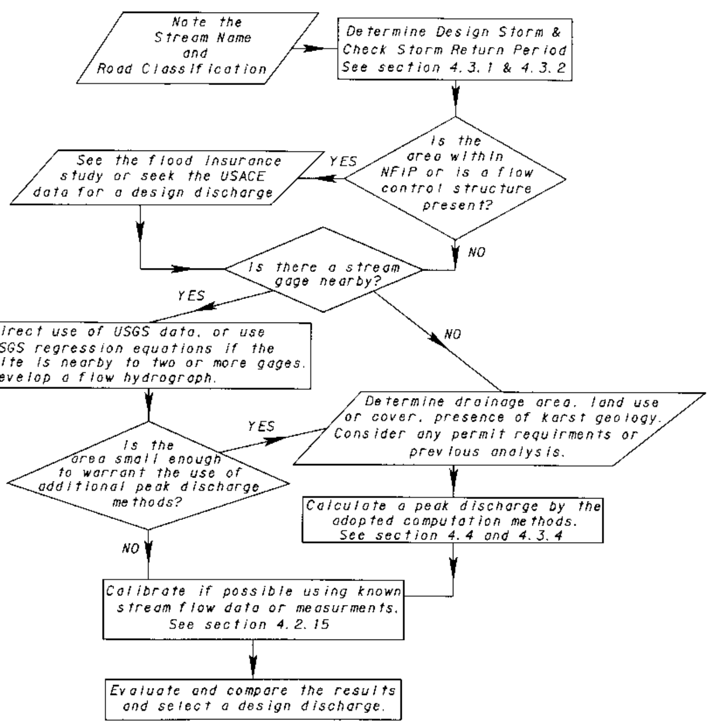
Unfortunately, stream flow gage records of sufficient length are seldom available and many small drainage areas have no records. In such cases, it is accepted practice to estimate peak discharge rates and hydrographs using statistical or empirical methods.

The decision on whether to use estimates of peak discharge or develop flow hydrographs should be made early in the design process. Peak discharge rates are generally adequate for most drainage facilities; however, if the design includes flow routing or storage considerations, a hydrograph is usually required. The development of hydrographs is typically more complex than estimating a peak discharge. Single event hydrographs are generally considered adequate for routing; however, continuous event hydrographs over long periods of time (typically years, covering periods of runoff as well as dry periods) may be required for special studies such as sediment transport investigations.

The designer shall use the procedure provided in the flowchart in Figure 4-1 to select the appropriate source of a peak discharge.

Figure 4-1

Hydrology Flowchart



Source: Formulated by DOH Drainage Unit

4.2.14 HYDROLOGIC METHODS

- **Analysis of Stream Gage Data -** If systematic stream gage data of sufficient length are available, they can be used to develop peak discharge estimates using statistical analysis such as the USGS' Bulletin 17B procedures. This has already been performed for rural unregulated streams in West Virginia. See the Water-Resources Investigation Report 00-4080 published by the USGS.

- **Rational Method -** This is the oldest and most widely used method of calculating peak runoff rates for urban and rural watersheds that are less than 200 acres.
- **Graphical Peak Discharge Method -** The Soil Conservation Service (SCS), now known as the NRCS (Natural Resources Conservation Service), has a widely used simplified graphical peak discharge method known as TR-55. This method may be used to determine rainfall runoff for a 24 hour duration with a return period of 1 year.
- **Regional Regression Equations -** Regional regression equations are easy to use and a commonly accepted method for estimating peak flows at ungaged sites or gaged sites with insufficient data. Regression equations are based on statistical analysis of stream gage data.
- **Hydrograph Methods -** The SCS Tabular Hydrograph Method (TR-55) is one of the most commonly used methods to produce a single event hydrograph. Other hydrograph methods include the Modified Rational Method and the Unit Hydrograph Method developed by the SCS (NRCS). Hydrograph methods can be complex and tedious to use, thus computer software is usually utilized. Their primary purpose is to determine a pre-land development and post-land development peak rate of runoff for use in the design of a stormwater management facility.
- **Watershed Model –** Computer programs, such as the U.S. Department of Agriculture's TR-20 and the U.S. Army Corps of Engineers' HEC-HMS, can be used to determine runoff rates from complex watersheds. Examples of some complex watersheds pertaining to DOH development are those that may contain a detention structure such as a stormwater pond or those that have a mixture of rural and urban conditions. If a watershed has residential and business development with storm sewers in one section and forested areas with open channels in another section, it would be difficult to account for the variance in the behavior of the runoff with any other hydrologic method.

4.2.15 CALIBRATION

The accuracy of the hydrologic estimates will have a major effect on the design of drainage or flood control facilities. Calibration is the process of varying the parameters or coefficients of a hydrologic method so that it will estimate peak discharges and hydrographs that are consistent with local rainfall and stream flow measurements. Although it might be argued that one hydrologic procedure is more accurate than the other, all the methods discussed in this chapter, if calibrated, can produce acceptable results consistent with observed or measured events. Therefore, calibrating the method to local conditions will result in more accurate and consistent estimates of peak flows and hydrographs.

4.3 DESIGN CRITERIA

4.3.1 DESIGN DISCHARGE OR STORM

The proper hydraulic design of a drainage structure begins with the determination of a design flow or discharge. It is common practice to refer to this discharge by the rainfall or storm that generates it. It is not normally economical to design a drainage structure to carry the maximum possible peak flow. Therefore it is desirable to ascertain how much below the maximum we can design for without seriously impairing the operation of the highway. Factors that control the determination of the design storm result from the possibility of exceeding the hydraulic capacity of a structure. These factors are: the extent and cost of repairing damage to the highway, the hazard and inconvenience to the user of the highway, and the importance of the highway.

The extent of damage and the hazard and inconvenience to the highway user are factors that are difficult to predict and measure. They vary with the amount by which the capacity is exceeded and the frequency with which such events occur. Since excessive rainfall and high runoff occur at random intervals it is desirable to consider a peak flow on a probability of occurrence (frequency) basis.

Drainage facilities shall be designed to accommodate the discharge for the minimum specified design storm criteria. Table 4-2 presents the minimum criteria to protect roadways from flooding or damage based on the frequency, return period or the annual probability of occurrence. It should be emphasized that these values only apply to the minimum level of protection afforded to the roadway.

Table 4-2

Design Storm Criteria

Facility Type	Description	Return Period (Probability of Occurrence)
Storm Drainage	Inlet Design (Pavement Runoff)	10-year (10%)
	Pipe Outlet System	10-year (10%)
Ditches	Roadside, Secondary, and Median	10-year (10%)
	Protective Linings	10-year (10%)
Channels, Culverts and Bridges	Divided Highways Principal Arterial Highway with high ADT*	50-year (2%)
	Highways Over 400 ADT*	25-year (4%)
	Highways Under 400 ADT*	10-year (10%)

*ADT: Average Daily Traffic Volume.

Exceptions to these frequencies may occur when stream records show higher discharges and/or when potential property damage justify a higher level of protection, as approved by the Director of Engineering. Consideration shall also be given to the return period that was used to design other structures along the highway corridor.

In certain situations, it may be necessary to size a drainage replacement structure to carry a smaller discharge than that specified by the minimum design criteria. One such situation would be in an area of urban development already subject to flooding downstream. Alteration of the pre-construction site hydrology needs to be minimized to control runoff; therefore a smaller structure could induce some flow detention.

The selected design storm frequency shall not result in a condition that is worse than the existing flow condition. For example: replacement structures where approach work is limited by physical or monetary constraints shall convey the design discharge such that the backwater elevation is not higher than that created by the existing structure. In many such cases, the replacement structure will be overtopped by the design discharge.

4.3.2 CHECK STORM

Proposed structures designed to accommodate a storm of a particular return period or probability of occurrence shall be reviewed using a check storm of a larger magnitude or higher return period (i.e., lower frequency). The check storm shall be one that represents the 100-year storm or the overtopping of the roadway, whichever is less. This check storm shall be used to evaluate the proposed condition against the water surface elevation of the existing condition. The goal of the check storm is to avoid increasing the water surface elevation such that it becomes destructive to property upstream and downstream of the project area.

This water surface elevation:

- Shall comply with National Flood Insurance Program regulations.
- Shall not be detrimental to nearby property.
- Shall not be detrimental to the roadway and appurtenances.

After examination of the water surface elevation, the impacts or absence of impacts shall be noted in the hydraulic report. Detrimental impacts to nearby property may require a more detailed design analysis and right-of-way taking or easement acquisition.

4.3.3 DESIGN DISCHARGE SOURCE PRIORITY

The recommended priority for obtaining a design discharge for a new or replacement structure is:

1. The Flood Insurance Study (FIS) for the local community, if available. A community is the county, town, or city, whichever has political jurisdiction over the floodplain site. Design discharge differing by more than 10% from the published FIS values will need justification.
2. U.S. Army Corps of Engineers (USACE) or Natural Resources Conservation Service (NRCS) modified frequency discharges taking into account stream regulation by reservoirs and detention dams upstream (e.g., Wheeling Creek by NRCS and the Guyandotte River by USACE)
3. Local gaging station records and frequency, taking into account the length of unregulated systematic record.
4. Hydrologic runoff estimation methods provided in this manual.

If it is determined that WVDOH activities will cause changes beyond regulatory limits in the published FIS floodplain data, 100-year flood profile, or floodway widths, the designer shall notify the Division for guidance. A Conditional Letter of Map Revision (CLOMR) or Letter of Map Revision (LOMR) request to FEMA may be required.

4.3.4 DESIGN DISCHARGE SELECTION

The three hydrologic runoff estimation methods presented in this manual provide a guide to making a responsible engineering judgment regarding the discharge from a particular watershed. The suggested acreage limits to which the methods are applicable are not definite, but within a professionally accepted range that may be exceeded by a minor amount deemed reasonable by the designer (see Table 4-3). It is not the intent of this manual to serve as a comprehensive text for the presented methods. Their application to a watershed and their limitations in estimating the runoff are discussed in this chapter.

Table 4-3

Discharge Method Range

Rational Method
0 to 200 acres or 0 to 0.31 mi^2
TR-55 Method
5 to 16,000 acres or 0.01 to 25 mi^2
USGS Method
(by region) 64 to 5,357,440 acres or 0.1 to 8,371 mi^2

There is no single method for determining a peak discharge that is applicable to all watersheds within the state of West Virginia as each method was formulated from disparate concepts. This means the conditions upon which they were established and the processes of measurement or estimation due to those conditions are different. When multiple methods are warranted for estimation of the runoff, the selection of the design discharge shall be based on how the size and complexity of the watershed relate to the methods. The designer should become familiar with the application and the limitations of the method in order to consider its results properly. To select this discharge, the results from the various methods shall be compared to each other and to the site conditions. The results shall <u>never be averaged nor shall the highest value be selected</u> in order to be conservative.

The intention of comparing the results is to look at the relative differences in the calculated discharge. A wide variation (or scatter) in the results means the designer should examine the process of each method and re-evaluate the variables for their range of limitation, reasonableness, and representative accuracy.

The selected discharge should result from one particular method that is supported by this re-evaluation and should not be “arbitrary”. If one hydrologic method is initially identified as the preferred method, it is still recommended that the results of this method be compared to at least one other appropriate method as a means of validation. The final discharge selected shall be reproducible so that it can be verified or legally defended at a later date.

For example: Say you have a 247 acre watershed (0.39 square miles) in Raleigh County within the Beckley city limits. This watershed has three main cover types:

woods in good condition, paved streets or rooftops, and lawns in good condition, fair condition and poor condition. The longest flow path from the hydrologically most distant point is a complex route from steep rocky slopes to trapezoidal shaped open channel to storm sewer pipes and back to open channel. The slope of the watershed along this path is 10.7% for the first 30% of the flow distance and 2.2% for the last 70% of the flow distance. The travel time along this flow path (time of concentration) was physically measured by monitoring the flow of dye that was placed at the hydrologically most distant point. This time was measured to be 75 minutes. There are primarily two underlying soil types within the watershed. The first type is named Gilpin which has a moderate to high available moisture capacity with moderate permeability and a low amount of organic material. The second type is named Dekalb which has low to moderate available moisture capacity and rapid permeability. Precipitation frequency estimates for this watershed were obtained from NOAA Atlas 14. The applicable hydrologic runoff estimation methods and their results in cubic feet per second are as follows:

Return Period in years	**25**	**50**	**100**
Rational Method	207	229	253
TR-55 Method	218	265	316
USGS Method	172	207	245

Selecting a design discharge for a return period shall be based on how each method applies to the watershed and the limitations of each method within that application. In this example the quality of the input data is very good, especially with a physically measured value for time of concentration. The variation in discharge among the methods is pretty small with the USGS method as the outlier. Since the USGS method is derived from data gathered from river gages, its application for this small stream in a semi-urban area pushes the methods' applicability. However the values obtained from the USGS method compare well with the values obtained from the Rational and TR-55 methods, thus validating the results. Removing the USGS method from consideration leaves discharge values from the other two that are very close.

The effects of ground cover have been accounted for equally within both of the remaining methods under consideration. In this example similar cover types were broken up into sub-areas to derive a weighted curve number with the TR-55 method and a weighted runoff coefficient with the Rational Method. However, the size of the drainage area is 47 acres over the acreage limit for the Rational Method (200 acres). Another limitation of the Rational Method is that it assumes that the rate of runoff resulting from a rainfall intensity is a maximum when that intensity lasts as long or longer than the time of concentration (see Section 4.4.2.2). This means the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed. For areas that have a large time of concentration it is unlikely that a storm with a rainfall intensity for that length of time will occur. Therefore, with a measured time of concentration of 75 minutes and the watershed area being 47 acres over the acreage limit the Rational method should also be removed from consideration. Therefore the TR-55 method becomes the best method applicable to this example watershed.

4.4 DESIGN DISCHARGE ESTIMATION METHODS

This section provides an overview of the three WVDOH adopted peak discharge computation methods. Drainage Computation Form 4-1and Form 4-2 should be used to record the computations. These forms will provide consistency in the data presentation.

Form 4-1

Peak Discharge Computation Form

PEAK DISCHARGE COMPUTATION FORM DR 4-1

CALCULATED BY: ________ **DATE:** ________

CHECKED BY: ________ **DATE:** ________

PROJECT NAME: ________

STATE PROJECT NUMBER: ________

AREA NUMBER: ATTACH WATERSHED MAP

LOCATION DESCRIPTION:

DRAINAGE AREA = ________ **ACRES** ________ **MI^2**

STATION ________ **TO** ________

DESIGN RETURN PERIOD: ________ **YEARS**

RATIONAL METHOD
1 acre - 200 acres

TIME OF CONCENTRATION

OVERLAND FLOW

SHEET FLOW

$T_{t\,sh}$ = ________ Min.

SHALLOW CONCENTRATED FLOW

$T_{t\,sc}$ = ________ Min.

CHANNEL FLOW

$T_{t\,ch}$ = ________ Min.

$T_c = T_{tsh} + T_{tsc} + T_{tch}$ = ________ Min.

Method: Kirpich (rural areas) ☐

Segments (urban areas) ☐

IDF REGION ________

Rainfall Intensity i = ________ in/hr

C	A	CA
____	____	____
____	____	____
____	____	____
Total	____	____

Weighted Coefficient "C" = ________

$C = \sum (CA) / \sum A$

Q = ________ cfs

TR - 55
5 acres - 16,000 acres

INFO FROM WORKSHEET 4-1

CN = ________

24 hr P = ________ in.

Runoff Depth Q = ________ in.

INFO FROM WORKSHEET 4-2

T_c = ________ hr.

INITIAL ABSTRACTION (Table 4-13)

I_a = ________ in.

I_a / P = ________

UNIT PEAK DISCHARGE q u

USE Tc AND I a / P WITH CHART 4-8

= ________ cfs / mi^2 / in

POND AND SWAMP AREAS

Percent of watershed

= ________ %

(Table 4-8) Factor F_p = ________

PEAK DISCHARGE

$q_p = q_u (A \text{ in mi}^2)\, Q\, F_p$

q_p = ________ cfs

USGS METHOD
10 square miles - 1,619 square miles

REGION: FROM MAP 4-9

EASTERN PANHANDLE ☐

CENTRAL MOUNTAINS ☐

WESTERN PLATEAUS ☐

EQUATION:

FROM TABLE 4-15

Eqn: ________

PRELIMINARY DESIGN

DRAINAGE AREA 5 TO 10 MI ^{2}

ADD THE

STANDARD PREDICITION ERROR

= ________ %

Q = ________ cfs

SELECTED DESIGN DISCHARGE

Q = ________ cfs

REASON FOR SELECTION (BASED ON COMPARISON) SEE SECTION 4.3.4 ________

Source: Created by the WVDOH Hydraulic and Drainage Unit

Form 4-2

Peak Discharge Computation Form

PEAK DISCHARGE COMPUTATION FORM DR 4-2

CALCULATED BY: ________ **DATE:** ________ **CHECKED BY:** ________ **DATE:** ________	**PROJECT NAME:** ________ **STATE PROJECT NUMBER:** ________
AREA NUMBER: ATTACH WATERSHED MAP **LOCATION DESCRIPTION:** **DRAINAGE AREA =**________ **ACRES** ________ **MI^2**	**STATION** ________ **TO** ________ **DESIGN RETURN PERIOD:** ________ **YEARS**

FLOOD INSURANCE STUDY DATA	**UNITED STATES ARMY CORP OF ENGINNERS DATA**	**USGS METHOD** 10 square miles - 1.619 square miles
FIS DATE: ________ COMMUNITY NAME AND NUMBER: ________ ________ LOCATION: ________ ________ METHOD: RAINFALL RUNOFF MODEL ☐ REGIONAL REGRESSION EQUATIONS ☐ ANALYSIS OF GAGE RECORDS ☐ OTHER (PROVIDE DESCRIPTION) ☐ DESCRIPTION: ________ ________ ATTACH COPY OF SUMMARY OF DISCHARGES TABLE FROM FIS	FIS DATE: ________ COMMUNITY NAME AND NUMBER: ________ ________ LOCATION: ________ ________ METHOD: RAINFALL RUNOFF MODEL ☐ REG. REGRESSION EQUATIONS ☐ ANALYSIS OF GAGE RECORDS ☐ OTHER (PROVIDE DESCRIPTION) ☐ DESCRIPTION: ________ ________ ATTACH COPY OF DATA OF FROM USACE	REGION: FROM MAP 4-9 EASTERN PANHANDLE ☐ CENTRAL MOUNTAINS ☐ WESTERN PLATEAUS ☐ EQUATION: FROM TABLE 4-15 Eqn:________ PRELIMINARY DESIGN DRAINAGE AREA 5 TO 10 MI ^{2} ADD THE STANDARD PREDICITION ERROR = ________ %
Q = ________cfs	Q = ________cfs	Q = ________cfs

SELECTED DESIGN DISCHARGE **Q = ________ cfs**	REASON FOR SELECTION (BASED ON COMPARISON) SEE SECTION 4.3.4 ________ ________ ________

Source: Created by the WVDOH Hydraulic and Drainage Unit

4.4.1 ANALYSIS OF STREAM GAGE DATA

Many stream gage stations exist throughout West Virginia, whose data can be obtained and used for hydrologic studies. If a project is located near one of these gages and the gage record is of sufficient length in time, a frequency analysis can be made. Reliable 100-year discharge estimates require at least 25 years of continuous or synthesized stream flow record. At least 10 years of record is needed for the 10-year discharge estimate.

The most important aspect of gage station records is a series of annual peak discharges. Flood frequency curves are derived from a frequency analysis of the annual peak discharge data. Such curves can then be used in several different ways.

- If the highway facility site is at or very near the gaging site and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be used directly.
- If the facility site is nearby or representative of a watershed with similar hydrologic characteristics, transposition of frequency discharges is possible.
- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived. Regional regression relations have been developed by the USGS and the designer will not be involved in their development.

Two basic methods of estimating flood frequency curves from stream gage data are available. The Gumbel Method is a graphical procedure, and the Log-Pearson Type III Frequency Distribution Method is a statistical method. The Gumbel Method is not preferred because it is subjective, and each designer may derive different estimates from the same dataset. The Log Pearson III Method is the most reliable and widely accepted statistical method to determine flood frequency using annual maximum stream flow series data.

Bulletin 17B (see Section 4.5, References) describes the statistical procedure using the Log Pearson III Method for computing flood flow frequency curves, where a systematic stream gage record of at least a 10-year duration are available for unregulated (i.e., without dams) streams and rivers. The designer shall acquire approval from the Director of Engineering before using this method.

4.4.2 RATIONAL METHOD

4.4.2.1 APPLICATION

The Rational Method is recommended for estimating the design peak discharge for areas as large as 200 acres. While the Rational Method is relatively straightforward to apply, its concepts are quite sophisticated. Considerable engineering judgment is required to reflect representative hydrologic characteristics, site conditions, and a reasonable time of concentration (T_c). Its widespread use in the engineering community represents its acceptance as the standard of care in engineering design. Form 4-1 should be used to record the computations with the Rational Method.

Some important factors to consider when applying the Rational Method are:

- Obtain a good topographic map and define the boundaries of the drainage area. A field inspection should also be made to verify the drainage divides and to check if the natural drainage divides have been altered.
- Restrictions to natural flow such as highway crossings and dams that exist within the drainage area should be investigated to see how they affect the design flows.
- The charts and tables in this chapter are not intended to replace reasonable and prudent engineering judgment in the design process.

Engineering applications of the Rational Method primarily pertain to ditch design and urban drainage design. This is due to the characteristics and smaller size of the drainage areas involved in the design process. The Rational Method may be applied to culvert design for drainage areas less than the 200 acre limit.

4.4.2.2 LIMITATIONS

The following assumptions limit the use and effectiveness of the Rational Method:

- The intensity of a rainfall event (depth of water / duration of rainfall) is assumed to be uniform over the entire drainage area.
- The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long as or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.

  This assumption limits the size of the drainage basin that can be evaluated by the Rational Method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows.
- The frequency of the peak discharge is the same as that of the rainfall intensity for the given time of concentration.

  A peak discharge depends on rainfall frequency, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and principally impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics of the watershed will control. For drainage areas with few impervious surfaces (i.e., less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

- The fraction of rainfall that becomes runoff is independent of rainfall intensity or volume.

  This assumption is reasonable for impervious surfaces such as streets, rooftops and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. Thus, the application of the Rational Method involves the selection of a runoff coefficient that is appropriate for the storm, soil and land use conditions.

- The peak rate of runoff is sufficient information for the design.

  Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. With only the peak rate of runoff, the Rational Method severely limits the evaluation of design alternatives available in urban, and in some instances, rural drainage design.

4.4.2.3 *EQUATION*

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and the mean rainfall intensity. This estimate of the peak rate is for a duration equal to the time of concentration (i.e., the time required for water to flow from the most hydraulically remote point of the drainage basin to the location being analyzed). The Rational Method Formula is expressed as follows:

$$Q = C\ i\ A$$

Where: Q = Maximum rate of runoff in cubic feet per second (cfs)

C = Runoff coefficient representing a ratio of runoff to rainfall (dimensionless), see Table 4-4

i = Average rainfall intensity for a duration equal to the time of concentration (T_C) for a selected return period, inches per hour (in/hr)

A = Drainage area contributing to the design location, acres (ac)

4.4.2.4 *RUNOFF COEFFICIENT*

Good engineering judgment must be used when selecting a Runoff Coefficient (C) value because it represents the integrated effects of many drainage basin parameters. This coefficient must account for all of the factors affecting the relationship of peak flow to average rainfall intensity. The design and peak flows should be checked against observed data when it is available.

A range of C-values is typically offered to account for slope, condition of cover, antecedent moisture condition and other factors that may influence the amount of runoff. Table 4-4 provides recommended runoff coefficients for both rural and urban land use conditions.

As the slope of the drainage area increases, the velocity of overland and channel flow will increase thus allowing less opportunity for water to infiltrate the ground surface. This means more of the rainfall will become runoff, thus a representative C-value should increase with the slope of the drainage area. The lowest range of C-values should be used for flat areas where the majority of the sub-basin slopes are less than 2 percent. The middle range of C-values should be used for moderately sloped areas where the majority of the sub-basin slopes range from 2 to 10 percent. The highest range of C-values should be used for steep areas with grades greater than 10 percent, or drainage basins with primarily clay soils. Soil properties influence the relationship between rainfall and runoff since different types have different rates of infiltration.

Table 4-4

Recommended Runoff Coefficient (C) Values

Description of Area	**Runoff Coefficient (C)**		
	Flat areas Slope 0% to 2%	**Moderate areas** Slope 2% to 10%	**Steep areas** Slope Over 10%
Pavement, Roof surfaces, etc.	0.80	0.90	0.95
Earth Shoulder	0.55	0.60	0.70
Gravel or Stone Shoulders	0.45	0.50	0.60
Grass Shoulders	0.30	0.35	0.40
Side Slopes–Earth	0.50	0.60	0.70
Side Slopes–Turf	0.40	0.50	0.65
Median Strips–Turf	0.30	0.35	0.40
Dense Residential Areas	0.60	0.65	0.80
Suburban Areas with Small Yards	0.40	0.50	0.60
Cultivated Land			
Clay and Loam	0.35	0.50	0.60
Sand and Gravel	0.25	0.30	0.35
Woods, Parks, Meadows, and Pasture Land	0.20	0.25	0.35

Source: WVDOH Drainage Manual, 1984

It is often desirable to develop a weighted or composite runoff coefficient based on the percentage of different surface types within the drainage area (see procedure on Form 4-1). The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for the entire area. C-values for residential areas do not account for the impervious area associated with roadways.

4.4.2.5 TIME OF CONCENTRATION

The time of concentration (T_c) is the time required for water to flow from the hydraulically most remote point of the drainage area to the point of interest. With the Rational Method, the duration of a rainfall event is set equal to the time of concentration and it is used to estimate the average rainfall intensity (i) from the intensity-duration-frequency curves (IDF) for a selected return period.

There are several methods for estimating the time of concentration. Most methods define T_c as the sum of the travel times within various consecutive flow segments in the drainage basin. These segments generally consist of the overland flow segment and the channel or pipe flow segment. The overland flow segment is subdivided into the sheet flow and shallow concentrated flow segments.

In an urbanized area, separate methods shall be used to obtain the respective travel times that make up the total time of concentration. The flow path in an urban location primarily consists of overland flow. This flow path is common for designing inlet spacing as part of an urban drainage design and for the design of roadside ditches. Both usually contain the sheet and/or the shallow concentrated flow segments and ditch design usually includes the channel or pipe flow segment. The following methods are also included as part of the TR-55 discharge estimation method which is discussed in Section 4.4.3.

Sheet flow - It is a shallow mass of runoff on a plane surface with the depth staying uniform across the sloping surface. Typically, flow depths will not exceed two inches. Flow travel rates are commonly estimated using the original form of the kinematic wave equation which is a derivative of Manning's equation. It was developed by two scientists named Henderson and Wooding, and they derived it based on the assumption of constant rainfall excess intensity (excess meaning rainfall which did not infiltrate the ground and resulted in runoff) resulting in turbulent overland flow. Their model was based on very small urban watersheds in which the overland flow dominates. The flow lengths in the model ranged from 50 to 100 feet. Beyond a distance of 100 feet sheet flow tends to become concentrated; therefore 100 feet shall be the maximum length for the sheet flow component. The equation is as follows:

$$T_{t\ \text{sheet flow}} = \frac{0.93}{i^{\,0.4}}\left(\frac{n\,L}{\sqrt{S}}\right)^{0.6}$$

Where: T_t = sheet flow travel time in minutes

n = roughness coefficient

L = flow length in feet

i = rainfall intensity for the storm return period in inches / hour

S = slope of the surface in ft / ft

In this paragraph the terms travel time and time of concentration are used interchangeably. As mentioned earlier, with the rational method the storm duration equals the time of concentration (in this case the travel time), thus the intensity is determined from the intensity-duration-frequency (IDF) curve using the time of concentration (see Section 4.4.2.7). However intensity depends on the time of concentration (travel time) and the time of concentration is not initially known, therefore the computation is an iterative process. An initial estimate of the time of concentration is assumed and used to obtain the intensity from the IDF curve for the latitude and longitude, or the rainfall intensity zone for the project location. The time of concentration (travel time) is then calculated from the kinematic wave equation and checked against the initial estimate. If they are not the same the process is repeated until the calculated value and the initial estimate are the same. These values will converge quickly so only a couple of calculations are usually necessary.

Table 4-5

Roughness Coefficient for Sheet Flow

Manning's Roughness Coefficient for Surfaces		
Concrete Surface (smooth)	0.012	
Concrete (rough), Lined Channel or Overlaid Surface	0.013	
Asphalt Surface (smooth)	0.011	
Asphalt Surface (rough), Lined Channel or Overlaid Surface	0.016	
Gravel Surface	0.024	
	May Use Values Between Those Provided Accompany with a Description	
Broken soil, Clean, No Residual Vegetation		0.05
Broken Soil, with Vegetation Cover ≤ 20%		0.06
Sparse Coarse Grass, Shrubby Vegetation, Bare Earth Present, Drier Less Fertile Soil		0.13
Short Grass, No Trees, Drier Less Fertile Soil		0.15
Broken Soil, Vegetation Cover > 20%, Fertile Soil		0.17
Medium Height Grass, Coarse, Possibly Trees, Fertile Soil		0.24
Forest Area, Light Density of Coarse Grass, Some Shrubs, Bushes, Small Trees		0.40
Dense Lawn Grass with Fertile Soil, Typical for a Golf Course		0.41
Forest Area, Dense Amount of Coarse Grass, Shrubs, Bushes, Small Trees		0.80

Source: Hydraulic Design Series 2, Highway Hydrology, October 2002

When selecting a roughness coefficient for a forested surface consider the cover or underbrush to a height of about 1.5 inches as this is the only part of the plant cover that will obstruct sheet flow.

To avoid the necessity to solve for the time of concentration (travel time) iteratively, the NRCS TR-55 method uses a variation of the original form of the kinematic wave equation. Details on this form are discussed in the TR-55 section of this chapter. Both forms of the equation yield similar values for overland sheet flow equal to or less than 100 feet. The original version lends itself to spreadsheet use as the TR-55 version is easier to use when hand calculations are utilized.

Shallow Concentrated flow – Beyond 100 feet, flow tends to concentrate in increasing proportions. In the case of inlet spacing design, the gutter flow is considered shallow and concentrated. In the case of roadside ditch design, flow over the shoulder or flow at the beginning of the ditch is considered shallow and concentrated. It is important to note that shallow concentrated flow may not always be perpendicular to the contour of the land. This can occur when benches or upland ditches are used along a cut slope to control erosion. The travel time for this flow segment is determined by a velocity method. The velocity for this type of flow can be estimated using an empirical relationship between velocity and the surface slope.

$$V = k\sqrt{S}$$

Where: V = velocity in feet / second

k = surface cover coefficient

S = slope of the surface in ft / ft

Table 4-6

Surface Cover Coefficients

Cover Type	**k**
Forested Surface with dense underbrush Lawn Grass Surface, such as Bermuda grass	2.5
Forested Surface with light underbrush Rough / Uneven Bare Soil Surface with sparse vegetation	5.0
Dense Grass on an even soil surface	7.0
Short Grass Surface	9.0
Smooth Bare Soil Surface before vegetation establishment	10.1
Vegetated Channel	15.1
Gravel or Soil and Gravel Surface	16.2
Asphalt or Concrete Paved Surface	20.4

Source: Hydraulic Design Series 2, Highway Hydrology, October 2002

Any shallow concentrated flow occurring within a small rock lined channel or swale with stones larger than gravel size should be considered as channel flow. An example of this situation would be “dump rock” at the head of a roadside ditch where the drainage divide is more than 100 feet away and the shape of the ditch channel is not defined.

Travel time is then calculated by dividing the length of flow by the velocity.

$$T_{t\ \text{shallow concentrated flow}} = \frac{L}{V * 60}$$

Where: T_t = shallow concentrated flow travel time in minutes

L = flow length in feet

V = velocity in feet / second

Channel or Pipe flow – As shallow concentrated flow continues it becomes deeper and wider and changes into channel flow. In many cases, the transition may be assumed to occur where channels are visible on aerial photographs or where blue lines (indicating streams) appear on USGS quadrangle maps. Channel lengths may be measured directly from the map or scale photograph. Pipe lengths should be taken from drawings for existing systems and design plans for future systems. The travel time for this flow segment is also determined by a velocity method. Manning's equation is used to estimate the average flow velocity in the channel or pipe and the travel time is calculated by dividing the channel or pipe length by the average flow velocity. In order to obtain the flow velocity a flow area must be known. The bankfull flow area shall be used to estimate this velocity.

$$V = \frac{1.49}{n} R^{\frac{2}{3}} \sqrt{S}$$

Where: V = velocity in feet / second

n = Manning's Roughness Coefficient

R = hydraulic radius in feet (flow area / wetted perimeter)

S = slope of the channel in ft / ft

$$T_{t\ \text{channel or pipe flow}} = \frac{L}{V * 60}$$

Where: T_t = channel or pipe flow travel time in minutes

L = flow length in feet

V = velocity in feet / second

Table 4-7

Manning's Roughness Coefficients (n) for Channel Flow

Surface Description	Recommended		Range
Existing Vegetative Lining			
Nearly bare, light grass	0.030		0.030 – 0.035
Grass, weeds, and light brush	0.040		0.030 – 0.050
Thick grass, thick brush, small trees	0.075		0.050 – 0.100
Planned DOH Vegetative Lining			
Type B Seed Mixture (mowed)	0.042		0.036 – 0.050
Type C-1 Seed Mixture (mowed)	0.036		0.030 – 0.040
Type C-2 Seed Mixture (mowed)	0.027		0.022 – 0.033
Type B Seed Mixture (unmowed)	0.090		0.050 – 0.140
Type C-1 Seed Mixture (unmowed)	0.080		0.050 – 0.120
Type C-2 Seed Mixture (unmowed)	0.030		0.025 – 0.040
Non Vegetative Lining	**Based on Depth of Flow**		
	0 - 0.5'	0.5 - 2.0'	> 2.0'
Concrete Lined Ditch or channel	0.015	0.013	0.013
Grouted Rock Lined Ditch or channel	0.040	0.030	0.028
Bare Soil with little or no vegetation	0.023	0.020	0.020
Bare Rock or Rock Cut Ditch	0.045	0.035	0.025
Rock Lined Ditch or channel D_{50} = 4 inches	0.090	0.058	0.035
Rock Lined Ditch or channel D_{50} = 6 inches	0.104	0.069	0.035
Rock Lined Ditch or channel D_{50} = 12 inches	-	0.078	0.040

Source: Formulated by DOH Drainage Unit

In a rural area where all three flow segments are known to exist and the channel or pipe flow segment is dominant, the Kirpich method may be used to determine the total time of concentration. Z. P. Kirpich developed this method in 1940 from SCS data taken from seven small rural watersheds near Jackson, Tennessee. These watersheds were located in agricultural areas ranging from 1.2 to 112 acres in size. They had well defined channels with steep slopes (3-10%), well drained soils, and timber cover ranging from 0 to 56 percent. These parameters work well for rural basins equal to or less than 200 acres in West Virginia. This equation has been used by the Division since

1963 and it is also used by many other states DOT in the U.S. Chart 4-1 presents the Kirpich equation and it may be used to determine the total time of concentration for all three flow segments in rural basins 200 acres or less in size. The designer should record the value from the Kirpich Method in the blank assigned to the total time of concentration, T_c on Form 4-1. The adjustment factors for overland flow on asphalt and grass surfaces and for flow in concrete channels have been removed. The kinematic wave formula and the velocity methods are more appropriate procedures for determining a time of concentration in those applications. The minimum allowable T_c shall be 5 minutes except in the case of a storm sewer system hydraulic design. Since the travel time is summed as a design progresses through a system, applying the minimum of 5 minutes at the beginning artificially lowers the rainfall intensity when determining the overall time of concentration at the outlet.

4.4.2.6 ERRORS IN ESTIMATING T_c

There are three common errors to avoid when calculating T_c. First, runoff from a highly impervious portion of the drainage area may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments should be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different times of concentration to determine the design flow.

Second, the overland flow path is not always perpendicular to the contours shown on the available mapping. Often the land will be graded so that swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. Therefore, field verification is a must. Care should be exercised in selecting sheet flow paths in excess of 100 feet. Beyond this distance sheet flow tends to become shallow concentrated flow. Overland flow paths (i.e., sheet flow + shallow concentrated flow) should generally not be greater than 200 feet in urban areas and 400 feet in rural areas.

Third, the flow path may change from open channel to pipe flow with changes in direction and size. This is common in old existing storm sewers in urban areas. Care should be taken when estimating T_c in these areas because an underestimation will cause the peak discharge to be very high. If details of the flow path cannot be determined it may be necessary to dye trace and time the flow through the system for an actual field measurement of the time of concentration.

4.4.2.7 RAINFALL INTENSITY

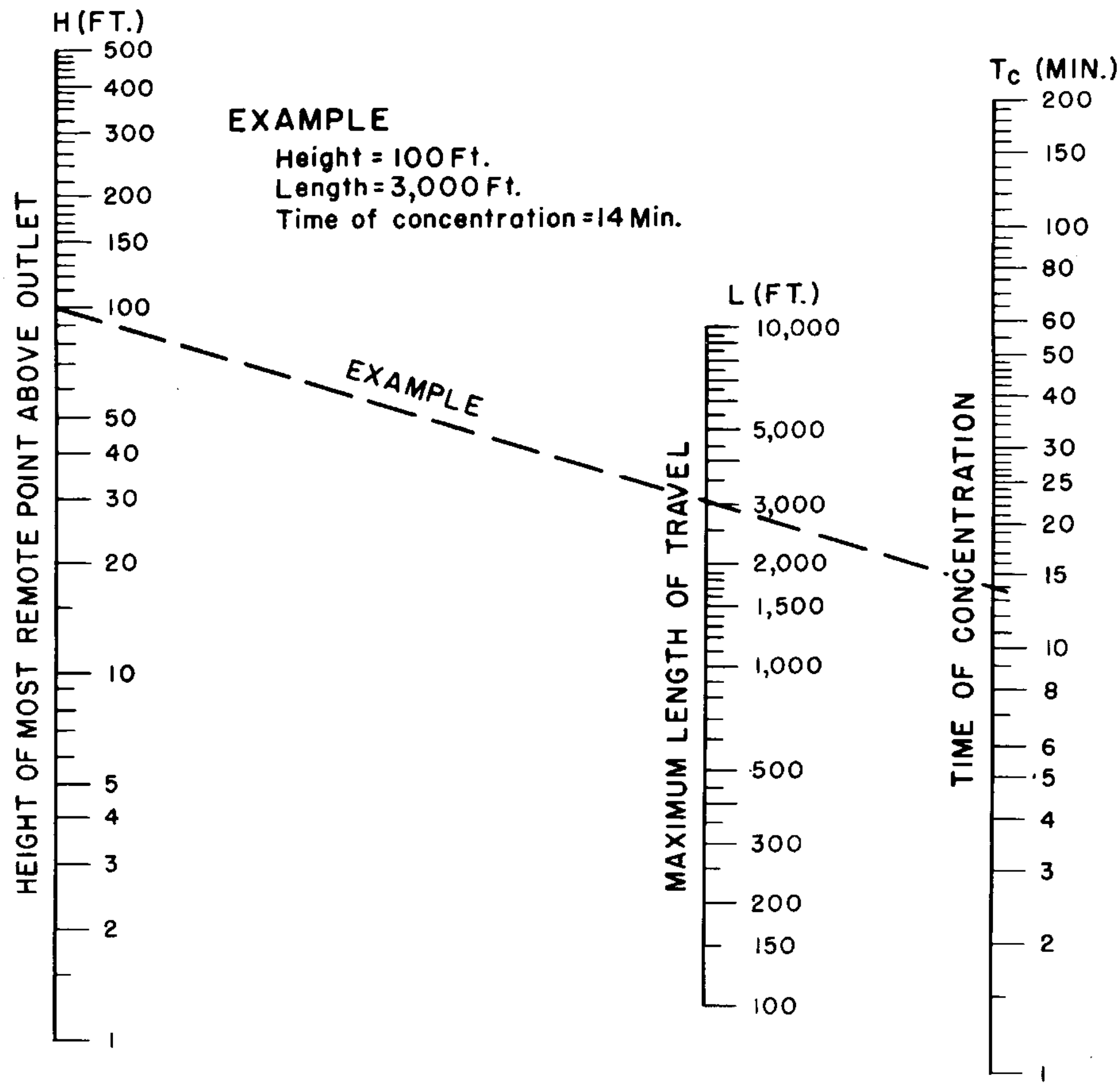
The rainfall intensity (i) is the average rainfall rate in inches per hour for a duration equal to the time of concentration for a selected return period. The rational method assumes uniform rainfall intensity over the total watershed.

Once a return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from rainfall-intensity-duration (IDF) curves. The IDF curves in this manual are based on the rainfall data from NOAA Atlas 14 (see Section 4.5). The rainfall intensity for a particular location can be determined directly from the IDF curve generated by entering the latitude and longitude of the project in the Precipitation Frequency Data Server Website (PFDS see Section 4.5). Alternatively, the rainfall intensity can be determined by selecting the appropriate rainfall intensity zone for West Virginia from Map 4-2 (Ohio Valley, Appalachian Plateau, Greenbrier, Ridge and Valley and Eastern Panhandle). Each zone corresponds to a regionalized set of IDF curves on Chart 4-2, Chart 4-3, or Chart 4-4.

Chart 4-1

Kirpich Method

T_c for Overland and Channel Flow Segments for Rural Basins



Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p. 362

$$T_C = \frac{0.0078\, L^{0.77}}{S^{0.385}}$$ S is in ft / ft

Map 4-2

Rainfall Intensity-Frequency Regions of West Virginia

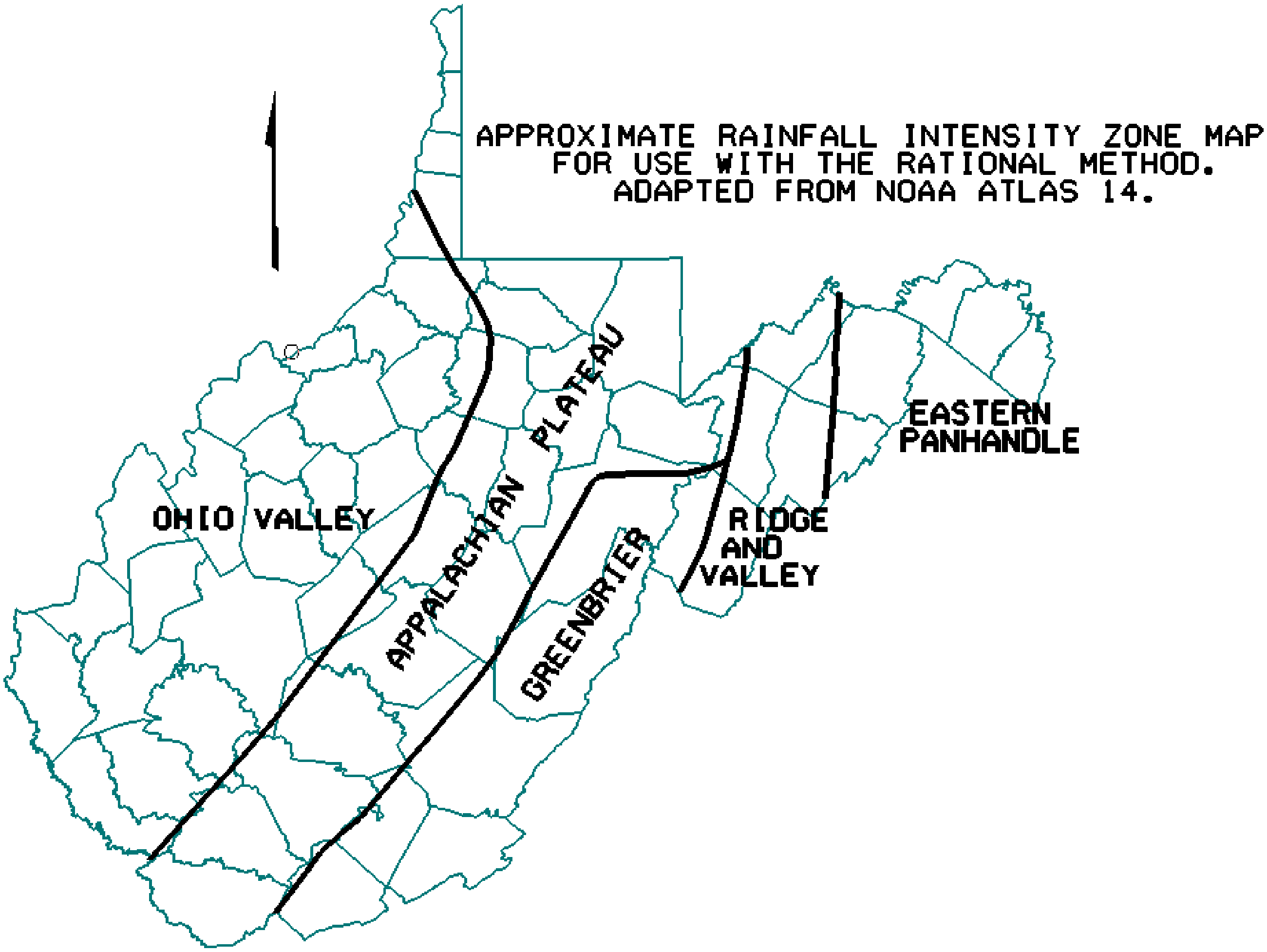
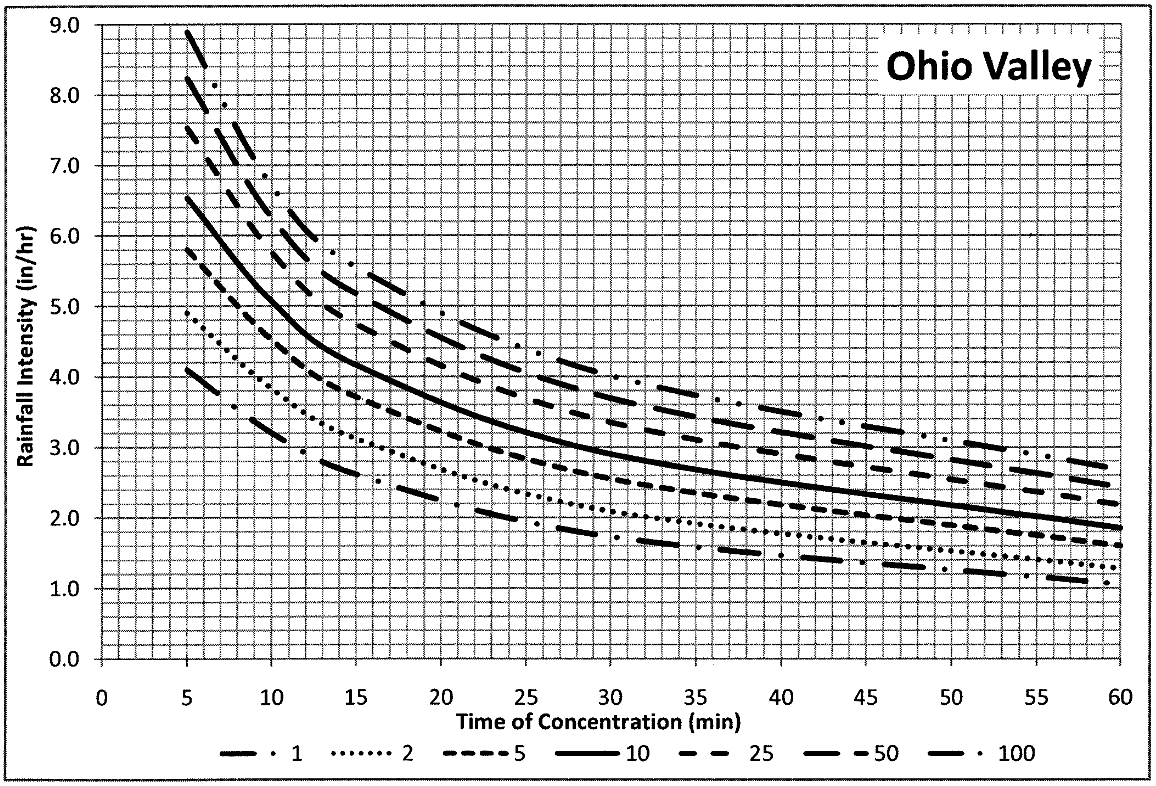


Chart 4-2

Intensity-Duration-Frequency Curves for West Virginia



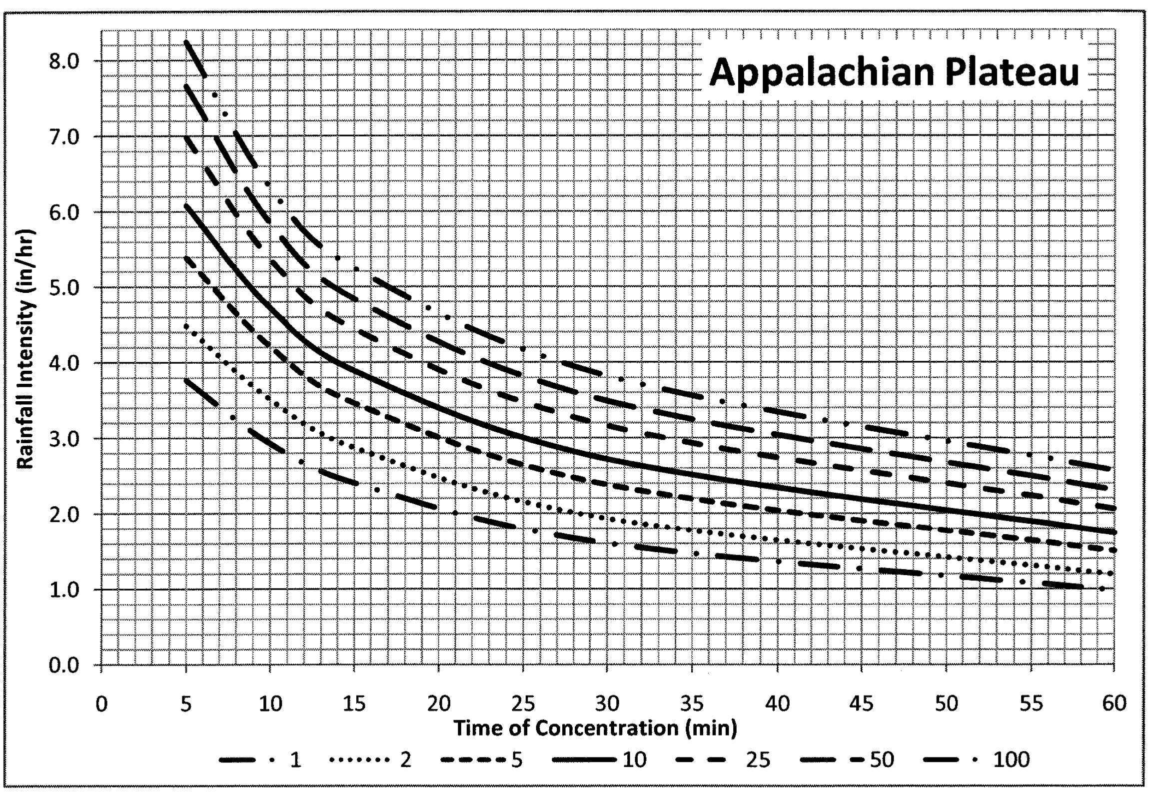
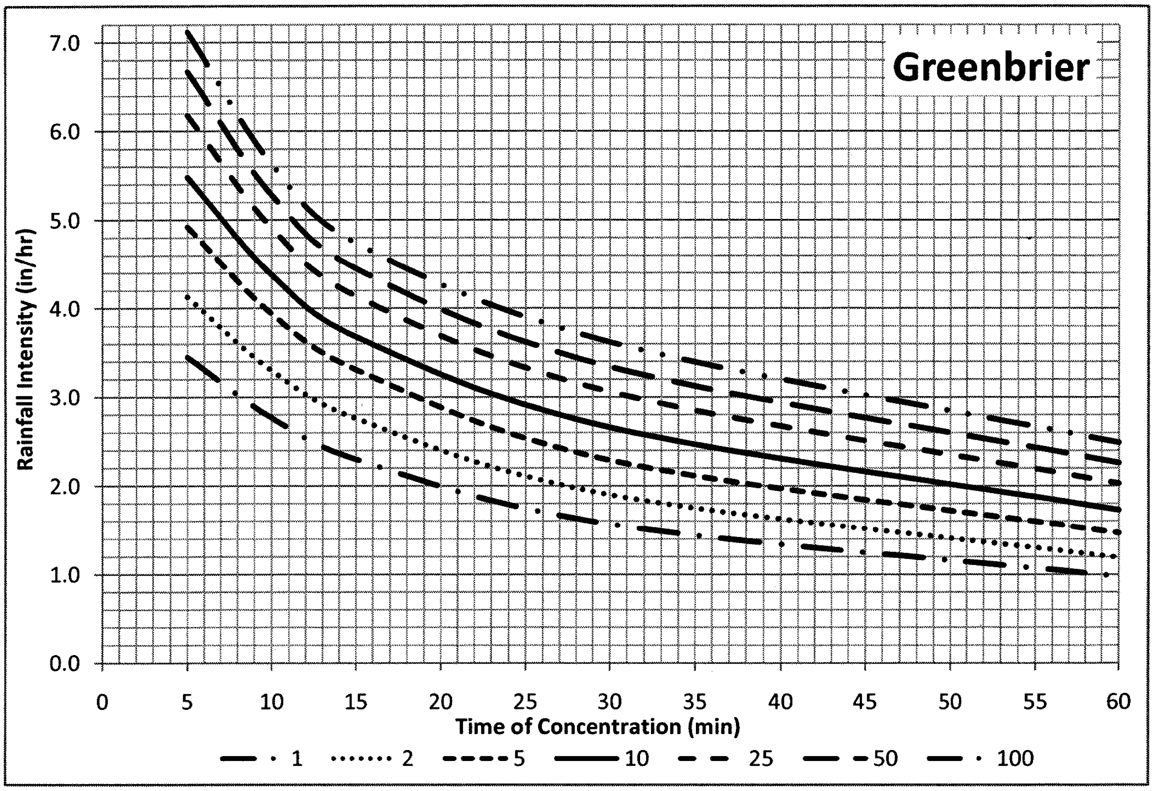


Chart 4-3

Intensity-Duration-Frequency Curves for West Virginia



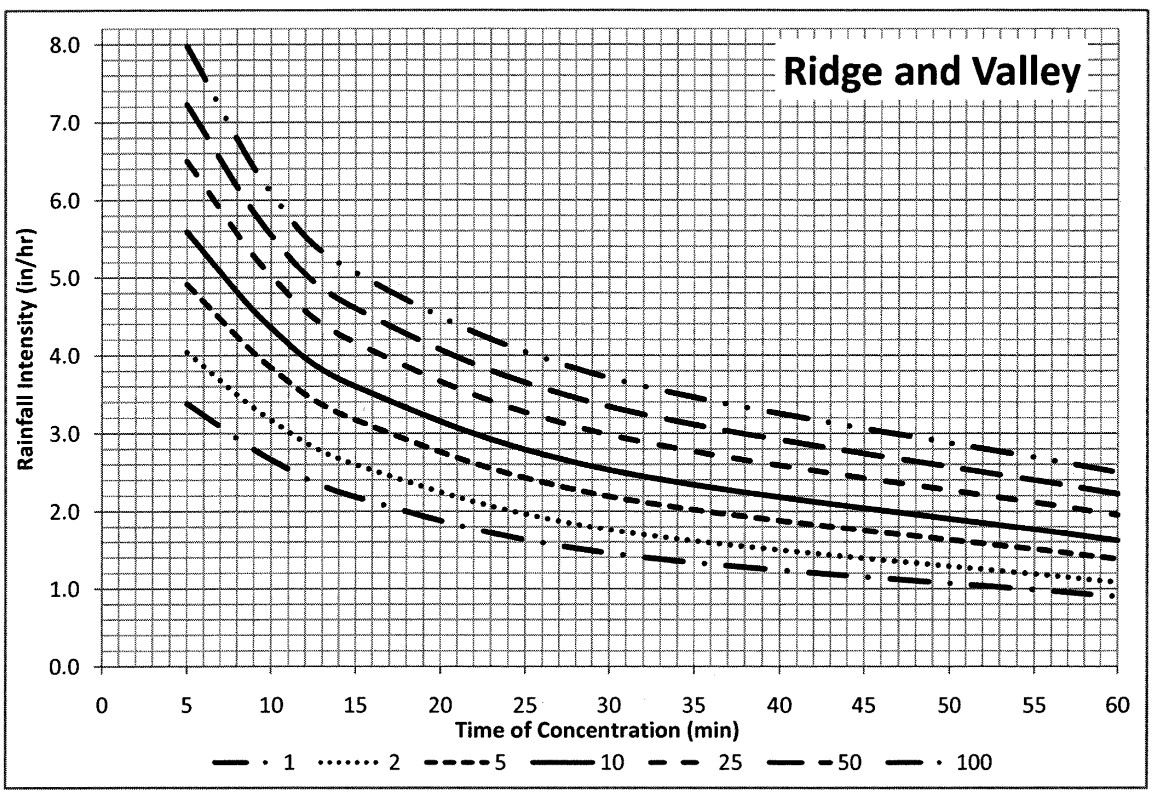
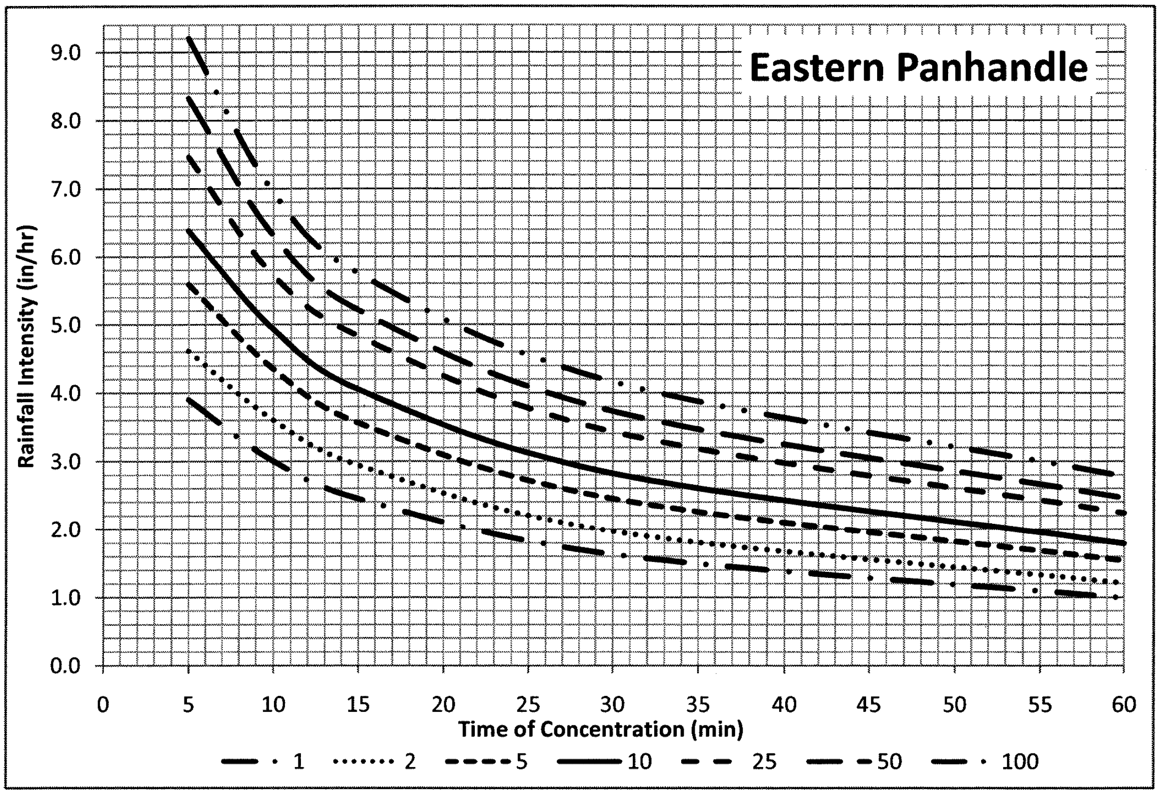


Chart 4-4

Intensity-Duration-Frequency Curves for West Virginia



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004

4.4.3 TR-55 GRAPHICAL PEAK DISCHARGE METHOD

4.4.3.1 APPLICATION

The NRCS publication Urban Hydrology for Small Watersheds, Technical Release No. 55 (TR-55) dated June 1986, presents a simplified procedure for computing peak discharge for rural and urban areas. This method was developed from hydrograph analyses using NRCS' computer program, TR-20 (see Section 4.5). It is recommended for estimating the peak discharge for areas between 5 and 16,000 acres. A Windows® based computer program for TR-55 was released in 2003 and is available on NRCS website.

Engineering applications of the TR-55 Method primarily pertain to ditch design and culvert design. This is due to the presence of all three different types of flow (sheet flow, shallow concentrated flow and channel flow) occurring within the drainage area. TR-55 may be used with urban drainage design if you have an offsite area flowing to an inlet along the roadway or flowing to an inlet that is connected to the roadway storm sewer. It may also be used for the hydraulic design of bridges if the drainage area is small enough to warrant comparison with the USGS method.

4.4.3.2 LIMITATIONS

- The watershed must be hydrologically homogeneous, that is, describable by one curve number. For such watersheds the land use, soils, and cover are distributed uniformly throughout the watershed.
- The watershed may have only one main stream or, if more than one, the branches must have nearly equal times of concentration.
- The pond and swamp adjustment factor can be applied only for ponds or swamps that are not in the time of concentration flow path.
- The initial abstraction term (Ia) was generalized as 20% of the potential maximum retention after runoff begins (S). This value was chosen based on data from agricultural watersheds. This approximation can be important in an urban application because of the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. To use a relationship other than Ia = 0.2S, one must redevelop the equation for Q by using the original rainfall-runoff data to establish new S or CN relationships for each cover and hydrologic soil group.
- Accuracy of peak discharge estimated by this method will be reduced if Ia/P values are used that are outside the range given in Chart 4-8 (0.10 to 0.50). The limiting Ia/P values are recommended for use.

- Time of concentration values may range from 0.1 to 10 hours.
- This method should be used only if the weighted curve number is greater than 40.

4.4.3.3 EQUATION

The graphical peak discharge equation is as follows:

$$q_p = q_u \ A_m \ Q \ F_p$$

Where: q_p = Peak discharge in cubic feet per second (cfs)

q_u = Unit peak discharge in cfs per square mile per inch (mi^2 / in)

A_m = Drainage Area in square miles (mi^2)

Q = Runoff in inches (in); and

F_p = Pond and swamp adjustment factor.

The input requirements for this method are as follows:

- Time of concentration (T_c) in hours
- Drainage area in square miles
- Rainfall distribution (West Virginia is a Type II)
- 24-hour rainfall depth in inches, and
- Curve Number.

If pond and swamp areas are spread throughout the watershed and are not in the T_c flow path, an adjustment for these areas is needed. Table 4-8 lists F_p values for the percentage of the watershed that are pond and swamp areas.

The 24 hour rainfall depths are provided in Map 4-3 through Map 4-9. These maps show the range of depth as they are shown in NOAA Atlas 14 (see Section 4.5). Using the rainfall depth in the middle of this range is recommended for design purposes. For a more precise value use the National Weather Service Precipitation Frequency Data Server at the web address http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=wv and input the project location.

Table 4-8

Adjustment Factor (F_p) for Pond and Swamp Areas

Percentage of 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

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

The runoff (Q) is defined by the SCS runoff equation:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Where: Q = Runoff Depth (inches)

P = 24 Hour Rainfall Depth for the return period (inches)

S = Potential maximum retention after runoff begins (inches)

I_a = Initial abstraction (inches).

Initial abstraction (I_a) represents runoff losses, such as water retained in surface depressions, intercepted by vegetation, evaporation and infiltration before runoff begins. Through studies of many small watersheds, I_a was approximated by the following empirical equation:

$$I_a = 0.2S$$

Substituting the above approximation into the runoff equation simplifies it and allows Q to be calculated for known values of P and S:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

S is related to the soil and ground cover (land use) conditions of the watershed through the Curve Number (CN). The curve number has a range from 40 to 100 and S is related to CN by:

$$S = \frac{1000}{CN} - 10 \quad \therefore \quad I_a = 0.2\left(\frac{1000}{CN} - 10\right)$$

4.4.3.4 CURVE NUMBER

The major factors that determine the curve number are the hydrologic soil group (HSG), cover type, hydrologic condition, and antecedent runoff condition.

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four hydrologic soil groups (A, B, C, and D) according to their minimum infiltration rate which is obtained for bare soil after prolonged wetting. Group A generally has the smallest runoff potential and group D generally has the greatest. Definitions for the four groups can be found in Appendix A of the TR-55 publication. The soils in the watershed of interest may be identified from a soil survey map which can be obtained from their website at the present address of www.wv.nrcs.usda.gov/soils.html. Once the soil is identified by name, the hydrologic group can be obtained via Appendix A of the TR-55 publication. The NRCS web soil survey is the most expedient way to obtain soil and hydrologic soil group information which is found here: http://websoilsurvey.nrcs.usda.gov/app/.

Hydrologic condition indicates the effects of the cover type (vegetated, bare soil, impervious surfaces etc.) on infiltration and runoff. A good hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group and cover type. Some factors to consider in estimating the hydrologic condition (effect of cover on infiltration and runoff) are: the density of vegetative cover, the degree of surface roughness, and the percent of cover that is not part of the major cover type for the basin area (residue cover).

The measure of runoff potential before a storm event occurs is the antecedent runoff condition (ARC). The ARC is an attempt to account for the variation in the curve number at a site from storm to storm. The curve number for the average ARC at a site is the median value as taken from sample rainfall and runoff data. It is this value that is used for design purposes. The curve numbers in Table 4-9 and Table 4-10 represent average antecedent runoff conditions for urban and rural areas.

Table 4-9

Runoff Curve Numbers for Rural Areas

Cover description Cover type and hydrologic condition	Curve numbers for the hydrologic soil group A	B	C	D
Pasture, grassland, or range land with continuous forage for grazing.				
Poor condition (< 50% cover, heavily grazed)	68	79	86	89
Fair condition (50% - 75% cover, not heavily grazed)	49	69	79	84
Good condition (> 75% cover, lightly grazed)	39	61	74	80
Meadow with continuous grass cover, protected from grazing and generally mowed.	30	58	71	78
Brush-weed-grass mixture with brush as the major element.				
Poor condition (< 50% cover)	48	67	77	83
Fair condition (50% - 75% cover)	35	56	70	77
Good condition (> 75% cover)	30	48	65	73
Woods 50% cover - grass 50% cover (such as an orchard or tree farm)				
Poor condition	57	73	82	86
Fair condition	43	65	76	82
Good condition	32	58	72	79
Woods only				
Poor condition (small trees, brush, forest litter)	45	66	77	83
Fair condition (medium trees, heavy brush, forest litter)	36	60	73	79
Good condition (large trees, thick undergrowth, undisturbed forest area)	30	55	70	77
Farm areas with buildings, driveways, gravel access roads mowed fields	59	74	82	86

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Table 4-10

Runoff Curve Numbers for Urban Areas

Cover description Cover type and hydrologic condition		Curve numbers for the hydrologic soil group A	B	C	D
Fully developed urban areas with established vegitation					
Open areas such as lawns, parks, golf courses etc. equivalent to pasture. For other combinations determine a composite curve number.					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% - 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas					
Paved parking lots, roofs, driveways etc.		98	98	98	98
Paved roads, curb and gutter		98	98	98	98
Concrete open ditches		83	89	92	93
Gravel roadway		76	85	89	91
Soil roadway		72	82	87	89
Urban districts	Average % impervious area				
Commercial and buisness	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acre	12	46	65	77	82
Developing urban areas					
Newly graded areas, all pervious with no vegetation Composite curve numbers should be used if this cover type represents an area under construction.		77	86	91	94
Idle lands not currently under development should be treated as rural areas see Table 4-		-	-	-	-

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Urban Impervious Area Modifications

Factors such as the percentage of impervious area and the means of conveying runoff from those impervious areas to the drainage system should be considered in determining the curve number for urban areas. For example, do the impervious areas connect directly to the drainage system or do they outlet onto lawns or other pervious areas where infiltration can occur before they reach the drainage system?

Connected impervious areas

An impervious area is considered connected if runoff from it flows directly into the drainage system. An example of this would be inlets located in a curb and gutter section along the roadway. It is also considered connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system. An example of this would be sheet flow runoff from a parking lot that outlets at a curb opening at the low point of the lot to a rock lined ditch that leads to a type G inlet. The flow at the outlet would be considered shallow concentrated flow that leads through the rock lined ditch, which would be considered a pervious area.

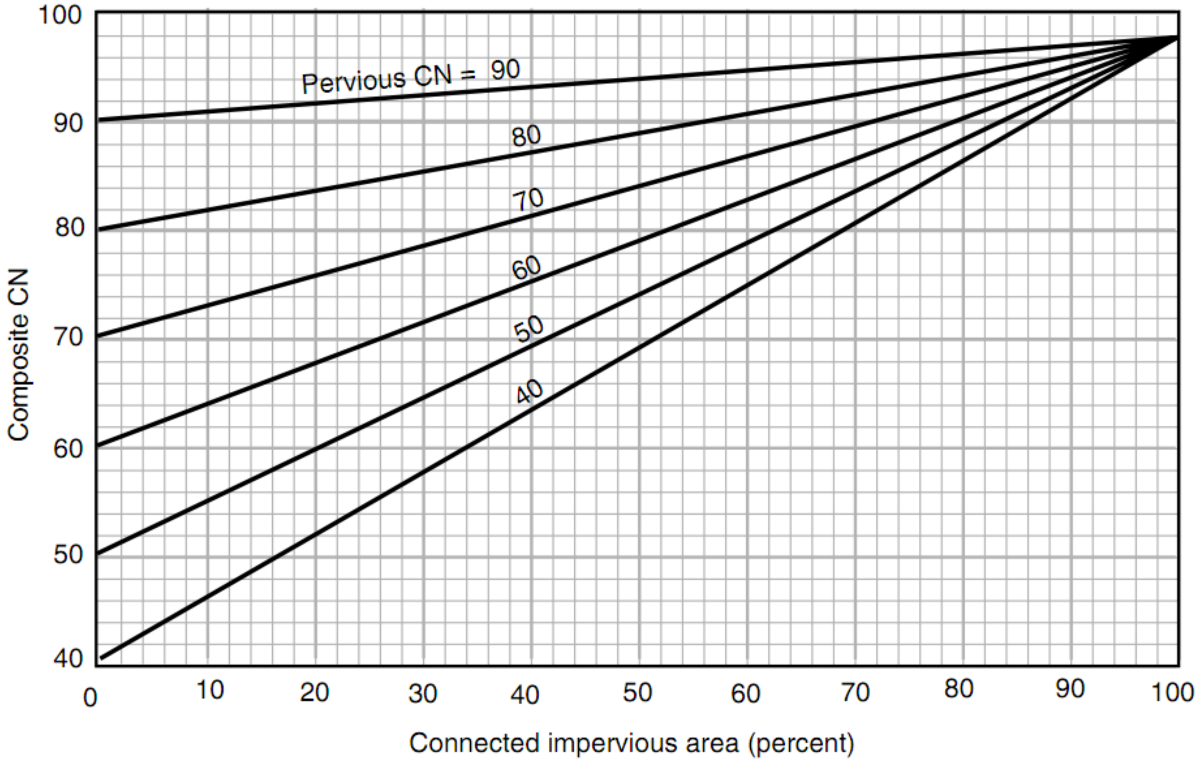
The urban curve numbers in Table 4-10 were developed on the assumptions that pervious urban areas are equivalent to pasture in good hydrologic condition with impervious areas that have a curve number of 98 and are directly connected to the drainage system. If all of the impervious area is directly connected to the drainage system but the impervious area percentages or the pervious land use assumptions in Table 4-10 are not applicable, use the top part of Chart 4-5 to determine a composite curve number.

Unconnected impervious areas

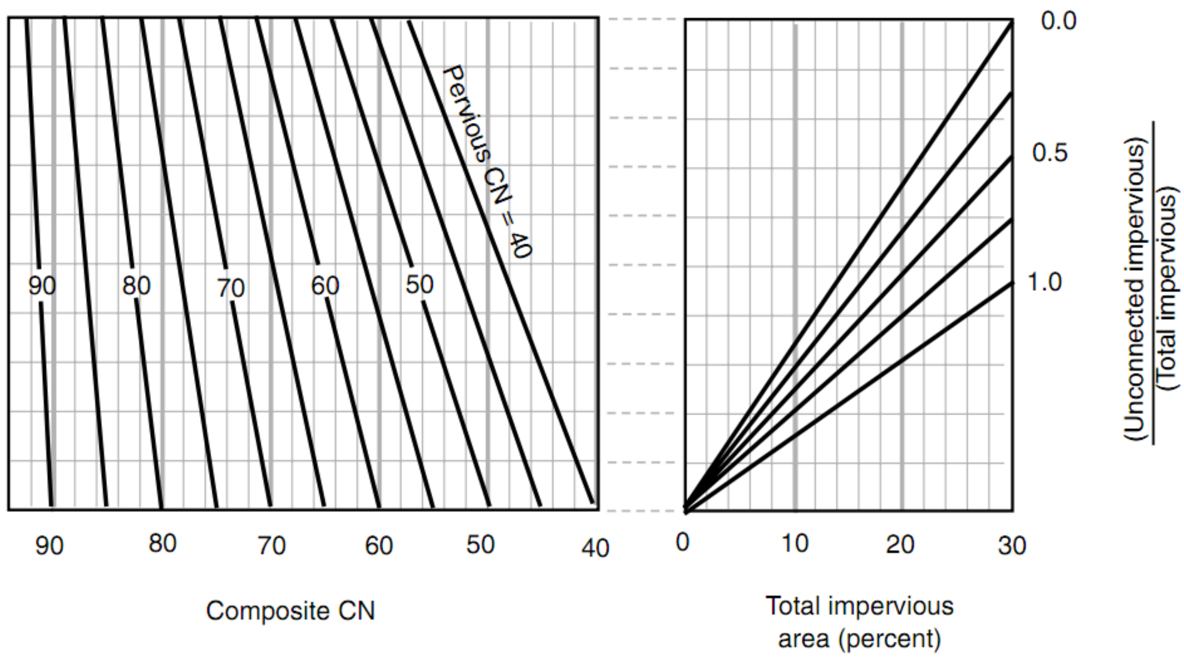
Runoff from these areas is spread over a pervious area as sheet flow before it reaches the drainage system. To determine the curve number when all or part of the impervious area is not directly connected to the drainage system and the total impervious area is less than 30%, use the bottom part of Chart 4-5. If the unconnected impervious area is greater than or equal to 30% of the drainage area use the top part of Chart 4-5. For unconnected impervious areas of this size, the absorptive capacity of the remaining pervious areas will not significantly affect the runoff.

Chart 4-5

Curve Number Modification for Urban Impervious Area



Composite Curve Number with Connected Impervious Area



Composite Curve Number with Unconnected Impervious Area

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

4.4.3.5 TIME OF CONCENTRATION

The TR-55 Method breaks down the overland flow segment into sheet flow and shallow concentrated flow for calculating the Time of Concentration (T_c). Water is conceptualized as moving through a watershed as sheet flow, shallow concentrated flow, open channel flow or a combination of these. The flow components that occur in a particular watershed are best determined by field inspection.

The travel time (T_t) is the ratio if the flow length to flow velocity.

$$T_t = \frac{L}{3600\ V}$$

Where: T_t = travel time in hours

L = flow length in feet

V = average velocity in feet per second

The total time of concentration T_c, in hours, is the sum of travel time T_t values for the consecutive flow segments:

$$T_C = T_{t\ \text{sheet flow}} + T_{t\ \text{shallow concentrat ed flow}} + T_{t\ \text{channel flow}}$$

Sheet Flow - It is a shallow mass of runoff on a plane surface with the depth staying uniform across the sloping surface. Typically, flow depths will not exceed two inches. A modified version of the original form of the kinematic wave equation was developed for this method. It replaces rainfall excess intensity with a 24 hour rainfall depth for a 2 year or 50% frequency event. The sheet flow T_t is determined from the 2 year event depth regardless of the 24 hour return period depth used to determine the total runoff.

$$T_{t\ \text{sheet flow}} = \frac{0.007(n\ L)^{0.8}}{\sqrt{P_{2-24}}\ S^{\ 0.4}}$$

Where: T_t = sheet flow travel time in hours

n = roughness coefficient (Table 4-5)

L = flow length in feet

P_{2-24} = the 2 year 24 hour rainfall depth in inches

S = slope of the surface in ft / ft

The 2-year, 24 hour rainfall depth can be taken from Map 4-4 which was created from NOAA Atlas 14 data. This time of concentration equation yields values for overland sheet flow equal to or less than 100 feet, thus sheet flow shall be limited to 100 feet in length.

The SCS developed four dimensionless rainfall distributions using earlier versions of NOAA Rainfall Frequency Atlases. Data analyses indicated four major regions for the U.S. and the resulting rainfall distributions were labeled type I, IA, II, and III. The type I and type II distributions plot as straight lines on log-log graphs. Although they do not agree exactly with the IDF values from all locations in the region for which they represent, the differences are within the accuracy of the rainfall depths taken from the rainfall atlases.

This version of the kinematic wave equation was developed by using the best fit straight line for each curve from the log-log graph. The equation for each line was then solved for the rainfall intensity and substituted into the original kinematic wave equation. A single weighted equation was then created from the two which resulted in the version presented above. This version is applicable for the regions covered by the type II (includes WV) and type III rainfall distributions. It assumes that rainfall excess intensity equals rainfall intensity which is reasonable for asphalt and other virtually impermeable surfaces. For short distanced permeable planes the error introduced by this assumption is partially counterbalanced by the lack of a perfectly flat plane. A lower intensity would increase the travel time while channelization on an irregular surface would decrease the travel time.

Shallow Concentrated Flow - Beyond 100 feet, flow tends to concentrate. The travel time for this flow segment is determined by an average velocity method. The average velocity for this type of flow can be determined from Chart 4-7 or the following equations. The average velocity is a function of watercourse slope and the type of channel.

$$\text{Unpaved} \qquad V = 16.1345 \sqrt{S}$$

$$\text{Paved} \qquad V = 20.3282 \sqrt{S}$$

Where: V = average velocity in feet / second

S = watercourse slope in ft / ft

Travel time is then calculated by dividing the length of flow by the velocity.

$$T_{t\ \text{shallow concentrated flow}} = \frac{L}{V * 3600}$$

Where: T_t = shallow concentrated flow travel time in <u>hours</u>

L = flow length in feet

V = velocity in feet / second

Open Channel Flow - Manning's equation is used to estimate the average flow velocity in the channel or pipe and the travel time is calculated by dividing the channel or pipe length by the average flow velocity.

$$V = \frac{1.49}{n} R^{\frac{2}{3}} \sqrt{S}$$

Where: V = velocity in feet / second

n = Manning's Roughness Coefficient (Table 4-7)

R = hydraulic radius in feet (flow area / wetted perimeter)

S = slope of the surface in ft / ft

$$T_{t\ \text{channel or pipe flow}} = \frac{L}{V * 3600}$$

Where: T_t = shallow concentrated flow travel time in <u>hours</u>

L = flow length in feet

V = velocity in feet / second

The travel time through reservoirs or lakes is normally very small and can be assumed to be zero.

In urban areas the channel flow may lead to pipe flow that continues down the watershed as a storm sewer. As stated in the Rational Method it is common error to underestimate this travel time thus inflate the peak discharge. Changes in pipe geometry, direction, and entrances and exits from inlets tend to slow down the flow. If details of the flow path cannot be determined it may be necessary to dye trace and time the flow through the system for an actual field measurement of the time of concentration.

4.4.3.6 COMPUTATION PROCEDURE

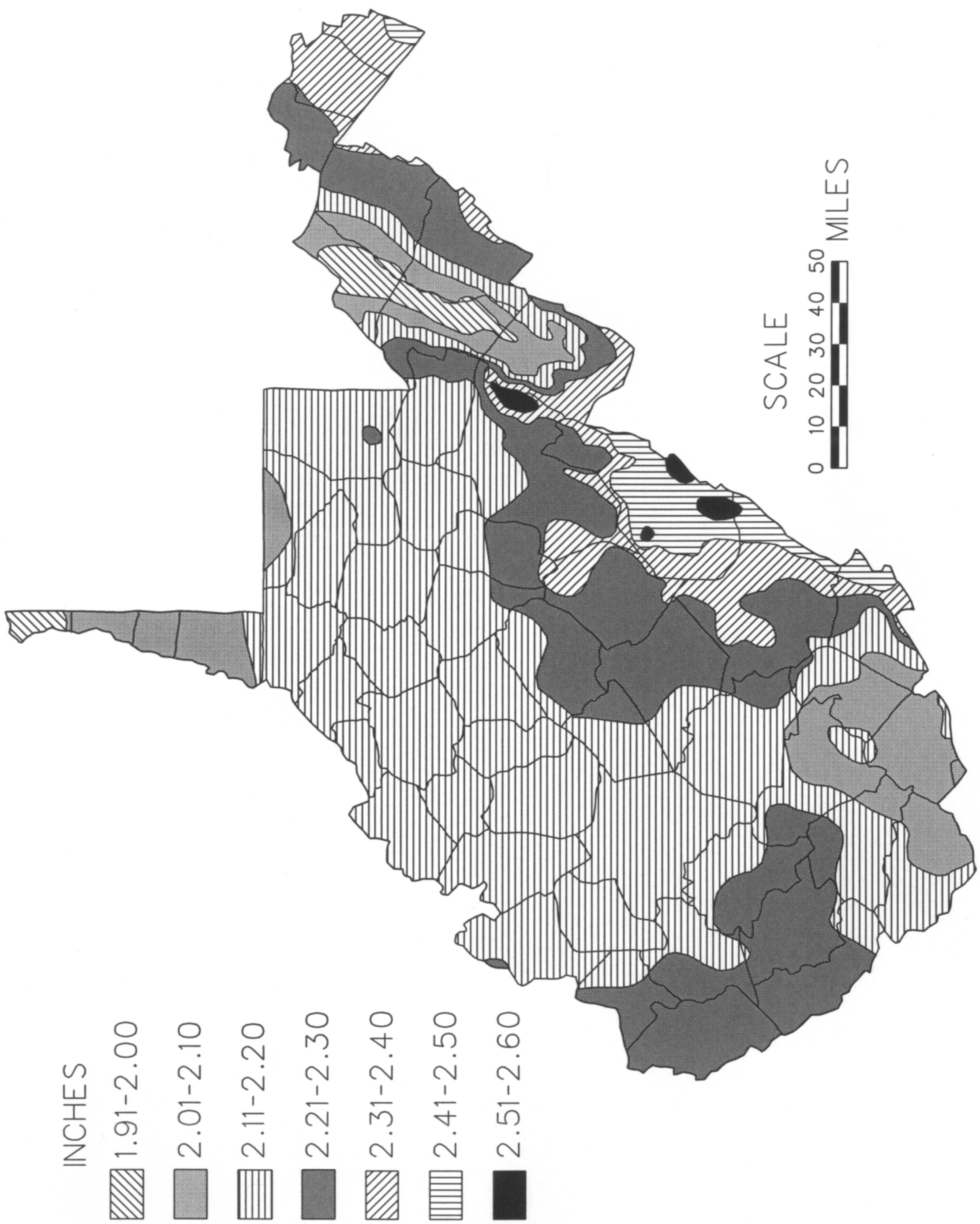
The NRCS' TR-55 computer program may be used for computing the peak discharge. Alternatively, Worksheet 4-1, Worksheet 4-2, and Form 4-1 may be used for calculating the runoff curve number, time of concentration, and peak discharge respectively.

The following steps outline the computation procedure for this method:

- Determine the Drainage Area (A_m) in square miles from a topographic map.
- Determine the Runoff Curve Number (CN) using Worksheet 4-1.
- Obtain the 24-hour Rainfall Depth (P) in inches for a selected frequency or return period from Map 4-3 through Map 4-9 (NOAA Atlas 14 rainfall maps).
- Map 4-3 through Map 4-9. The maps give a range of values for the depth. The chosen value within that range will depend on the distance from the boundary line to the watershed location (graphically interpolate). In most cases this depth value is adequate; however, for a more precise value use the National Weather Service Precipitation Frequency Data Server at the web address http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=wv and input the project location.
- Determine the Runoff depth (Q) in inches (rounded to the nearest hundredth of an inch) using either Table 4-11 according to the values of P and CN or Chart 4-6 by solving the runoff equation.
- Determine the Time of Concentration (T_c in hours) using Worksheet 4-2.
- Use Table 4-12 determine the initial abstraction (I_a) in inches according to the CN and compute I_a/P.
- Use Chart 4-8 for Type II rainfall distribution to obtain the Unit Peak Discharge (q_u) in cfs per square mile per inch of runoff using T_c and I_a/P. If the computed I_a/P ratio is outside the range of the chart, then the limiting value shall be used. If the ratio falls between the limiting values use linear interpolation to determine the unit peak discharge.
- Obtain the Pond and Swamp Adjustment Factor (F_p) from Table 4-8.
- Calculate the peak discharge (q_p) for the selected rainfall frequency by multiplying the values of q_u, A_m, Q, and F_p.

Map 4-3

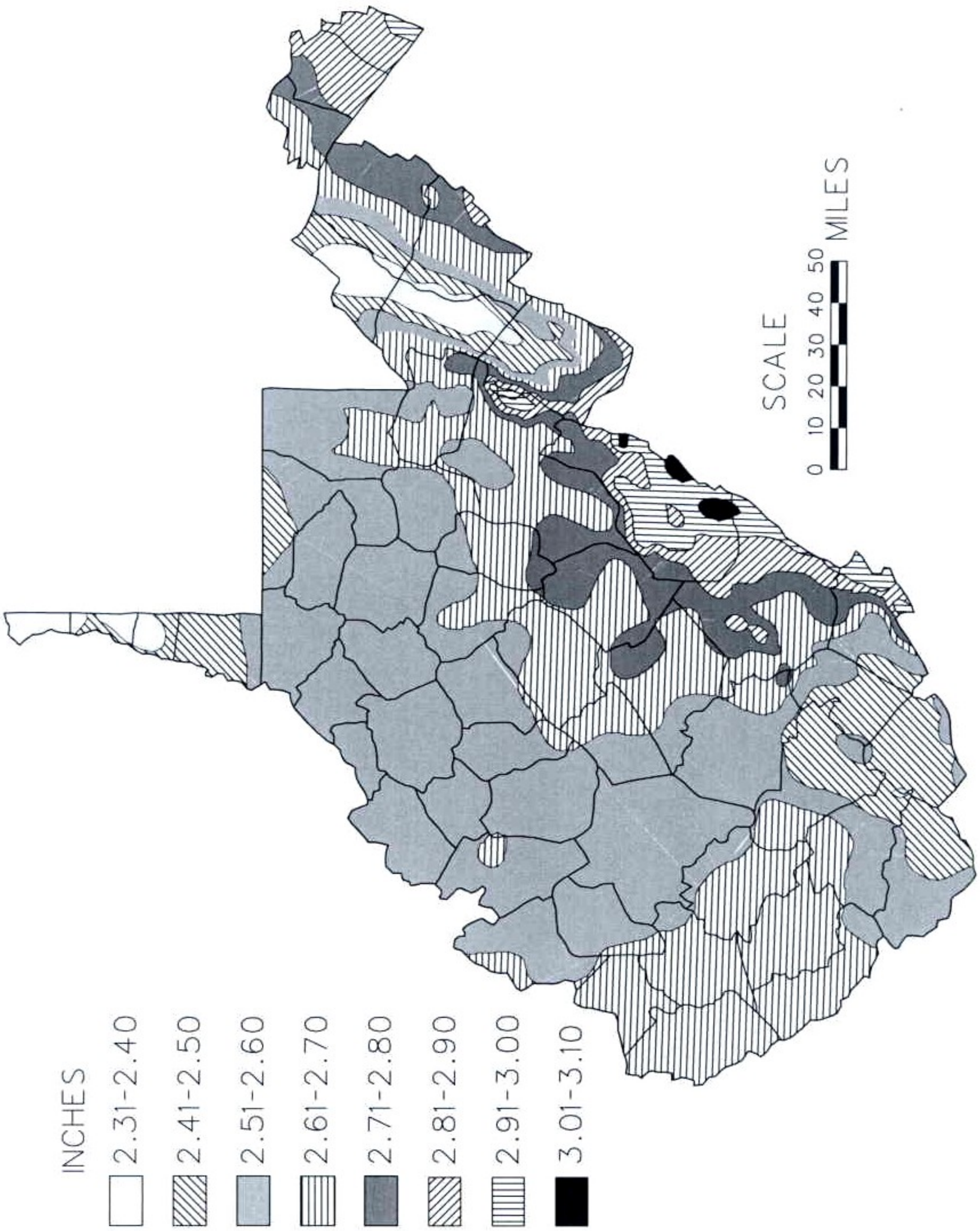
24-hour Rainfall Depth for a 1 Year Return Period



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

Map 4-4

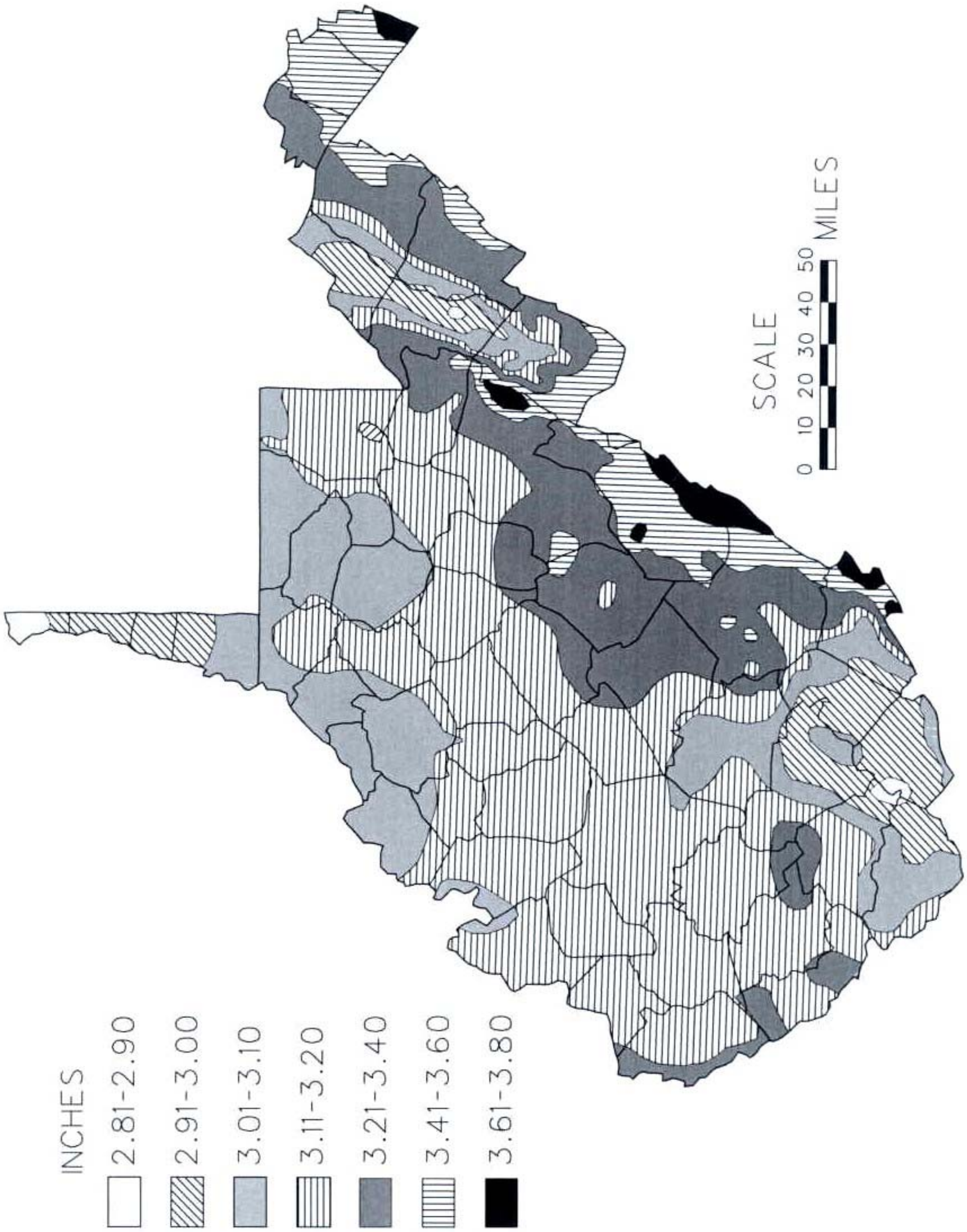
24-hour Rainfall Depth for a 2 Year Return Period



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

Map 4-5

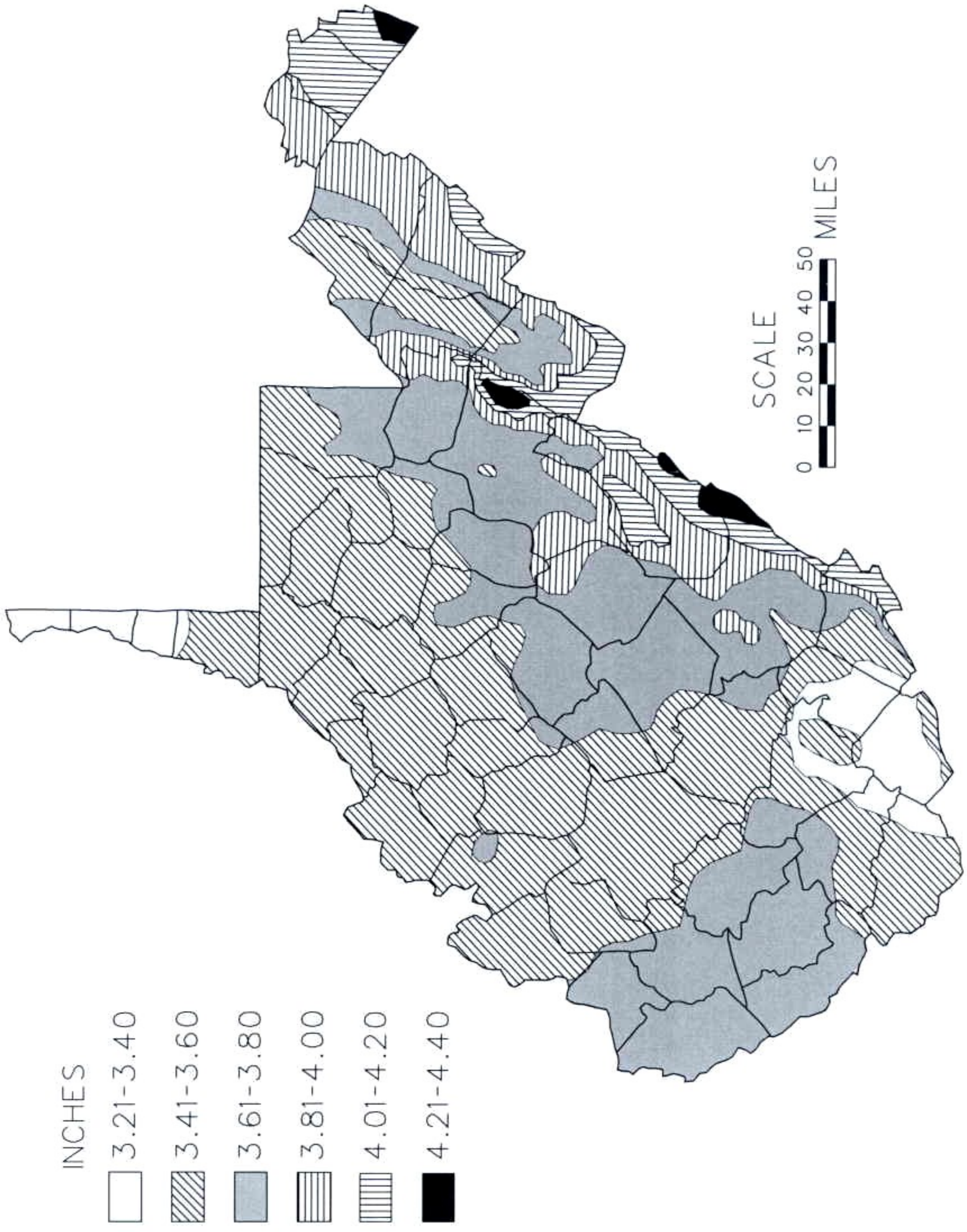
24-hour Rainfall Depth for a 5 Year Return Period



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

Map 4-6

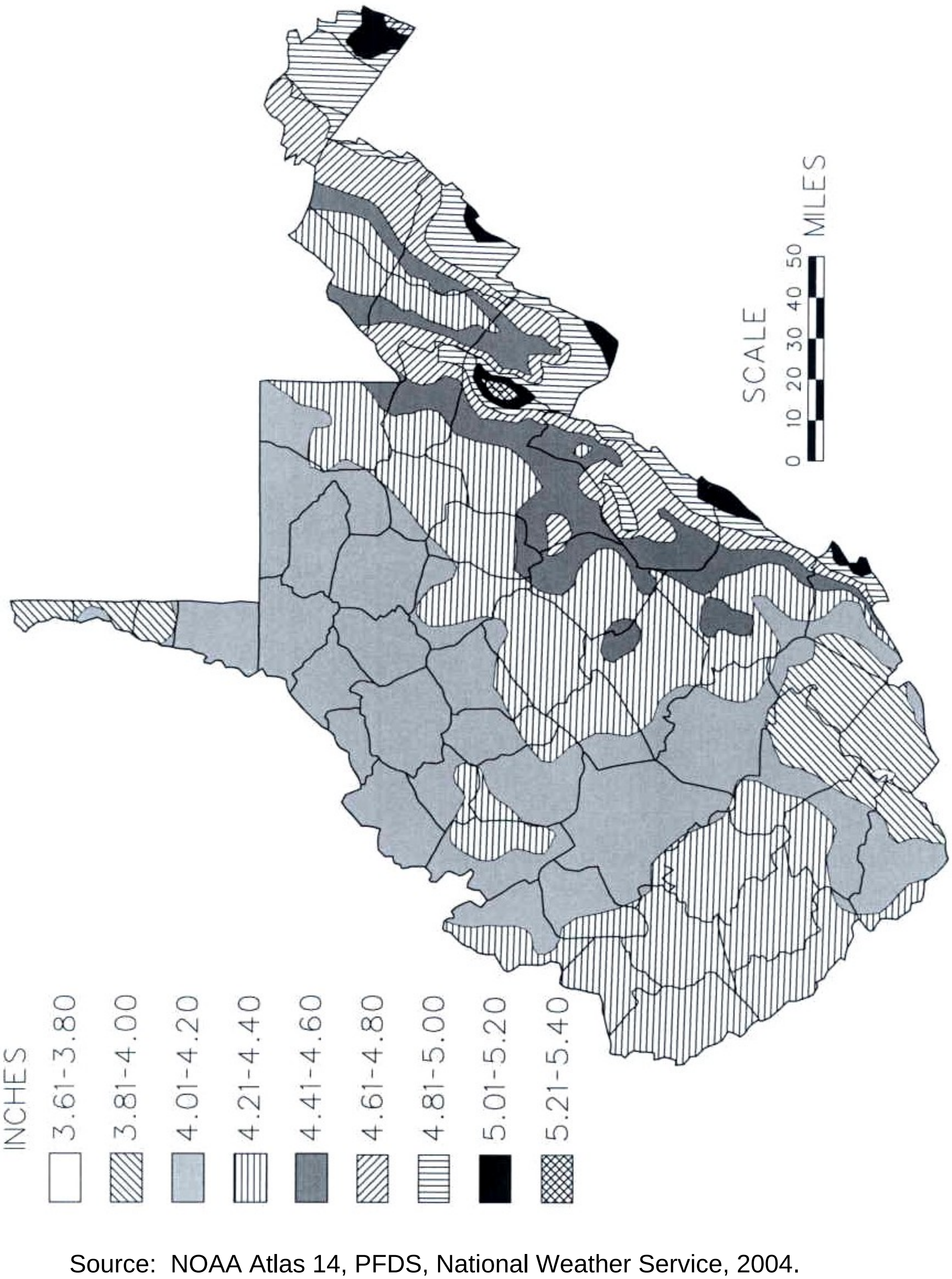
24-hour Rainfall Depth for a 10 Year Return Period



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

Map 4-7

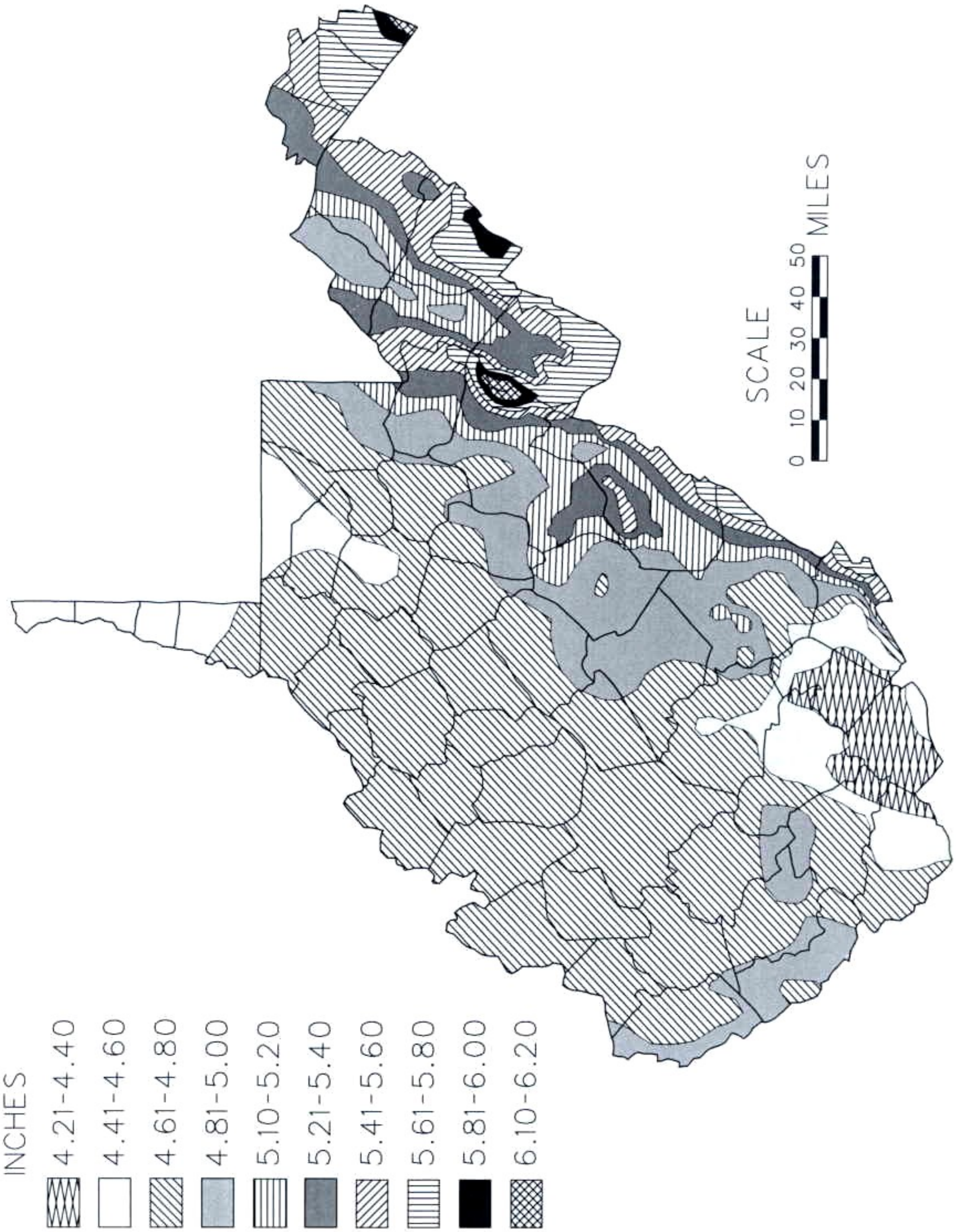
24-hour Rainfall Depth for a 25 Year Return Period



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

Map 4-8

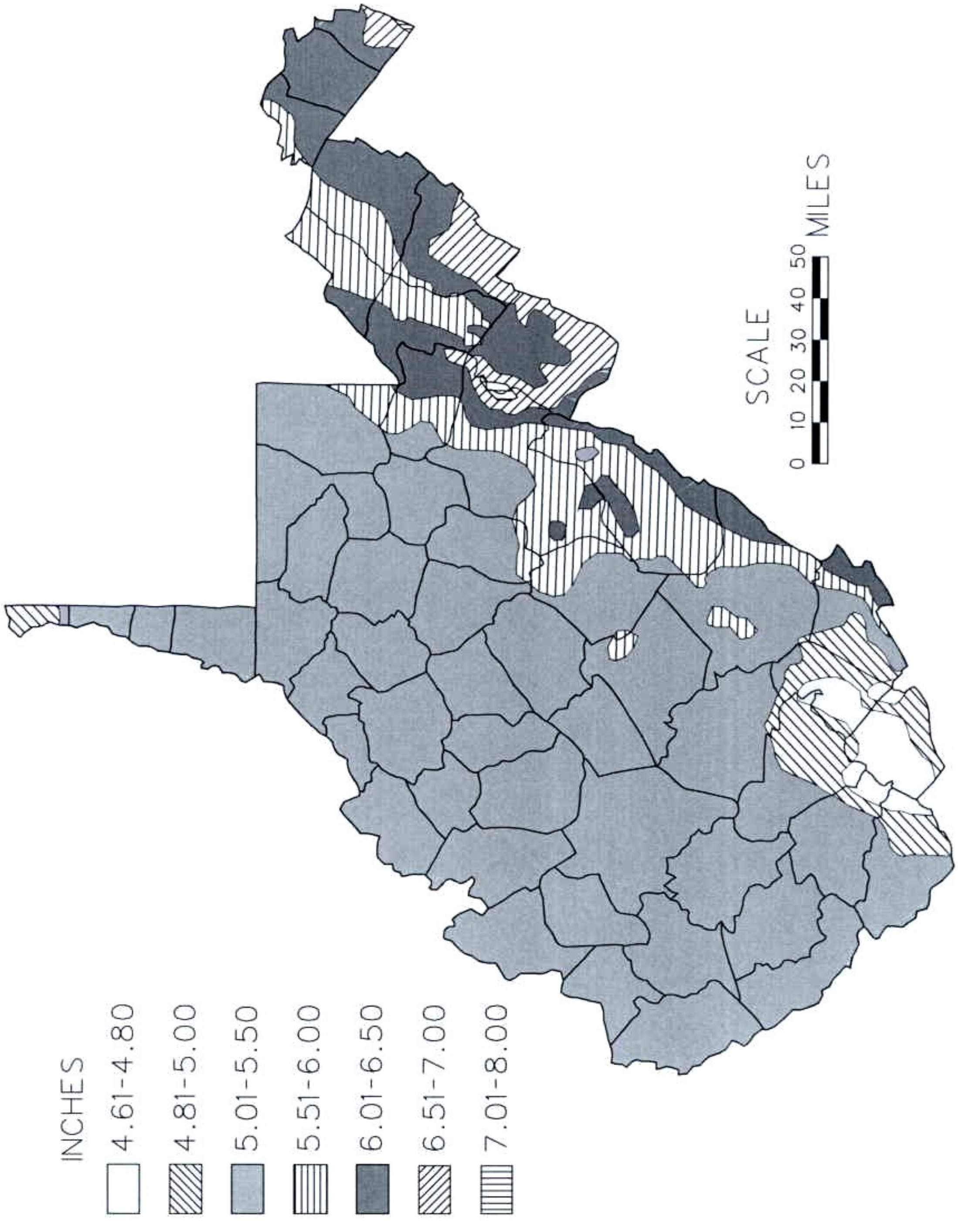
24-hour Rainfall Depth for a 50 Year Return Period



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

Map 4-9

24-hour Rainfall Depth for a 100 Year Return Period



Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

Table 4-11

Runoff Depth for Selected CN & 24 Hr Rainfall Depth

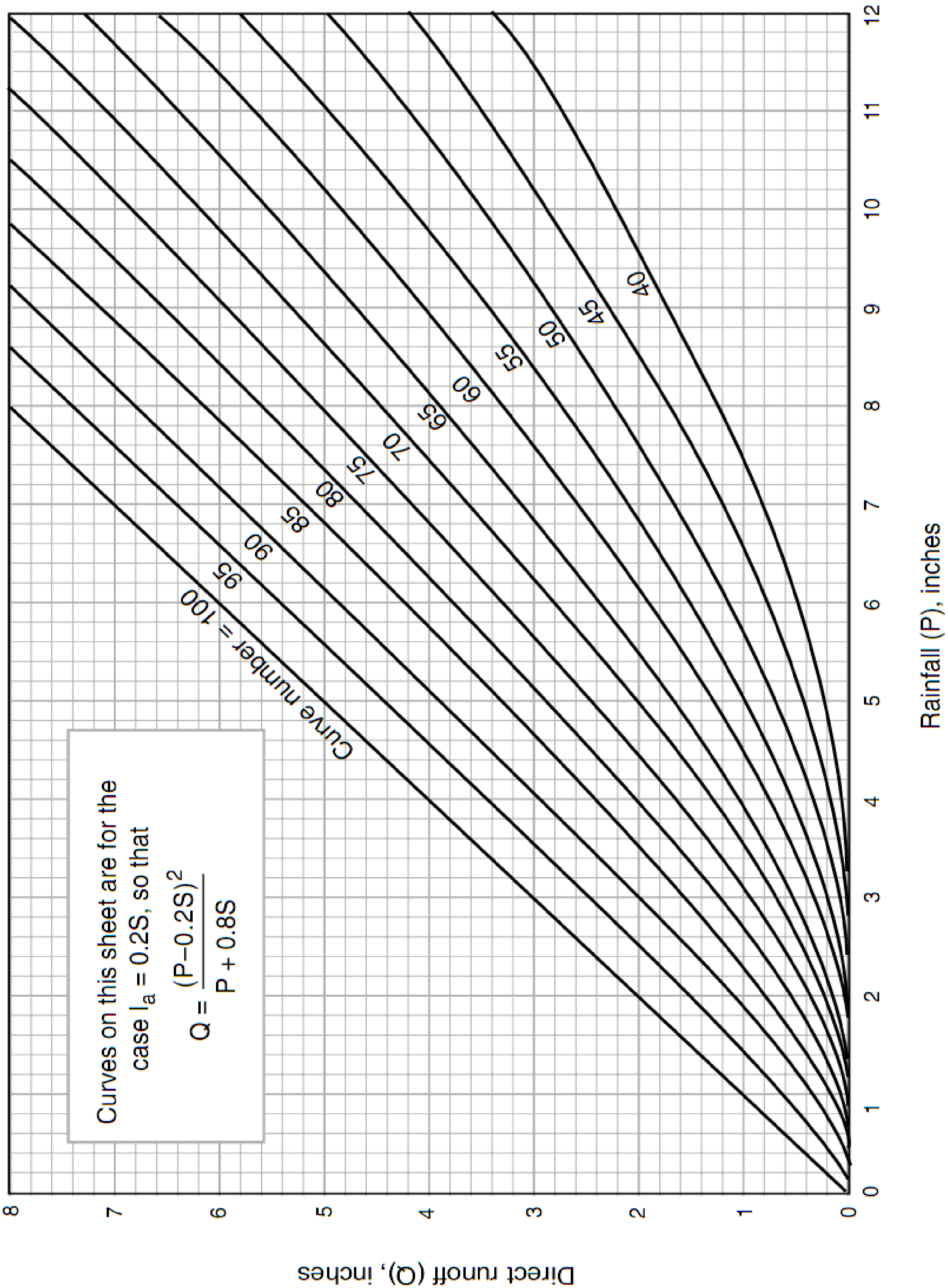
Rainfall	Runoff depth for curve number of—												
	40	45	50	55	60	65	70	75	80	85	90	95	98
	inches												
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	.00	.00	.00	.00	.00	.00	.03	.07	.15	.27	.46	.74	.99
1.4	.00	.00	.00	.00	.00	.02	.06	.13	.24	.39	.61	.92	1.18
1.6	.00	.00	.00	.00	.01	.05	.11	.20	.34	.52	.76	1.11	1.38
1.8	.00	.00	.00	.00	.03	.09	.17	.29	.44	.65	.93	1.29	1.58
2.0	.00	.00	.00	.02	.06	.14	.24	.38	.56	.80	1.09	1.48	1.77
2.5	.00	.00	.02	.08	.17	.30	.46	.65	.89	1.18	1.53	1.96	2.27
3.0	.00	.02	.09	.19	.33	.51	.71	.96	1.25	1.59	1.98	2.45	2.77
3.5	.02	.08	.20	.35	.53	.75	1.01	1.30	1.64	2.02	2.45	2.94	3.27
4.0	.06	.18	.33	.53	.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
4.5	.14	.30	.50	.74	1.02	1.33	1.67	2.05	2.46	2.91	3.40	3.92	4.26
5.0	.24	.44	.69	.98	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	.50	.80	1.14	1.52	1.92	2.35	2.81	3.28	3.78	4.30	4.85	5.41	5.76
7.0	.84	1.24	1.68	2.12	2.60	3.10	3.62	4.15	4.69	5.25	5.82	6.41	6.76
8.0	1.25	1.74	2.25	2.78	3.33	3.89	4.46	5.04	5.63	6.21	6.81	7.40	7.76
9.0	1.71	2.29	2.88	3.49	4.10	4.72	5.33	5.95	6.57	7.18	7.79	8.40	8.76
10.0	2.23	2.89	3.56	4.23	4.90	5.56	6.22	6.88	7.52	8.16	8.78	9.40	9.76
11.0	2.78	3.52	4.26	5.00	5.72	6.43	7.13	7.81	8.48	9.13	9.77	10.39	10.76
12.0	3.38	4.19	5.00	5.79	6.56	7.32	8.05	8.76	9.45	10.11	10.76	11.39	11.76
13.0	4.00	4.89	5.76	6.61	7.42	8.21	8.98	9.71	10.42	11.10	11.76	12.39	12.76
14.0	4.65	5.62	6.55	7.44	8.30	9.12	9.91	10.67	11.39	12.08	12.75	13.39	13.76
15.0	5.33	6.36	7.35	8.29	9.19	10.04	10.85	11.63	12.37	13.07	13.74	14.39	14.76

1/ Interpolate the values shown to obtain runoff depths for CN's or rainfall amounts not shown.

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Chart 4-6

Runoff Depths for Selected CN & 24 Hr Rainfall Depth



Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Worksheet 4-1

Runoff Curve Number Determination

WORKSHEET 4-1 RUNOFF CURVE NUMBER DETERMINATION

CALCULATED BY: ____________ DATE: ___________ PROJECT NAME: ______________________

CHECKED BY: ____________ DATE: ___________ STATE PROJECT NUMBER: ______________________

Soil Name	Hydrologic Group	Cover Description percent impervious unconnected/connected impervious area ratio	CN Source: Table 4-9	CN Source: Table 4-10	CN Source: Chart 4-5	Area in mi^2	CN X Area

one CN source per line

Totals → | | |

Weighted CN = Total CN X Area / Total Area

Weighted Curve Number	
Potential Maximum Retention, S in inches	

	Storm #1	Storm #2
Return Period in years		
24 Hour Rainfall Depth, P in inches		
Runoff Depth, Q in inches		

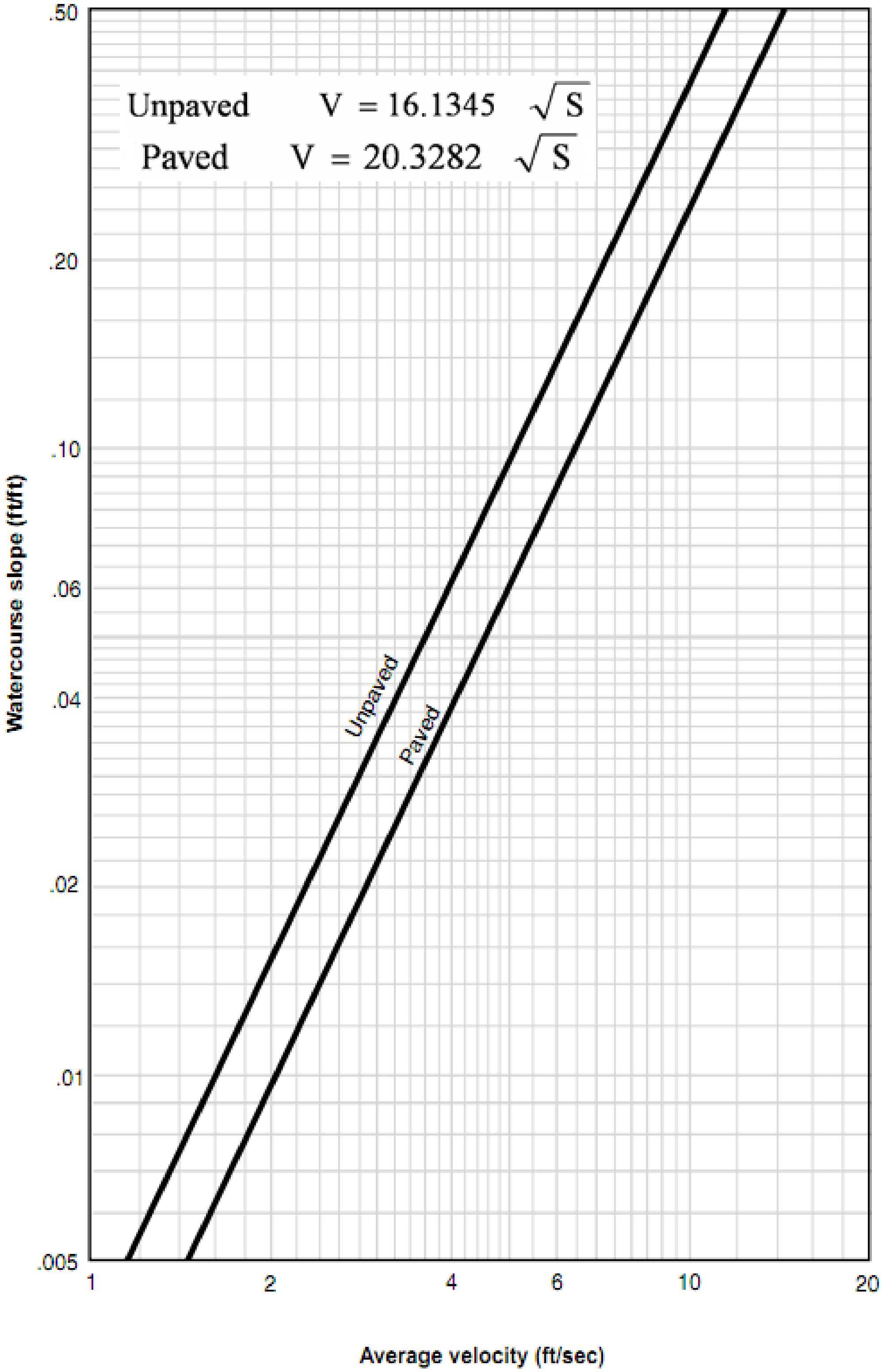
24 hour Rainfall Depth from Table 4-11, or Map 4-3 through Map 4-8

Runoff Depth from Table 4-12 or Chart 4-6

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Chart 4-7

Average Velocity for Shallow Concentrated Flow



Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Worksheet 4-2
Time of Concentration Calculation

WORKSHEET 4-2 TIME OF CONCENTRATION COMPUTATION

CALCULATED BY: ________	DATE: ________	PROJECT NAME: ________
CHECKED BY: ________	DATE: ________	STATE PROJECT NUMBER: ________

Space for two sections per flow type can be used for each worksheet.
Include a map, schematic or description of the flow segements

OVERLAND FLOW SEGMENT, SHEET FLOW TYPE

Section ID		
Surface description (Table 4-5)		
Roughness coeff. n (Table 4-5)		
Flow length L in ft (should be ≤ 100 ft)		
2 Yr 24 Hr rainfall depth P in inches (Map 4-3)		
Land slope S in ft / ft		
Computed travel time T_t in hours		=

OVERLAND FLOW SEGMENT, SHALLOW CONCENTRATED FLOW TYPE

Section ID			
Cover type			
Surface cover coefficient in equation			
Watercourse slope S in ft / ft			
Average velocity V in ft / s (Chart 4-7)			
Flow length in ft			
Computed travel time T_t in hours	+	=	

note: overland flow (sheet flow + shallow concentrated flow should be < 200' urban areas, < 400' rural areas)

CHANNEL FLOW SEGMENT

Section ID			
Cross sectional flow area A in ft^2			
Wetted flow perimter P in ft			
Hydraulic radius R = A / P in ft			
Channel slope S in ft / ft			
Mannings roughness coeff. n (Table 4-7)			
Velocity from Mannings equation, V in ft / s			
Flow length L in ft			
Computed travel time T_t in hours	+	=	

Watershed time of concentration T_c in hours

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Table 4-12

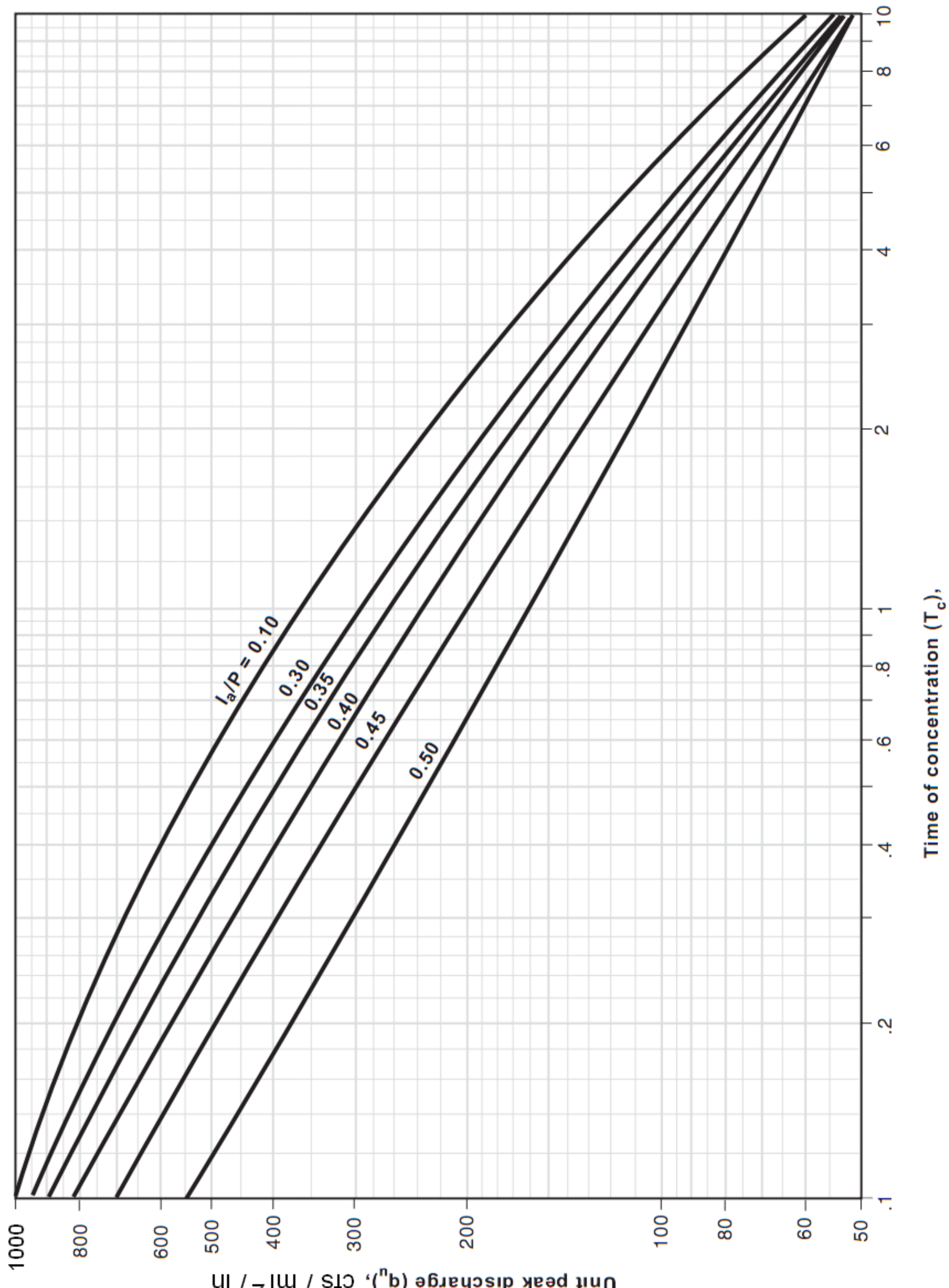
Initial Abstraction Values (I_a) for Curve Numbers

Curve number	I_a (in)	Curve number	I_a (in)
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		

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Chart 4-8

Unit Peak Discharge for Type II Rainfall Distribution



Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

Table 4-13

Unit Peak Discharge Equation

$$\log\left(q_u\right) = C_0 + C_1 \log\left(T_c\right) + C_2 \left[\log\left(T_c\right)\right]^2$$

Coefficients for the Unit Peak Discharge Equation according to the I_a / P value are given in the following table. Values between the range of 0.10 and 0.50 can be interpolated.

I_a / P	C_0	C_1	C_2
0.10	2.553	-0.615	-0.164
0.30	2.465	-0.623	-0.117
0.35	2.419	-0.616	-0.088
0.40	2.364	-0.599	-0.056
0.45	2.292	-0.570	-0.023
0.50	2.202	-0.516	-0.013

Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

In a case where an estimated T_c is being checked using a known unit peak discharge use this form of the equation:

$$T_c = 10^{\left(\frac{-C_1 \pm \sqrt{C_1^2 - 4C_2\left(C_o - \log q_u\right)}}{2C_2}\right)}$$

There will be two roots to this equation with one as the obvious answer. This form is useful when an estimated headwater at a pipe entrance for an estimated 24 hour rainfall depth (P) is assumed from field information and used to back calculate T_c. This back calculated value can be used to compare and calibrate the initial estimated T_c value.

4.4.4 USGS METHOD

4.4.4.1 APPLICATION

In 2010, the USGS, in cooperation with the West Virginia Division of Highways, revised previously developed regression equations for estimating the magnitude and frequency of peak discharges on rural, unregulated streams in WV. Data collected from many river gage stations were analyzed by multiple regression techniques to develop the equations. They are applicable for a wide range of drainage areas depending on the hydrologic region.

Map 4-10 shows a map of West Virginia with hydrologically homogeneous regions (Western Plateaus, Central Mountains, and Eastern Panhandle) having similar magnitude and frequency relationships. These regions are separated by topographic features. The boundary between the Eastern Panhandle and Central Mountains Regions follows the Potomac River Basin boundary. The Central Mountains Region contains the drainage areas of the Cheat River, Tygart Valley River, Elk River upstream from the confluence of Birch River, and the New and Gauley Rivers upstream of the Kanawha River. The Western Plateaus Region encompasses the remainder of the state. The boundaries between the hydrologic regions are not meant to be precise and more than one set of equations may apply for areas along the regional boundaries.

Where the findings from the regression equations are found to vary significantly (±10%) from historical stream gage data, the Log Pearson III Method should be used (with approval from the Director of Engineering) provided there is at least 10 years of continuous or synthesized stream gage record. Where the stream gage record is less than 10 years, prudent judgment along with consideration of the standard regression error shall be used in reaching a design decision.

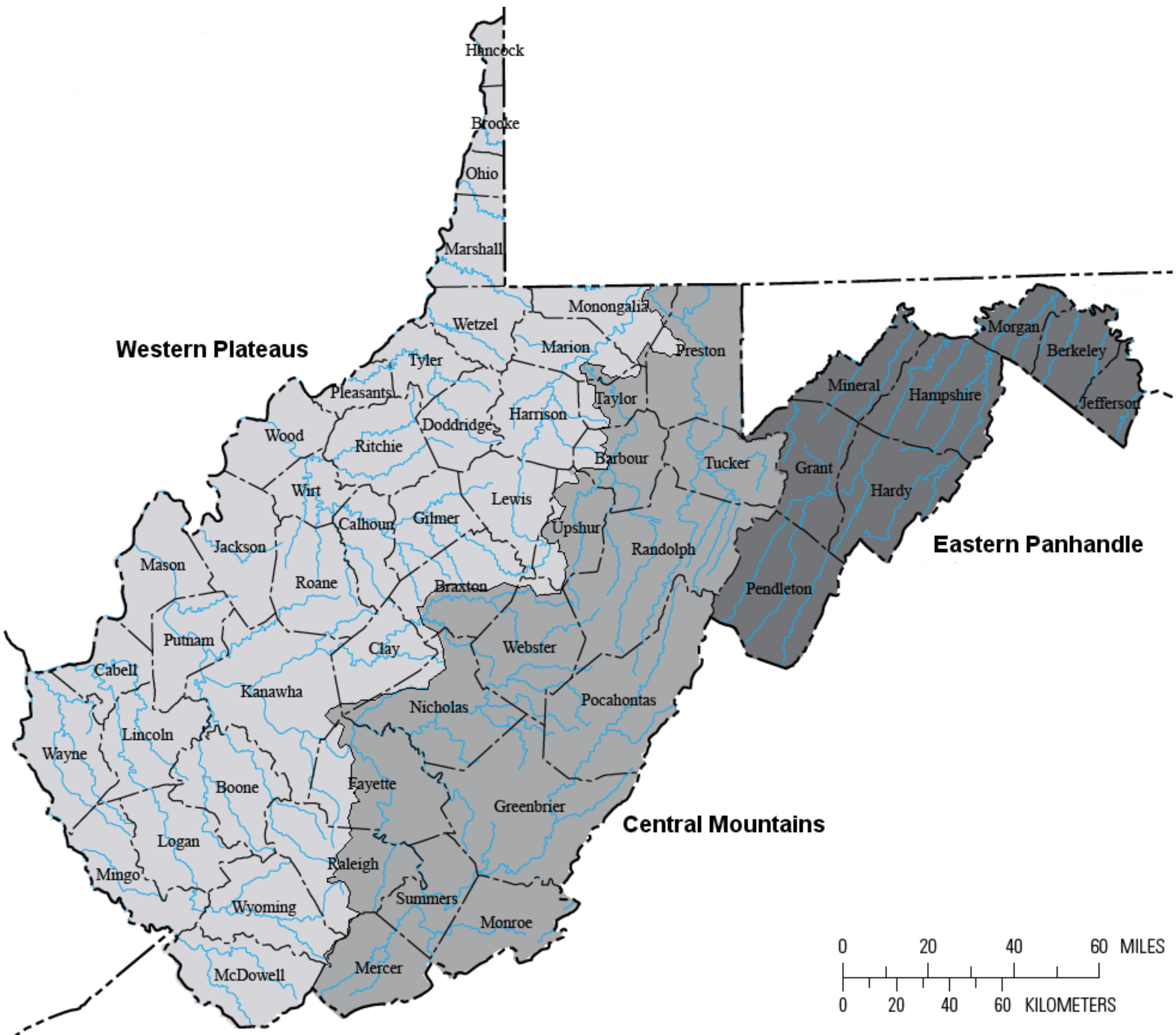
Engineering applications of the USGS Method primarily pertain to the hydraulic design of bridges and culverts.

4.4.4.2 LIMITATIONS

USGS discharge equations are based on discharge data from gages on rural streams with drainage areas greater than 10 square miles. The equations are appropriate to predict discharge values for rural streams with drainage areas greater than 10 square miles. The equations typically underestimate the discharge values for smaller streams; therefore they should not be used for final design purposes on rural streams with drainage areas less than 10 square miles. The equations may be used for preliminary design purposes in watersheds less than 10 square miles but greater than 5 square miles by adding the “standard prediction error” (between 18% and 54%) to the calculated value. USGS Scientific Investigation Report 2010-5033 provides a detailed explanation of this issue. The equations should also be avoided in karst regions where excessive runoff is diverted into, outside, or within the basin through solution channels or other cavities in carbonate (limestone and dolomite) rocks. The equations should not be used in areas where the discharges have been altered significantly by dams or flood detention structures.

Map 4-10

Hydrologic Regions in West Virginia



Source: USGS Scientific Investigations Report 2010-5033 (2010)

4.4.4.3 EQUATION

Table 4-14 presents the regression equations to calculate the 1.1, 1.5, 2, 5, 10, 25, 50, 100, 200 and 500-year discharges for the three hydrologic regions.

Table 4-14

USGS Regional Regression Equations for Rural Areas (2010)

DRAINAGE AREA (A) IS IN SQUARE MILES 1 MILE ^{2} = 640 ACRES 1 MILE ^{2} = 27,878,400 SQUARE FEET					
RECURRENCE INTERVAL OR RETURN PERIOD	EXCEEDENCE PROBABILITY OR FREQUENCY	REGRESSION EQUATION	STANDARD ERROR OF MODEL IN PERCENT	AVERAGE STANDARD ERROR OF SAMPLING IN	AVERAGE PREDICTION ERROR IN PERCENT
EASTERN PANHANDLE REGION					
1.1	90%	$29.6\ A^{0.818}$	43.4	10.3	44.8
1.5	67%	$46.4\ A^{0.828}$	35.7	8.9	36.9
2	50%	$59.8\ A^{0.832}$	32.1	8.6	33.4
5	20%	$105\ A^{0.838}$	25.6	8.9	27.2
10	10%	$145\ A^{0.842}$	22.5	9.5	24.5
25	4%	$204\ A^{0.848}$	19.7	10.3	22.4
50	2%	$254\ A^{0.852}$	18.6	11.1	21.7
100	1%	$307\ A^{0.855}$	18.3	11.6	21.7
200	0.50%	$365\ A^{0.859}$	18.4	12.4	22.4
500	0.20%	$447\ A^{0.864}$	19.4	13.5	23.8
CENTRAL MOUNTAINS REGION					
1.1	90%	$33.4\ A^{0.914}$	40.0	8.3	41.0
1.5	67%	$53.8\ A^{0.887}$	34.6	7.3	35.4
2	50%	$69.4\ A^{0.873}$	33.4	7.3	34.2
5	20%	$116\ A^{0.845}$	34.1	8.0	35.1
10	10%	$153\ A^{0.831}$	36.3	8.6	37.4
25	4%	$206\ A^{0.816}$	39.9	9.8	41.2
50	2%	$250\ A^{0.807}$	42.9	10.6	44.4
100	1%	$297\ A^{0.800}$	46.2	11.3	47.9
200	0.50%	$347\ A^{0.793}$	49.7	12.0	51.5
500	0.20%	$420\ A^{0.785}$	54.3	13.1	56.3
WESTERN PLATEAUS REGION					
1.1	90%	$56.9\ A^{0.763}$	38.2	7.6	39.1
1.5	67%	$97.8\ A^{0.741}$	33.4	6.5	34.1
2	50%	$129\ A^{0.730}$	31.6	6.1	32.2
5	20%	$221\ A^{0.710}$	29.3	6.5	30.0
10	10%	$292\ A^{0.699}$	28.9	6.5	29.7
25	4%	$391\ A^{0.688}$	29.4	7.3	30.3
50	2%	$472\ A^{0.681}$	30.2	7.6	31.3
100	1%	$557\ A^{0.674}$	31.4	8.0	32.5
200	0.50%	$647\ A^{0.668}$	32.7	8.3	33.9
500	0.20%	$775\ A^{0.661}$	34.8	8.9	36.1

Source: USGS SIR Report 2010-5033 (2010)

4.4.5 HYDROGRAPH METHODS

Three hydrograph methods are discussed. The choice of which method to use should be based on the watershed size and complexity.

The NRCS TR-55 Tabular Hydrograph Method is a simplified procedure based on TR-20. It was created to avoid the computer calculations for TR-20 in the 1980's. TR-20 was run several times for many different watersheds to create a set of representative tables. It can be used to create partial composite hydrographs at any point in a watershed by dividing it into homogeneous sub-areas. The designer should refer to the Technical Release-55 User's Manual, Urban Hydrology for Small Watersheds, for detailed guidance.

The Modified Rational Method is a simplified hydrograph procedure for small, non-complex, homogeneous watersheds.

The SCS Unit Hydrograph Method is a detailed procedure for large, complex, non-homogeneous watersheds which involve calculations that require the use of computer programs such as the Hydrologic Modeling System (HEC-HMS developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center, HEC), and Technical Release No.20 (TR-20 developed by the U.S. Department of Agriculture's NRCS). The designer should refer to the SCS National Engineering Handbook, Part 630 Hydrology for detailed guidance.

4.4.5.1 TR-55 TABULAR HYDROGRAPH METHOD

This method shows tabular discharge values for various rainfall distributions. West Virginia has a Type II rainfall distribution. These tables were developed by computing hydrographs for 1 square mile of drainage area for selected time of concentrations and routing them through stream reaches with a range of travel times. The resulting runoff estimates were used to convert the hydrographs to cubic feet per second per square mile per inch of runoff from the watershed. An assumption in the development of the tabular hydrographs was that all discharges for a stream reach flow at the same velocity. By this assumption, the sub-area flood hydrographs may be routed separately and added at the reference point.

It is important to distinguish the difference between travel time and time of concentration with this method. Travel time is the time it takes water to travel from one location to another in a watershed. It is a component of time of concentration, which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. Time of concentration is computed by summing all the travel times for consecutive components of the drainage conveyance system.

4.4.5.2 MODIFIED RATIONAL METHOD

The Modified Rational Method can be used to develop and route trapezoidal shaped hydrographs for small drainage areas of generally 20 acres or less. Hydrographs produced by this method can also be used for sizing storm water detention ponds up to 200 acres in size for flood control or temporary sediment retention ponds for erosion control. The use of this method should be limited to the simplest watersheds (such as: no major changes in channel size or shape, no existing detention structures, the cover type is primarily uniform) or for preliminary design only.

The Modified Rational Method produces hydrographs based upon different duration storms of the same frequency with the following parameters:

- Time of concentration (T_c) = Time to peak (T_p)
- Time to recede (T_r) = T_p
- The duration D_e, of the storm is from 0 minutes until the time of selected duration.
- Base of hydrograph (T_b) = $D_e + T_r$
- The peak Q (top of trapezoidal hydrograph) is calculated using the intensity (i) value found on the rainfall IDF curve for the selected duration and frequency.
- Hydrographs are normally calculated for durations of t_c , 1.5t_c, 2t_c, and 3t_c.
- Longer duration hydrographs may need to be calculated if reservoir routing computations show that the water elevation in a basin is increasing with each successive hydrograph that is routed through the basin.

Hydrographs with durations less than t_c are not valid and should not be calculated.

The Modified Rational Method recognizes that the duration of a storm can be longer than the time of concentration. A longer duration storm can produce a larger volume of runoff than a duration equal to the time of concentration of the drainage area, even though the longer duration storm produces a lower peak discharge.

The operation of a detention basin is dependent on the interaction of the inflow hydrograph, storage characteristics of the basin and the performance of the outlet control structure. Therefore, a basin can respond differently to different duration storms.

The proper use of the Modified Rational Method requires the calculation of the volume of runoff for the critical storm duration (T_c), which is the duration that produces the greatest volume of storage and highest water surface elevation within a detention basin. Reservoir routing computations for the basin will need to incorporate several different duration storms in order to determine the critical duration and the highest water level for each frequency storm. Therefore, a trial and error approach is required to determine the critical storm duration. Computer programs with built-in routines for determining the critical storm duration are recommended when using this method.

4.4.5.3 SCS UNIT HYDROGRAPH METHOD

The SCS Unit Hydrograph Method developed by the NRCS requires the same basic data as the Rational Method: drainage area, a runoff factor, time of concentration and rainfall. The SCS approach is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception, depression storage and an infiltration rate that decreases during the course of a storm. With this method the direct runoff can be calculated for any storm, either real or synthetic, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in the SCS National Engineering Handbook, Part 630 Hydrology.

In hydrograph applications, runoff is often referred to as rainfall excess or effective rainfall defined as the amount by which rainfall exceeds the capacity of the land to infiltrate or otherwise retain water. The principal physical watershed characteristics affecting the rainfall-runoff relationship are land use, land treatment, soil types and land slope.

Land use is the watershed cover and it includes both agricultural and nonagricultural uses such as type of vegetation, water surfaces, roads, roofs, etc. Land treatment applies mainly to agricultural land use and includes mechanical practices such as contouring or terracing and management practices such as rotation of crops. Runoff curve numbers (CN) indicate the runoff potential of an area when the soil is not frozen. The higher the curve number, the higher the runoff potential.

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C and D). Soil type A has the highest infiltration and soil type D has the least amount of infiltration. Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected.

Runoff curve numbers vary with the antecedent soil moisture conditions defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall the more direct runoff there is from a given storm. A five day period is used as the minimum for estimating antecedent moisture conditions. These conditions also vary during a storm. Heavy rain falling on a dry soil can change the moisture condition from dry to average to wet during the storm period.

The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area or to locate storm water drainage facilities and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems should also be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the sub-drainage areas.

The SCS method is based on a 24-hour storm event, which has a Type II typical storm time distribution in West Virginia.

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. Data for land treatment measures (such as contouring and terracing) from experimental watersheds were included. The equation was developed mainly for small watersheds whose daily rainfall and properties are ordinarily available. It was developed from recorded storm data that included the total amount of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall.

Two types of hydrographs are used in the SCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from one-inch of direct runoff occurring over the watershed in a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless hydrograph is plotted in non-dimensional units of time versus time to peak, and discharge at any time versus peak discharge.

Characteristics of the dimensionless hydrograph vary with the size, shape and slope of the tributary drainage area. The most significant characteristics affecting the shape of the dimensionless hydrograph are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Lag (L) can be considered as a weighted time of concentration (T_c) and is dependent on the physical properties of the watershed such as area, length and slope. The SCS derived an empirical relationship between the lag and the time of concentration: $L=0.6\ T_c$. Steep slopes, compact shape and an efficient drainage network tend to make lag time short and peaks high. Flat slopes, elongated shape and an inefficient drainage network tend to make lag time long and peaks low.

4.4.6 WATERSHED MODELING

Although many streams have been gaged to provide a record of streamflow over time, most streams encountered in highway drainage do not have available streamflow information. Precipitation data, however, are relatively abundant and numerous models are available that allow the determination of runoff. Engineers must rely on synthesis and simulation as tools to generate synthetic flow sequences used for design discharge rates and decision making regarding the effects of land use and flood control measures. Simulation is defined as the mathematical description of a real system to imitate the behavior of the system. A hydrologic simulation model is a set of equations and algorithms that describe the response of a hydrologic system to a series of events during a selected time period. It is commonly used for generating streamflow hydrographs from rainfall data and watershed characteristic data.

The process of developing a model can be divided into three phases: identification, conceptualization, and implementation. The identification phase analyzes existing and proposed components of the system to be studied. It collects all pertinent data to be used in the analysis. Examples of information that may be necessary are subwatershed characteristics, channel characteristics, meteorological data, streamflow data, and reservoir/storage information.

The conceptualization phase identifies system components that are important to define the behavior of the system and it frequently provides feedback to the identification phase by defining actual data requirements. This phase chooses the techniques to be used to represent the system elements, and selects the simulation models that best provide these techniques.

In the implementation phase, the model is run and the results are reviewed and analyzed. The validity of the model is determined by demonstrating that the model results represent a reasonable estimate of the actual system behavior. If the model output is not deemed to be sufficiently valid, the input data or modeling technique is modified, and the model is rerun, until the model produces valid results.

4.4.6.1 Watershed Modelling System (WMS)

WMS is a comprehensive environment for hydrologic analysis. It was developed by the Environmental Modeling Research Laboratory of Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station and is currently owned and developed privately. Land use tables with corresponding SCS curve numbers vary across the United States. WMS supports a user-definable method for relating land use to the SCS curve number. This is done through a simple table file that is imported prior to computing the composite curve number. The following Runoff Curve Number table relates the values of the National Land Cover Database data to the SCS curve number of the TR-55 graphical peak discharge method. This table shall be used with WMS for watersheds within West Virginia.

Table 4-15

Runoff Curve Number for National Land Cover Database 2006

NLCD 2006 Cover Type Description	CN Hydrologic Soil Group				TR-55 Equivalent* Cover Type Description
	A	**B**	**C**	**D**	
Open Water	98	98	98	98	N/A
Perennial Ice/Snow	98	98	98	98	N/A
Developed Open Space	39	61	74	80	Open Area - Good Condition (grass cover >75%)
Developed Low Intensity	51	68	79	84	Residential District – 1 acre average lot size (20% average impervious area)
Developed Medium Intensity	54	70	80	85	Residential District – ½ acre average lot size (25% average impervious area)
Developed High Intensity	89	92	94	95	Urban District – Commercial & Business (85% average impervious area)
Barren Land	77	86	91	94	Developing Urban Areas
Deciduous Forest	30	55	70	77	Woods Only – Good Condition
Evergreen Forest	30	55	70	77	Woods Only – Good Condition
Mixed Forest	30	55	70	77	Woods Only – Good Condition
Shrub/Scrub	30	58	71	78	Meadow with continuous grass cover
Grassland/Herbaceous	30	58	71	78	Meadow with continuous grass cover
Pasture/Hay	39	61	74	78	Pasture – Good Condition (>75% cover)
Cultivated Crops	65	75	82	86	Farm Areas
Woody Wetlands	30	55	70	77	Woods Only – Good Condition
Emergent Herbaceous Wetlands	98	98	98	98	N/A

*Equivalents were selected from Table 4-9 and Table 4-10 of the WVDOH Drainage Manual

Note: Curve numbers may be adjusted to accommodate varying site conditions. Curve numbers were generated based on general criteria and help to serve as a guide if adjustments are necessary.

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WEST VIRGINIA

DEPARTMENT OF TRANSPORTATION

DIVISION OF HIGHWAYS

2007

DRAINAGE MANUAL

CHAPTER 5:
STORM DRAINAGE SYSTEMS

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

DRAINAGE COMPUTATION FORMS

LIST OF CHARTS

CHAPTER 5: STORM DRAINAGE SYSTEMS

5.1 INTRODUCTION

This chapter provides guidance on storm drainage design and analysis, which includes the following:

- System planning
- Pavement drainage
- Gutter flow calculations
- Inlet spacing
- Storm drain pipe sizing
- Hydraulic grade line calculations

The design of a drainage system must address the needs of the traveling public as well as those of the local community through which it passes. Drainage design in urban areas is more complex due to:

- wide roadway sections;
- flat grades in both longitudinal and transverse directions;
- shallow watercourses; and
- absence of roadside channels.

The most serious effects of an inadequate roadway drainage system are:

- damage to surrounding property resulting from water overflowing the roadway curb;
- erosion at pipe outlets;
- risk of accidents and traffic delays due to excessive spread or flow of water along the roadway; and
- weakening of the roadway base and sub-grade due to saturation from frequent ponding of long duration.

The primary goal of storm drainage design is to limit the amount of water on the traveled way so that the passage of traffic is not interrupted for the design storm. This is accomplished by:

- placing inlets at appropriate locations and at appropriate intervals to intercept flow and control the spread of water on the roadway;
- properly sizing a storm drain conduit to convey flow from the inlets to a suitable outfall location; and
- providing an outfall with acceptable backwater conditions.

5.2 DESIGN POLICY AND CRITERIA

5.2.1 GENERAL POLICY

Storm drainage facilities consisting of inlets, curbs, gutters, and storm drains shall be designed in accordance with the policy, criteria, and guidelines provided in this chapter.

Highway storm drainage facilities shall be designed to collect storm water runoff and convey it in a manner that adequately drains the roadway and minimizes the potential for adverse effects to adjacent properties. The placement and hydraulic capacities of storm drainage facilities shall consider possible damage to adjacent property, the design traffic service requirements, and available funding.

5.2.2 HYDROLOGY

The Rational Method is recommended for the design of storm drainage systems with drainage areas of 200 acres or less. The minimum time of concentration is 5 minutes. Large or complex storm drainage systems involving detention storage, or pumping stations shall require the development of a runoff hydrograph. The Rational Method and hydrograph methods are described in Chapter 4.

5.2.3 DESIGN FREQUENCY

Pavement drainage and inlet spacing shall be designed for a 10-year frequency. The design frequency for storm drains shall be designed for a 10-year frequency. When a storm drain drains a sag location or depressed area of the roadway that may be subject to ponding, a 50-year frequency shall be used. It is important to note that the storm sewer outlet pipe from an inlet in a sag location or depressed area should also be designed for a 50-year frequency.

5.2.4 PAVEMENT GEOMETRY

Longitudinal slope and gutter grades in curbed pavements shall not be less than 0.5 percent. The minimum longitudinal slope in sag vertical curves shall not be less than 0.3 percent within 50 feet of the level point in the curve. The minimum longitudinal slope in extremely flat crest vertical curves shall not be less than 0.3 percent.

The minimum longitudinal slope for bridge deck drainage shall be 0.5 percent. Zero gradients, sag vertical curves, and superelevation transitions with flat pavement sections shall be avoided on bridges. The coincidental occurrence of superelevation transitions and sag points or zero grades shall be avoided on pavements.

The minimum pavement cross-slope shall not be less than 2 percent except during the occurrence of a superelevation transition. Maximum allowable cross slope on superelevated sections shall be 8%.

5.2.5 INLETS

Inlet efficiency decreases as spread increases; therefore, inlet capacities shall be checked to ensure efficient inlet performance.

Inlets shall be spaced along a curb, median barrier, or bridge parapets on continuous grades to pick up flow as it accumulates. Curbs and inlets shall be used where runoff from the pavement could erode highway fill slopes and/or to reduce the right-of-way needed for shoulders and roadside ditches.

In addition to inlets spaced along a curb, the following locations shall require inlets:

- Immediately upstream of median breaks, crosswalks, street intersections, and bridges.
- Immediately downstream of a bridge to prevent water from the bridge deck from eroding the roadway shoulder.
- Immediately before the pavement cross slope reverses in a transition to prevent accumulated flow from crossing the pavement.
- In pavements with sag points in the gutter grade.
- Behind curbs, shoulders, or sidewalks to drain low areas
- At concentrated flow interception points for offsite sources.

Depressions of grate and curb inlets shall be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them.

Grate inlets shall also be bicycle safe when used on roadways that permit bicycle traffic.

5.2.6 STORM DRAINS

Storm drain systems shall have adequate capacity to accommodate the design runoff that enters the system. The minimum size of a storm drain conduit shall not be less than 12 inches in diameter or its equivalent for non-circular shapes.

Where feasible, storm drains shall be designed to avoid existing utilities that are to remain in place. The minimum horizontal clearance shall be at least equal to the storm drain pipe diameter when the storm drain system is parallel to an underground waterline. When the storm drain system crosses a waterline, the minimum clearance shall be at least equal to the utility pipe or utility casing diameter or 1 foot, whichever is greater. Coordination with utility companies is required.

Storm drains shall be designed to prevent sedimentation in the conduit. Watertight joints shall be used if velocities are high, or if the pipe is placed under the ground water table. Buoyancy issues must also be evaluated.

Pipe size shall not decrease in the downstream direction, regardless of the available pipe gradient. Storm drain outfalls shall be designed to ensure that the potential for erosion is minimized.

5.2.7 ACCESS STRUCTURES

The maximum spacing of storm drain access structures, whether manholes or inlets, shall not be more than 400 feet for 12 inch through 54 inch diameter storm drains and shall not be more than 800 feet for storm drains 60 inches and larger in diameter. Minimum manhole diameter shall be determined by the maximum pipe size and the deflection angle of the pipes, but in no case should the diameter be less than 4 feet.

5.2.8 HYDRAULIC GRADE LINE

The hydraulic grade line shall be checked for all storm drain systems. The storm drain shall be designed such that the hydraulic grade line does not exceed any critical elevation for the design storm. A critical elevation is defined as the point above which there would be unacceptable flooding of the traveled way or adjoining property. This includes the tops of manholes, junctions, and inlets. Because the inlet design is predicated on free-fall conditions, the hydraulic grade line for a particular design frequency shall not exceed an elevation that interferes with the proper functioning of the inlet (see Section 5.3.6.4).

5.2.9 OUTLET PROTECTION

Rock channel protection at the outlet of storm drain pipes is essential to the protection of the downstream channel. It protects the channel from erosion due to scour from high velocity flows. This is accomplished by reducing the velocity of the flow to an acceptable level before it enters the downstream channel. For these structures to work properly, it is essential that these rocks remain stable and have enough resistance to shear forces to handle peak flows during large rain events.

5.3 DESIGN GUIDELINES

5.3.1 SYSTEM PLANNING

System planning is critical prior to commencing the design of a storm drainage system. The design of any storm drainage system should involve the accumulation of basic data, familiarity with the project site, a basic understanding of hydrologic and hydraulic principles, and the applicable drainage policies and design criteria.

5.3.1.1 DESIGN APPROACH

The typical steps of the design process for a storm drainage system are listed below:

- Collect data
  - Coordinate with other agencies
  - Conduct site survey, including the existing drainage system, if present
  - Evaluate existing drainage system for comparison of effects of proposed improvements
  - Determine and document effects of offsite drainage on the project
- Draw preliminary layout
  - Locate inlets and determine spacing
- Prepare the plan layout of the proposed storm drainage system:
  - Locate pipe outfall
  - Layout the existing drainage system, if present
  - Determine the direction of flow
  - Locate existing utilities
  - Locate connecting mains
- Locate manholes
- Size pipes
- Calculate the hydraulic grade line
- Prepare plans
- Provide documentation

5.3.1.2 DATA COLLECTION

The designer should become familiar with the land use patterns, the nature of the physical development, the area to be served by the storm drainage system, local storm water management plans, and the ultimate pattern of drainage (both overland and by storm drains) to some existing outfall location. Furthermore, the designer should understand the nature of the outfall location since it usually has a significant influence on the storm drainage system. Environmentally sensitive areas must comply with applicable water quality rules and regulations.

One or more field reviews and a topographic survey are generally required before beginning the design of a storm drainage system. Photogrammetric mapping has become an important method of obtaining the data required for drainage design, particularly for busy urban roadways with extensive urban development. Research should be conducted to obtain existing topographic maps from sources such as the United States Geological Survey (USGS), Natural Resource Conservation Service (NRCS), municipalities, county governments, or private developers.

Developers and governmental planning agencies should be consulted regarding drainage and development plans for the area in question. The physical characteristics of rapidly growing urban areas can change drastically in a short time. For such areas, the designer should consider future development in the storm drainage design. Comprehensive stormwater management plans, land use zoning, and floodplain ordinances should be reviewed, if available.

5.3.1.3 PRELIMINARY LAYOUT

Preliminary layout sketches or schematic diagrams showing the basic components of the system can be useful to the designer. Unless the proposed system is very simple and small, the preliminary layout should not be ignored. Such sketches should show:

- watershed areas and land use;
- existing drainage patterns and problems;
- plan and profile of the roadway;
- street and driveway layout with respect to the project roadway;
- underground utility locations and elevations;
- locations of proposed retaining walls, bridge abutments and piers;
- logical inlet and access hole locations;
- preliminary lateral and trunk line layouts; and

- clear definition of the outfall location and characteristics.

The preliminary layout sketches should be included in the final design documentation.

The preliminary layout should be reviewed for conflicts in conjunction with the traffic staging plans and the soils report recommendations. This process saves time and effort and enables the designer to proceed with detailed storm drainage design calculations, adjustments, and refinements.

5.3.1.4 SPECIAL CONSIDERATIONS

Special attention should be directed toward avoiding utilities and deep cuts. Some cases may require maintenance of traffic, including temporary bypasses and temporary drainage during the construction phase. Special consideration should also be given to the actual trunk line layout and its constructability with regard to the maintenance of the traffic plan. Some sites may require a trunk line on both sides of the roadway with very few laterals while others might require a single trunk line. Such requirements are usually a function of the project cost, but may also be controlled by other physical features.

5.3.2 ROADWAY FEATURES

The roadway right-of-way can generally be divided into the roadway pavement and the roadside. The roadway pavement generally consists of the travel lanes, shoulders, parking lanes, bicycle lanes, the median area, and the curb. The roadside generally consists of pedestrian sidewalks, and drainage ditches.

Roadway features that should be considered for pavement drainage calculations include:

- Longitudinal Slope
- Cross Slope
- Curb and Gutter Sections
- Medians and Median Barriers
- Roadside and Median Ditches
- Bridge Decks
- Hydroplaning
- Debris
- Pavement Texture

- Pavement Structure
- Underdrains

The pavement width, cross slope, and profile control the time it takes for the stormwater to drain to the gutter section. The gutter cross-section and longitudinal slope control the quantity of flow that can be carried in the gutter section.

5.3.2.1 Longitudinal Slope

The minimum longitudinal slope should be maintained to facilitate good pavement drainage. This is more important for curbed pavements than for uncurbed pavements because curbed sections are susceptible to the spread of stormwater against the curb. Flat gradients in curbed sections can lead to ponded conditions and poor drainage. Flat gradients on uncurbed pavements can also lead to the ponding if vegetation is allowed to build up along the pavement edge. Minimum grades can be maintained in very flat terrain by using a rolling profile. The minimum longitudinal slope of 0.3 percent within 50 feet on either side of the level point in sag vertical curves can be checked by verifying that the length of the curve divided by the algebraic difference in grades is equal to or less than 167.

5.3.2.2 Cross Slope

This section deals mainly with highway pavements with normal crown sections. Crowned sections require drainage design for both sides of the highway, which may complicate traversing at-grade intersections due to several abrupt changes in the cross section. Flatter cross slopes can improve the ability to traverse highway intersections and are better suited for divided highways with wide depressed medians and full or partial access control.

Proper cross slope is very important in providing adequate drainage by removing water from the pavement. Pavement cross slope will vary with the road surface material. Drainage of superelevated pavements requires careful consideration.

A uniform slope toward the outside edge of the pavement can improve the ability to traverse at-grade intersections. This requires drainage design for one side of each roadway. But with each lane contributing to runoff, the potential for hydroplaning is increased. Freeze-thaw periods can also create a safety problem when snow plowed to the median melts and drains across the travel lanes.

On multi-lane pavements with normal crown sections when three or more lanes are inclined in the same direction, it is desirable that each successive pair of lanes have an increased cross-slope. The two lanes adjacent to the crown line should be pitched at the normal slope and successive lane pairs should be pitched at an increased slope of about 0.5 to 1%. Cross slope breaks should preferably be at two lanes, with three lanes as the upper limit.

The hazard potential due to hydroplaning should be considered in superelevated curve transitions and areas with flat profile grades and wide pavement sections. The design should be carefully checked to minimize the number and length of flat pavement sections in cross-slope transition areas. Consideration should be given to increasing cross-slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades.

5.3.2.3 CURB AND GUTTER

The use of a curb and gutter is normal practice for low-speed urban highways. Curbs on the outside edge of pavements contain and control surface runoff within the roadway section and direct it away from adjacent properties.

The curb and gutter form a triangular channel that can efficiently convey runoff of a lesser magnitude without interrupting the flow of traffic. Flow in gutters is governed by the principles of open channel flow. When a design storm flow occurs, the water can spread across the gutter width, shoulders, parking lanes and portions of the travel lanes. The gutter may have a uniform cross-slope with the same slope as the adjacent pavement, or a composite cross-slope with a steeper slope than the adjacent pavement. The total width of the flow spread is controlled by the proper placement of inlets.

It is desirable to intercept runoff from cut slopes and other areas draining toward the roadway before it reaches the curbed section, thereby minimizing sediment and other debris on the roadway and reducing the amount of water that must be carried in the gutter section.

5.3.2.4 MEDIANS AND MEDIAN BARRIERS

Medians are commonly used to separate opposing lanes of traffic on divided highways. Where median barriers are used, it is necessary to provide inlets and connecting storm drains to collect water that accumulates against the barrier. Slotted drains adjacent to the median barrier can also be used for this purpose.

5.3.2.5 ROADSIDE AND MEDIAN DITCHES

Roadside ditches or swales may be used where curbs are not needed for traffic control. The advantages include: a lesser hazard to traffic than a near-vertical curb and hydraulic capacity that is not dependent on the pavement spread. Ditches can be used in cut sections, depressed sections and other locations where sufficient right-of-way is available and driveways or intersections are infrequent.

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

5.3.2.6 BRIDGE DECKS

Bridge deck drainage is similar to roadway sections with curbs. Deck drainage is often less efficient due to flatter cross slopes and smaller drainage inlets (scuppers) that have a higher potential for clogging from debris. It is difficult to provide and maintain adequate deck drainage systems; therefore, gutter flow from the roadway should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried over several spans to the end of the bridge for disposal.

The water spread should be checked to ensure compliance with the design spread criteria. Many bridges may not require any drainage structures at all. To determine the permitted length of deck without drainage structures and without exceeding the allowable spread, the following equation may be utilized.

$$L = \frac{24400\left(S_x^{\,1.67}\right)\left(S^{0.5}\right)\left(T^{2.67}\right)}{CnIW}$$

This equation is based on a uniform cross slope (FHWA Report No. RD-79-31, 1979).

Where: L = length of deck, ft

S_x = cross slope, ft/ft

S = longitudinal slope along the toe of the parapet, ft/ft

T = allowable spread, ft

C = runoff coefficient

n = Manning's Roughness Coefficient

I = rainfall intensity, in/hr

W = width of drained deck, ft

Bridge decks can also be drained with scuppers, which are vertical holes through the deck. Scuppers have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Scupper outlets should not be located over driving lanes, embankments, near pier foundations, navigation channels, railroad tracks, or other sensitive locations. More detailed guidance on the design of bridge deck drainage can be found in HEC-21, Design of Bridge Deck Drainage.

5.3.2.7 HYDROPLANING

Hydroplaning occurs when water causes vehicle tires to lose contact with the pavement surface. The National Transportation Safety Board reports that about 13% of fatal accidents and 25% of all accidents occur on wet pavements. The Urban Drainage Design Manual (HEC-22) published by the Federal Highway Administration should be consulted for a more detailed discussion on hydroplaning. The National Cooperative Highway Research Program (NCHRP) Research Project 1-29, “Improved Surface Drainage of Pavements,” suggests that hydroplaning conditions can develop for relatively low vehicle speeds and at low rainfall intensities for storms that frequently occur each year. Analysis methods developed through this research effort provide guidance in identifying potential hydroplaning conditions. Wide pavement sections are especially prone to hydroplaning conditions during high intensity rainfall. Some of the primary factors controlling hydroplaning are:

- Vehicle speed
- Tire conditions (pressure and tire tread)
- Pavement micro and macro texture
- Roadway geometries (pavement width, cross slope, grade)
- Pavement conditions (rutting, depressions, roughness)

Since speed is an important factor that affects hydroplaning, it is the responsibility of the driver to exercise caution while driving during wet conditions. Designers do not have control over all of the factors involved in hydroplaning. However, many remedial measures can reduce hydroplaning potential. The following guidance should be considered in accordance with the AASHTO Policy on Geometric Design of Highways:

- Maximize transverse slope on pavement
- Maximize pavement roughness

- Limit spread on the traveled way (inlet spacing)
- Maximize interception of gutter flow above superelevation transitions
- Limit duration and depth of ponded water in sag locations
- Limit depth and duration of overtopping flow

5.3.2.8 DEBRIS

The potential for debris should be considered when selecting the type of inlet. A grate inlet generally traps debris before it can enter the adjoining pipe. The use of a curb inlet will usually allow the debris to flow into the inlet and be discharged through the pipe outfall unless the debris is large enough to become lodged in the pipe and restrict the flow. Due to the possibility of clogging, the storm drainage system must be made accessible for cleanout of debris.

5.3.2.9 PAVEMENT TEXTURE

Pavement texture is an important consideration for roadway surface drainage. Although the hydraulic engineer will have little control over its selection, it should be recognized that the pavement texture has an impact on the build-up of water depth on the pavement during rainstorms. A good macro texture provides a channel for the water to escape from the tire-pavement interface and thus reduces the potential for hydroplaning.

Macro texture in a concrete pavement can be achieved by longitudinal and transverse grooving. Combinations of longitudinal and transverse grooving provide the most adequate drainage for high-speed conditions.

5.3.2.10 PAVEMENT STRUCTURE

Drainage aspects in the design of the pavement structure are important for areas with high groundwater tables where excess moisture can attack the pavement sub-grade. Pavement structure drainage is generally accomplished by using a free draining base course, which usually includes an underdrain system connected to a free draining base trench. The free draining base course shall be in accordance with the furnished pavement design for the project.

5.3.2.11 UNDERDRAINS

In soils where groundwater is a problem, a system of underdrains should be used to remove excess moisture from the pavement structure. Underdrains typically consist of a network of perforated pipes, French Drains or collector fields. The available types include: pipe with granular-filled trench and filter-fabric-wrapped granular material in a trench. Details for size and installation are provided in the Standard

Details Book, Volume I, Sheets 3 and 4 of DR8. The following items address considerations for placement:

- They should be placed transversely on the downgrade end of cuts approaching a major fill in order to intercept water that might saturate the fill, subsequently causing pavement failure (see the grading transition detail in design directive 405). Outlet spacing shall not exceed 250 feet in fill sections. Outlets in cut sections shall be connected to the nearest drainage structure.

- Longitudinal underdrains should be specified where the rock strata is dipping in such a manner that flow would saturate and damage the base course, or in special cases where the water table after would create a pressure head under the base course construction.

- They should be specified at known locations of constant sources of water, such as wells and springs, and in sag vertical curves where water in the subgrade is expected to accumulate and flow to the surface.

- In urban locations with curb and no side ditches, underdrains should be used to drain the base course.

- Consideration should be given to placing them in existing channels located under embankments. For these installations, the designer should specify that they are installed to conform to the existing channel alignment. The use of select rock fill obtained from unclassified excavation should always be considered in lieu of underdrains.

The following note shall be on the plans when there is reason to believe that ground water will be encountered during construction which requires indeterminate amounts of underdrain installation: “The quantities of underdrain have been increased for control of ground water that could be detrimental to the completed facility. This quantity shall be used as directed by the Engineer.”

The elevations of the proposed storm drainage system must account for the depth of underdrain and underdrain outfalls. The outfalls may go directly into an inlet, to a culvert, ditch, or channel with the use of a slope-wall.

5.3.3 DESIGN SPREAD

The design spread governs the amount of water that can be allowed in the curb and gutter section and on the adjacent roadway. The designer calculates the spread and when the allowable spread is reached, an inlet is proposed to intercept all or a portion of the flow.

For roadways designed for speeds of 40 mph or greater, spread of the flow on a bridge deck or curbed section of pavement is generally limited to the shoulder width. If the design speed is less than 40 mph, spread is generally limited to the shoulder width plus 3 feet into the traveled way. If a parking lane is present, then the spread will be limited to 8 feet.

Table 5-1

Allowable Spread for the Design Speed

Design Speed	Allowable Spread	
	10-year Design Storm	50-year Check Storm for Sag Vertical Curves
less than 40 mph	shoulder + 3 feet	one lane open to traffic
40 mph or greater	shoulder	

Use of a gutter slope of 0.06 is recommended for most projects because of the decrease in total spread and the increased flow efficiency resulting from carrying a larger amount of water near the curb. Most inlets are more efficient in intercepting such concentrated flows rather than flows that are spread over a wide, flat cross-slope.

Gutter flow calculations are necessary in order to relate the quantity of flow (Q) in the gutter to the total spread of water on the pavement. The following equation may be used to calculate the spread T, in a uniform cross slope gutter section:

$$T = 1.243(Qn)^{3/8} S_x^{-5/8} S^{-3/16}$$

Where T = Spread, ft

Q = Gutter flow rate, ft^3/s

n = Manning's roughness coefficient

S_x = Pavement cross slope, ft/ft

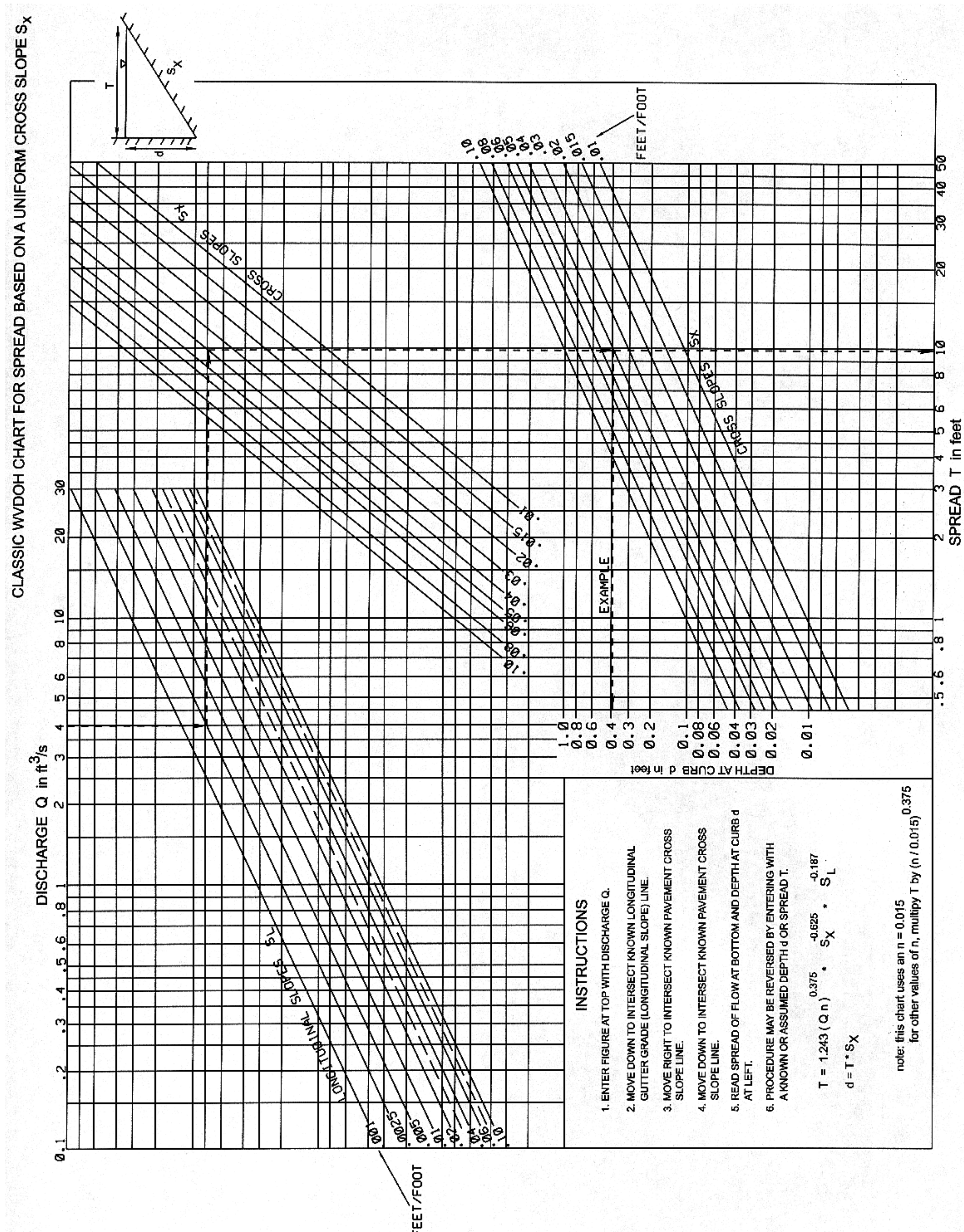
S = Longitudinal (gutter) slope, ft/ft

Chart 5-1 and Chart 5-2 can be utilized to solve for spread for the uniform cross slope gutter sections and V shape gutter sections.

Calculation of spread in sag conditions requires special attention. Grate inlets in sag conditions operate as a weir to a depth of about 0.4 feet above the grate. When the depth exceeds 1.4 feet above the grate, the inlet operates as an orifice. Between these two depths, flow is in a transitional state. Curb opening inlets operate as a weir at depths up to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transitional state. See Section 5.3.4.10 for design of flanking inlets.

Chart 5-1

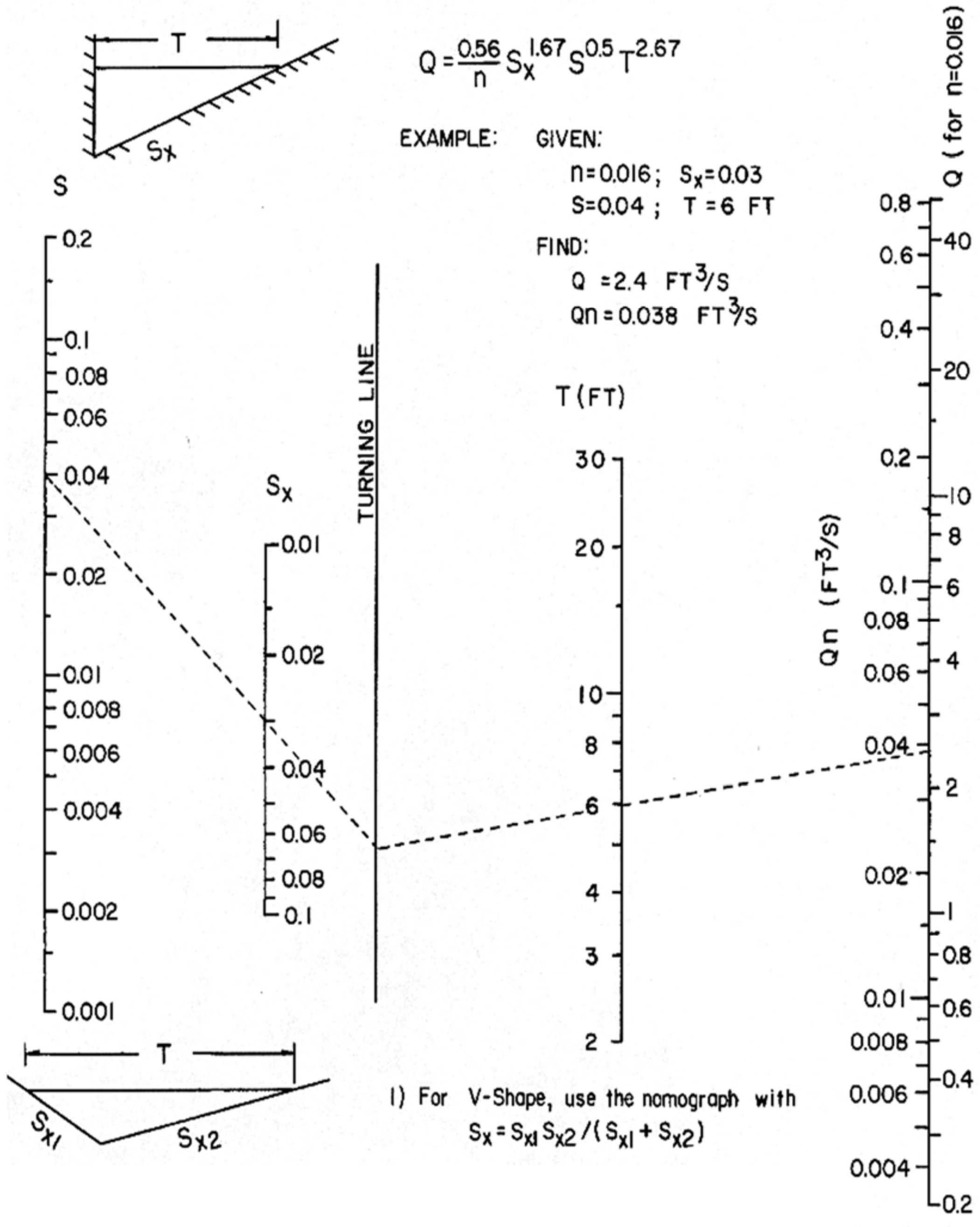
Spread and Depth Based On A Uniform Cross Slope



Source: WVDOH 1984 Drainage Manual

Chart 5-2

Spread and Discharge Based on a Uniform Cross Slope



Source: Federal Highway Administration Hydraulic Engineering Circular-22

5.3.4 DRAINAGE INLETS

Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts.

In general, inlet type, location, and spacing should consider the following factors:

- Inlet capacity, width of gutter spread (i.e., spread of flow), and depth of flow at the curb line
- Movement of vehicles to and from adjacent property on turnouts
- Pedestrian and bicycle safety
- Maximum pipe length without maintenance access
- Roadway geometry
- Hydraulic efficiency of the system
- Ability to be self-cleaning
- Potential for flooding and erosion of/from off-site property

Ditches, swales, or other collection systems beyond the curb shall be considered to intercept runoff from cut-slopes and areas outside the right of way. These interceptor and collection systems will reduce the amount of water that has to be picked up by inlets and will help prevent mud and debris from being carried onto the pavement, particularly at the high side of superelevated curves. Inlets or other collection systems shall be used to intercept runoff from side streets before it reaches the roadway.

Details for grate, curb opening, combination, and slotted drain inlets are provided in the Standard Details Book, Volume I, Sheets DR6-A through DR6-X. Details for concrete box and grate inlets are provided in the Typical Sections and Related Details Book, dated 2000, Sheet 81.

5.3.4.1 INLET TYPES

Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades where clogging with debris is not a problem. The use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. Special design (oversized) grate inlets or combination inlets should be utilized at major sag points if sufficient capacity is provided for clogging. Grate inlets should be bicycle safe unless located on highways where bicycles are not permitted. Grates should also be structurally designed to handle the appropriate wheel loads when subject to traffic. WVDOH standard grate inlets include:

- Type A, two 24” X 24” 30˚ tilt bar grates, parallel to the curb
- Type B, one 24” X 24” 30˚ tilt bar grate against the curb
- Type C, two 24” X 24” 30˚ tilt bar grates, perpendicular to the curb
- Type G, grate size ranging from 32” X 38” to 60” X 66”. The type 1 grate has 1” bars, 2” on center. The type 2 grate has 1” bars, 4” on center. This grate inlet is primarily used in median and roadside ditches. Its design use is addressed in Chapter 6 of this manual.

Table 5-2

Grate Inlet Weir and Orifice Flow Details

Inlet Type	**Effective Weir Perimeter**	**Clear Open Area Against Curb**	
A	21 ¾ “+42”+21 ¾” = 85.5” = 7.1’	2.7 ft^2	
B	21 ¾”+19”+21 ¾” = 62 ½” = 5.2’	1.4 ft^2	
C	45 ½”+19”+45 ½” = 110” = 9.2’	2.7 ft^2	
		Clear Open Area for All Sides	
		Type 1 Grate	**Type 2 Grate**
32X 38 G	31 ¾”+37 ¾”+31 ¾”+37 ¾” = 139” = 11.6’	4.4 ft^2	6.4 ft^2
36 X 42 G	35 ¾”+41 ¾”+35 ¾”+41 ¾” = 155” = 12.9’	6.5 ft^2	7.9 ft^2
42 X 48 G	41 ¾”+47 ¾”+41 ¾”+47 ¾” = 179” = 14.9’	8.8 ft^2	10.1 ft^2
45 X 51 G	44 ¾”+50 ¾”+44 ¾”+50 ¾” = 191” = 15.9’	9.3 ft^2	11.8 ft^2
48 X 54 G	47 ¾”+53 ¾”+47 ¾”+53 ¾” = 203” = 16.9’	10.6 ft^2	13.6 ft^2
54 X 60 G	53 ¾”+59 ¾”+53 ¾”+59 ¾” = 227” = 18.9’	13.5 ft^2	16.3 ft^2
60 X 66 G	59 ¾”+65 ¾”+59 ¾”+65 ¾” = 251” = 20.9’	15.7 ft^2	20.7 ft^2

Curb-Opening Inlets

These inlets are vertical openings in the curb covered by a top slab. They function as weirs with flow entering from the side and are best suited for use on flatter slopes and at sag points since they can convey large quantities of water with debris. They are a viable alternative to grates on continuous grades where the grates would be hazardous for pedestrians or bicyclists. Curb-opening inlets are generally not recommended for use on steep continuous grades due to significant bypass of flow. WVDOH standard curb-opening inlets include:

- Type D, a 4’ wide, 4” high curb opening depressed 2” below adjacent pavement. The lateral width of the depression is 1’. This inlet has a 45˚ inclined throat.
- Type E, widths of 6’, 8’, 10’, 12’, 14’, 16’, 18’, and 20’ long, 4” high curb opening depressed 2” below adjacent pavement. The lateral width of the depression is 1’. This inlet also has a 45˚ inclined throat.

The depression acts as superelevation for the flow. It helps to turn the water into the inlet opening.

Combination Inlets

There are various types of combination inlets with the curb opening and grate combination being the most common. Two types of the curb and grate combination are inlets with the curb opening adjacent to the grate and inlets with the curb opening upstream and adjacent to the grate. The latter are referred to as “sweeper” inlets. Slotted inlets are also used in combination with grates, located either longitudinally upstream of the grate or transversely adjacent to the grate. The interception capacity of the curb opening and grate combination on a continuous grade is no greater than that of the grate alone. The curb opening provides some capacity in the event of the clogging of the grate, but it does not significantly help in reducing the spread of flow.

The efficiency of inlets in passing debris is critical in sag locations because all runoff which enters the sag must be passed through the inlet. Total or partial clogging of inlets in these locations can result in hazardous ponding conditions. Combination inlets are recommended for use in these locations because of the tendencies of grates to become clogged. WVDOH standard combination inlets include:

- Type F, a 4’ wide, 4” high curb opening depressed 2” below adjacent pavement with two 24” X 24” 30˚ tilt bar grates, parallel to the curb. This inlet has a vertical throat.
- Type H, a 2’ wide, 4” high curb opening with no depression below the adjacent pavement. It has one 24” X 24” 30˚ tilt bar grate and a vertical throat.

Table 5-3

Combination Inlet Weir and Orifice Flow Details

Inlet Type	Effective Weir Perimeter	Clear Open Area Against Curb
F	21 ¾ "+42"+21 ¾" = 85.5" = 7.1'	2.7 ft^2
H	21 ¾"+19"+21 ¾" = 62 ½" = 5.2'	1.4 ft^2

Slotted Drain Inlets

These inlets consist of a slotted opening with bars perpendicular to the opening. They can be used to intercept sheet flow, to collect gutter flow (with or without curbs), to modify existing systems to accommodate roadway widening or increased runoff and to reduce ponding depth and spread at grate inlet locations. FHWA tests of slotted inlets indicate that the length of inlet required for total interception is applicable to both curb-opening inlets and slotted inlets. Therefore, the capacities and efficiencies of the two are calculated in the same fashion. The two types of slotted inlets in general use are the vertical riser type and the vane type. The WVDOH standard slotted inlet has two types of slot design. Type 1 consists of a slot width of 1-3/4" that may be flared to 3" at the bottom and a slot height of 2.5", 6" or 8.5". Type 2 consists of a 2" wide slot with a minimum slot height of 4".

Concrete Box & Grate Inlets

These inlets consist of a formed or pre-cast concrete box trench covered by a grate that is bolted to a frame support anchored to the trench. These inlets are typically placed perpendicular to the flow with the invert of the trench having a 1% slope. They are used to intercept sheet flow from side streets or driveways before the flow encounters the primary roadway. The WVDOH standard concrete box & grate inlet has three types of grates available in 2' long sections. The length refers to the direction perpendicular to flow; therefore installations shall be in increments of 2' across the side street or driveway. Type A grates have 1 1/8" long by 2 1/2" wide holes spaced 1 inch apart along the length of the grate. The long axes of the holes are parallel to the direction of flow. Type C grates have 4 7/8" long by 7/8" wide holes spaced 7/8" apart along the length of the grate. The long axes of the holes are perpendicular to the direction of flow. Type P grates have 1/4" long by 2 5/16" wide holes spaced 1" apart along the length of the grate. The long axes of the holes are parallel to the direction of flow. Type P grates are meant for areas with heavy pedestrian traffic. Type A grates are available in widths from 8 to 51 inches wide. Type C grates are available in widths from 8 to 45 inches wide. A Type P grate is available in widths from 8 to 20 inches wide.

It is important to note that there are advantages and disadvantages to the use of each type of inlet in a design situation. These should be examined against the physical surroundings for frequency of maintenance and cease function.

5.3.4.2 INLET SPACING

The spacing of inlets on a continuous grade is based upon the spread of flow. The spread of flow increases as runoff is contributed to the gutter section in the downstream direction. An inlet is located where the spread of flow in the gutter reaches the allowable design spread. The factors that determine the inlet spacing include: tributary drainage area, longitudinal slope of the gutter, the gutter/roadway cross section geometry, the capacity of the inlet grate and the effect debris has on that capacity.

For a continuous grade, the designer may establish uniform spacing between inlets of a given design if the contributing drainage area to each inlet has reasonably uniform runoff characteristics and is rectangular in shape. In this case, the drainage areas generally consist of pavement only and the time of concentration is assumed to be the same for each of the inlets.

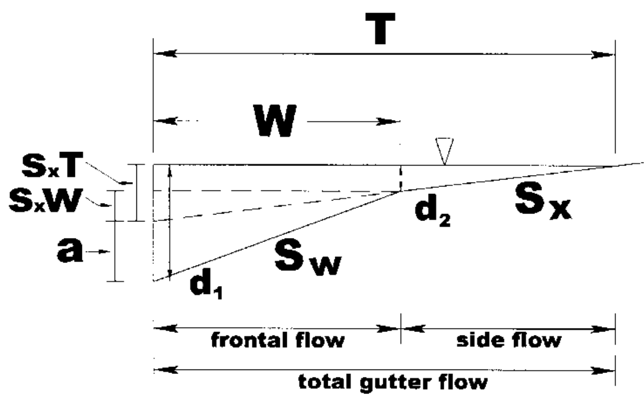
With the variability in the allowable design spread of flow (based on design speed, see Section 5.3.3) the process for the spacing of inlets begins with the gutter/roadway cross section geometry. Conveyance “K” describes the geometric carrying capacity of a hydraulic conduit and is defined as the flow divided by the square root of the longitudinal slope. In this case, the hydraulic conduit is the gutter section. Using the integrated form of the Manning’s equation, conveyance is a function of the flow depth, roughness and the cross slope of the gutter section (see Figure 5-1). For composite gutter sections the conveyance is calculated separately for flow in the gutter (frontal flow) and flow outside of the gutter (side flow). The total conveyance is equal to the sum of the parts.

The spread of flow “T” is dependent upon the flow depth; therefore, conveyance can be put in terms of spread. By defining an allowable spread, the flow that causes the spread can be determined. For drainage areas that consist of pavement only, the width of the area is known from the roadway cross section geometry and the length of the area can be determined from the flow that causes the allowable spread. Length is equal to area divided by width. The area that causes the runoff that creates the allowable spread on the pavement at the inlet location creates the link between spread and length of flow. This length would be the maximum allowable spacing between inlets. Interception capacity of the inlet opening and offsite drainage areas that contribute flow to the gutter must also be considered when determining this length of spacing.

Flow area is used to determine an average flow velocity in the gutter, which is important for calculating grate inlet capacity on grade. Flow depth at the curb is important for determining grate, curb and combination inlet capacities in a sag location.

Figure 5-1

Gutter flow geometry



flow depth at the curb

$$d_1 = a + S_x T$$

flow depth at the break in slope

$$d_2 = (T - W) S_x$$

conveyance in the gutter

$$\frac{Q_w}{\sqrt{S}} = \frac{0.56\left(d_1^{2.67} - d_2^{2.67}\right)}{n S_w}$$

conveyance outside of the gutter

$$\frac{Q_s}{\sqrt{S}} = \frac{0.56\, d_2^{2.67}}{n S_x}$$

total conveyance

$$\frac{Q}{\sqrt{S}} = \frac{0.56\left(d_1^{2.67} - d_2^{2.67}\right)}{n S_w} + \frac{0.56\, d_2^{2.67}}{n S_x}$$

flow area

$$A = \frac{1}{2} T^2 S_x + \frac{1}{2} a W$$

gutter depression

$$a = S_w W - S_x W$$

$$d_2 = d_1 - S_w W$$

5.3.4.3 INLET CAPACITY

Inlet interception capacity is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet is the percent of total flow that the inlet will intercept for those conditions. Efficiency of an inlet differs with changes in cross slope, longitudinal slope, total gutter flow and pavement roughness. Flow that is not intercepted by an inlet is termed carryover or bypass. The interception capacity of an inlet increases with increasing flow rate, but the efficiency generally decreases

with an increasing flow rate. Factors that affect the gutter flow geometry also affect inlet interception capacity.

The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors along with the total flow in the gutter/roadway cross section. For overland flow interception capacity, as is the case with concrete box and grate inlets, depth of flow and velocity of flow are the controlling factors.

Interception capacity of a curb-opening inlet is largely dependent on flow depth at the curb and the opening length. Flow depth at the curb and the interception capacity and efficiency is increased by the use of a local gutter depression at the curb opening.

Slotted inlets function in essentially the same manner as curb opening inlets on grade. Both take in flow that passes along the side or parallel to the inlet. Interception capacity is also dependent on the flow depth at the curb and the inlet length. For overland flow interception capacity, depth of flow and velocity of flow are the controlling factors.

5.3.4.4 GRATE AND COMBINATION INLET CAPACITY ON GRADE

Grates are effective highway pavement drainage inlets where clogging with debris is not a problem. Where clogging may be a problem combination inlets should be used. When the velocity approaching the grate is less than the "splash-over" velocity, the grate will intercept essentially all of the frontal flow. Conversely, when the gutter flow velocity exceeds the "splash-over" velocity for the grate, only part of the flow will be intercepted. A part of the flow along the side of the grate will be intercepted, dependent on the cross slope of the pavement, the length of the grate and the flow velocity. The ratio of frontal flow to total gutter flow (E_o) for a uniform cross slope is expressed by:

$$E_o = \frac{Q_W}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$

Where: W = width of the grate, ft

T = flow spread, ft

The ratio of frontal flow to total gutter flow (E_o) for a composite cross slope is expressed by:

$$E_o = \frac{1}{1 + \dfrac{\dfrac{S_W}{S_X}}{\left[1 + \dfrac{\dfrac{S_W}{S_X}}{\dfrac{T}{W} - 1}\right]^{2.67} - 1}}$$

Where: S_W = depressed gutter slope, ft/ft

S_X = cross slope outside of the depressed gutter, ft/ft

It is important to note that the frontal flow to total gutter flow ratio (E_o) for composite gutter sections assumes by definition a frontal flow width equal to the depressed gutter section width. The use of this ratio when determining a grate's efficiency requires that the grate width be equal to the width of the depressed gutter section (W, see Figure 5-1). If a grate having a width less than W (in other words, the gutter width is more than the grate width) is specified, E_o must be modified to accurately evaluate the grate's efficiency. Since an average velocity has been assumed for the entire width of total gutter flow, the grate's frontal flow ratio, E_o' can be calculated by multiplying E_o by a flow area ratio. The area ratio is defined as the depressed gutter flow area in a width equal to the grate width divided by the flow area in the depressed gutter section (see Figure 5-2). This adjusted frontal flow area ratio is:

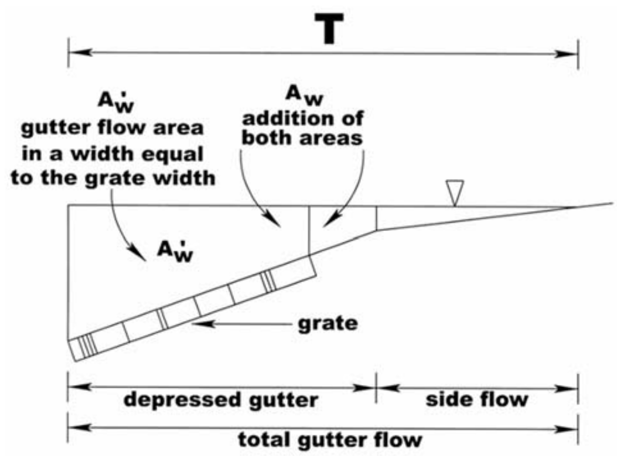
$$E_o' = E_o \left(\frac{A_W'}{A_W} \right)$$

Where: A'_W = depressed gutter flow area in a width equal to the grate, ft^2

A_W = flow area in the depressed gutter width, ft^2

Figure 5-2

Flow Area Ratio Grate Width Less Than Gutter Width



The ratio of side flow to total gutter flow (Q_s/Q) is:

$$\frac{Q_S}{Q} = 1 - \frac{Q_W}{Q} = 1 - E_o$$

The ratio of frontal flow intercepted to total frontal flow (R_f) is:

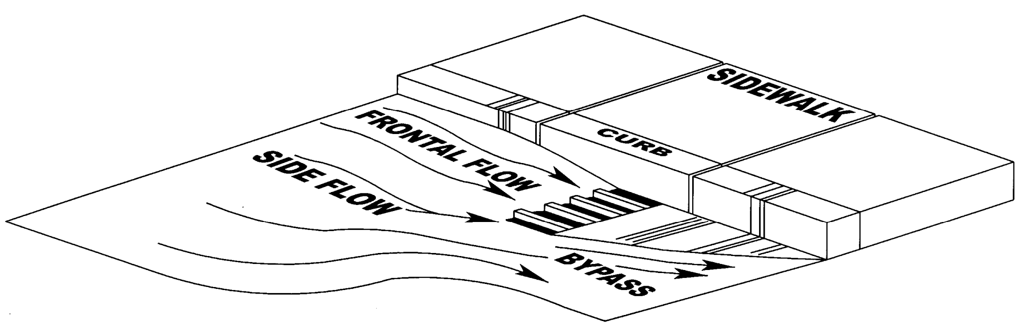
$$R_f = 1 - 0.09\left(V - V_o\right)$$

Where: V = average gutter velocity, ft/s

V_o = gutter velocity where splash-over occurs, ft/s (see Chart 5-7)

Figure 5-3

Intercepted Flow with Bypass



This ratio is equivalent to frontal flow interception efficiency or the ability of the inlet grate to accept the frontal flow approaching it. The average gutter velocity or total gutter flow divided by the gutter flow area is needed to calculate this ratio. Note that this ratio cannot exceed one. If it is equal to one, all of the flow is within the depressed gutter width and the grate is accepting the entire flow with no bypass.

The ratio of side flow intercepted to total side flow (R_s), or the ability of the inlet grate to accept the flow going by beside it (side flow efficiency) is:

$$R_S = \frac{1}{1 + \frac{0.15\,V^{1.8}}{S_X\ L^{2.3}}}$$

Where: L = length of clear opening area, ft

The total efficiency of a grate (E) is:

$$E = R_f\ E_o + R_S\left(1 - E_o\right)$$

The right side of the equation becomes the frontal flow intercepted / total gutter flow plus the side flow intercepted / total gutter flow. The second term on the right side is insignificant with high velocities and short grates. This leads to the total intercepted flow or capacity of the inlet grate being:

$$Q_i = E\,Q$$

The interception capacity of the curb opening and grate combination on a continuous grade is no greater than that of the grate alone; therefore, the above formulas are applicable to the combination inlets as well.

Chart 5-3 provides the ratio of frontal flow to total gutter flow (E_o or Q_w/Q) for uniform cross slopes. Chart 5-4 through Chart 5-6 are multi-purpose graphs that provide information for composite cross slopes based on the spread of flow. They are applicable for the composite cross slope that is diagramed at the bottom of each chart. Enter the chart with the conveyance then read the spread for that conveyance at the bottom. Where the vertical line crosses E_o, flow area and flow depth at the curb, read across to the left for the respective value. For values other than the composite cross slopes that are diagramed, use the provided formulas. Chart 5-7 and Chart 5-8 give the interception efficiency for frontal (R_f) and side (R_s) flow for the WVDOH standard inlets.

Chart 5-3

Ratio of Frontal Flow to Total Gutter Flow for A Uniform Cross Slope

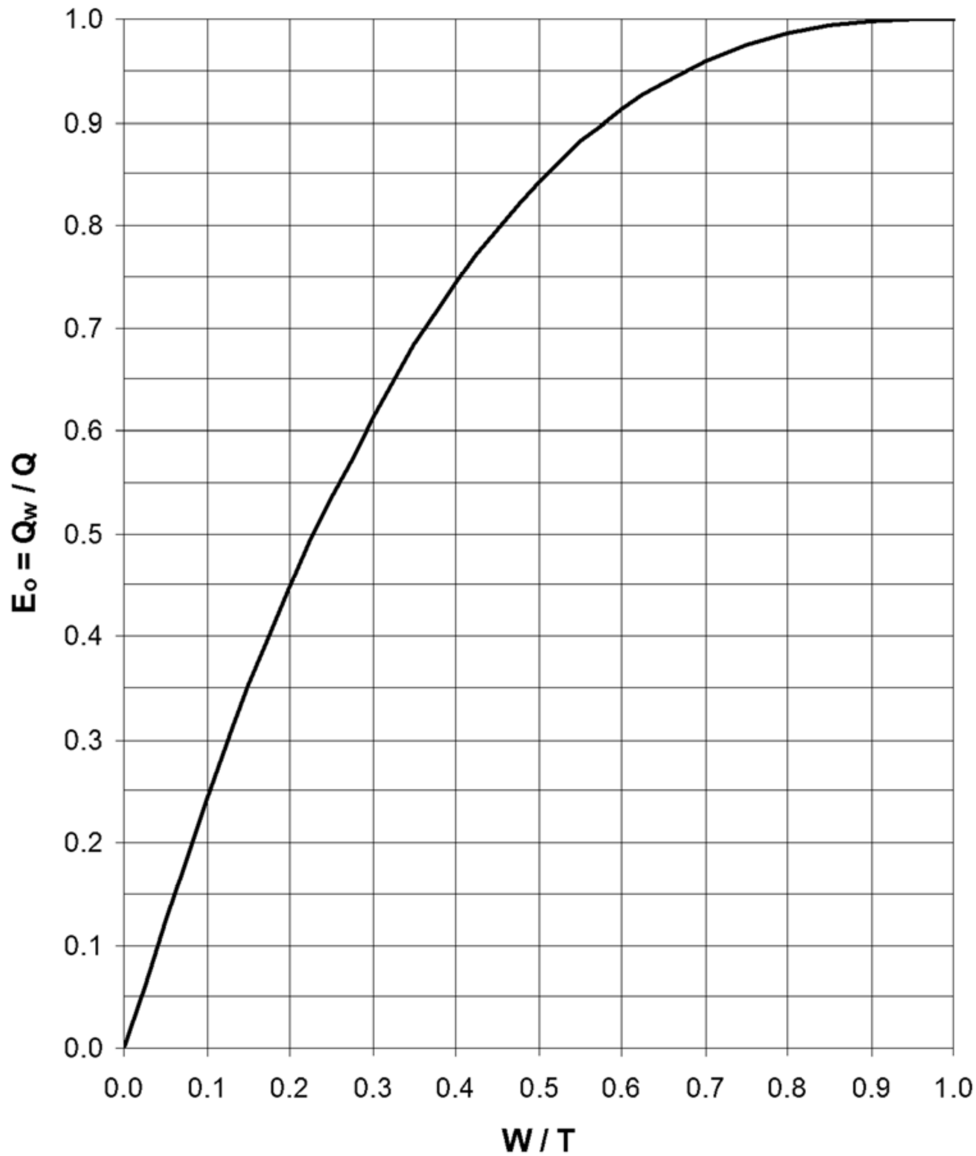


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Chart 5-4

Composite Cross Slope Conveyance 1% - 6%

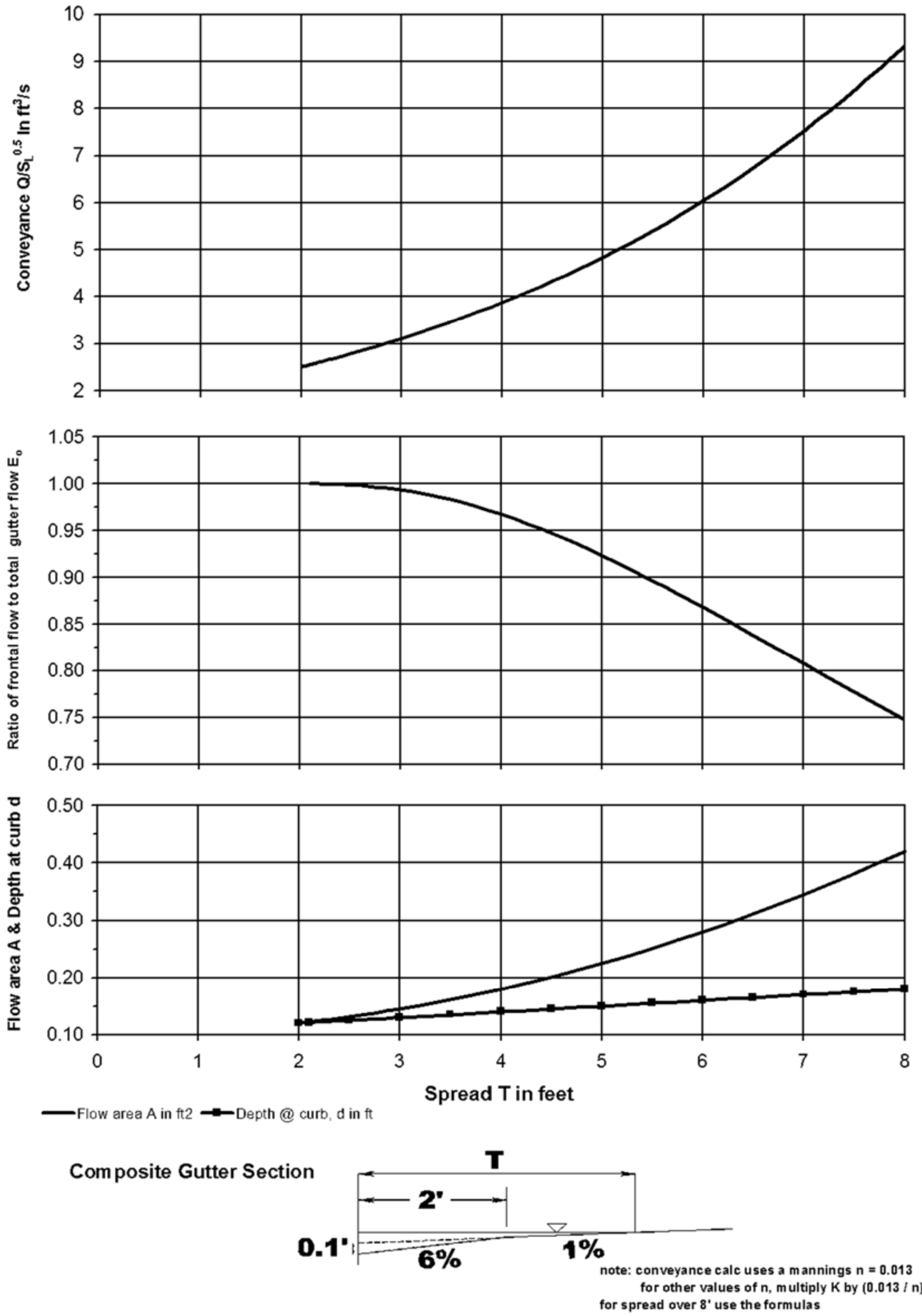

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Chart 5-5

Composite Cross Slope Conveyance 2% - 6%

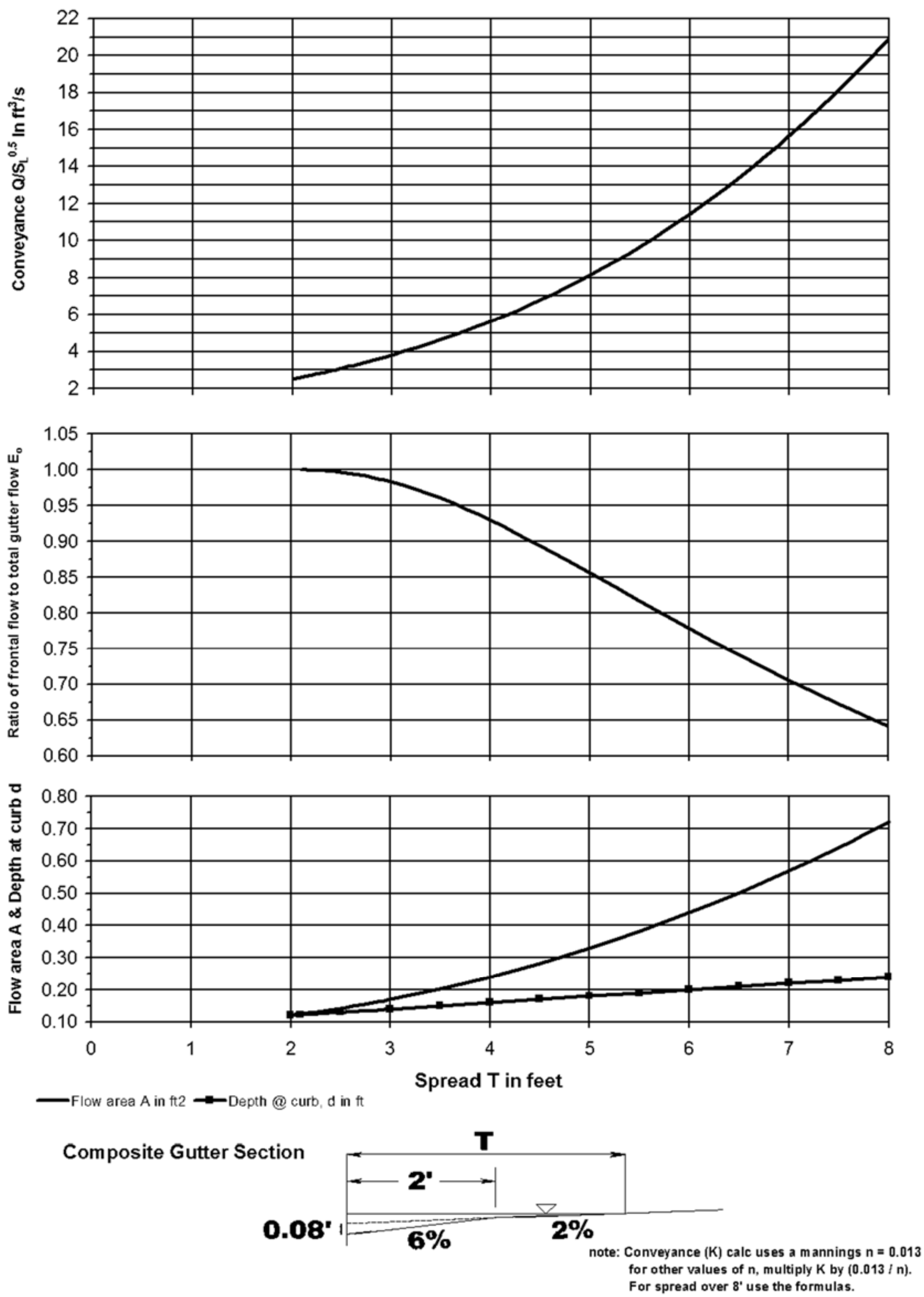

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Chart 5-6

Composite Cross Slope Conveyance 4% - 6%

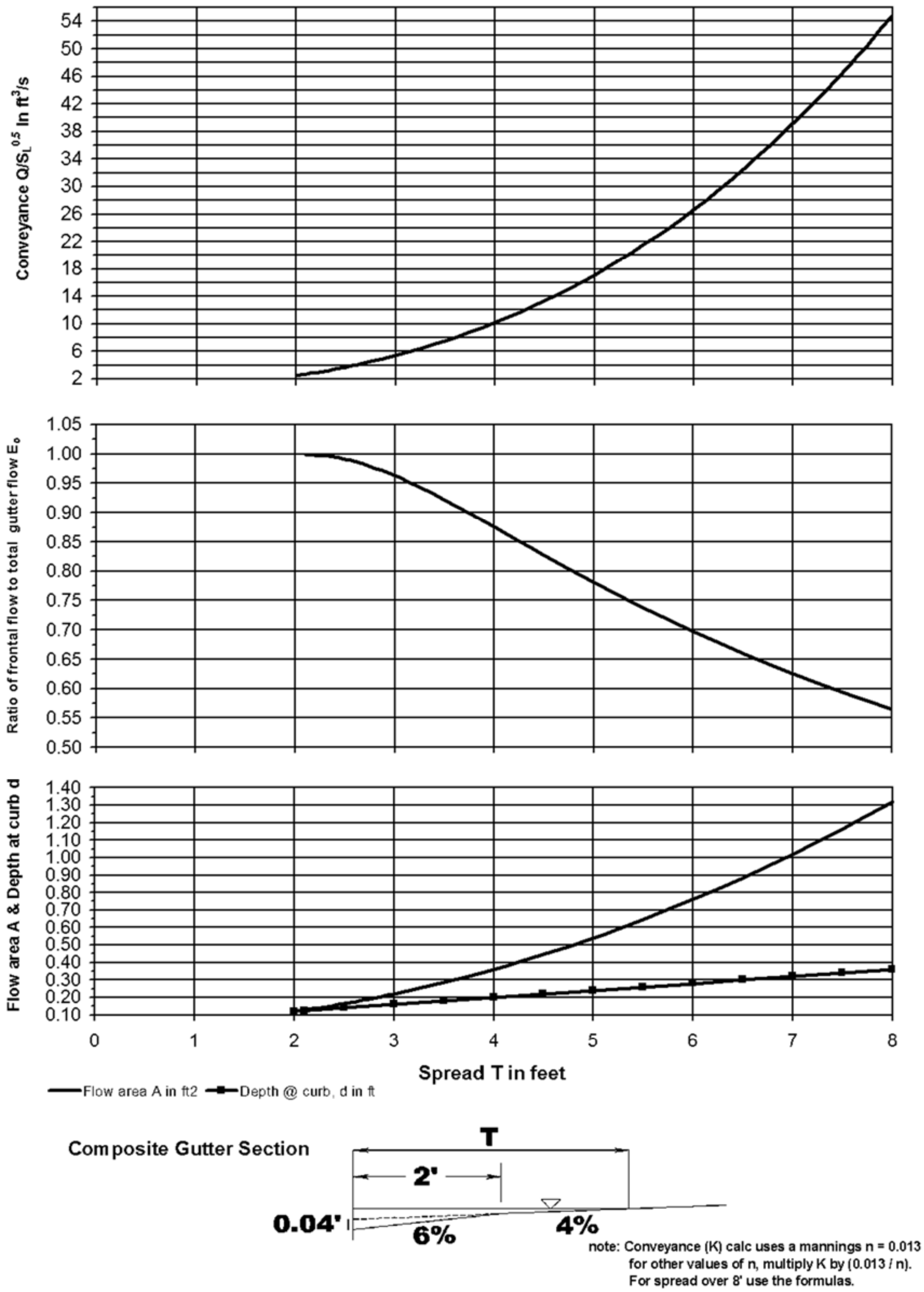

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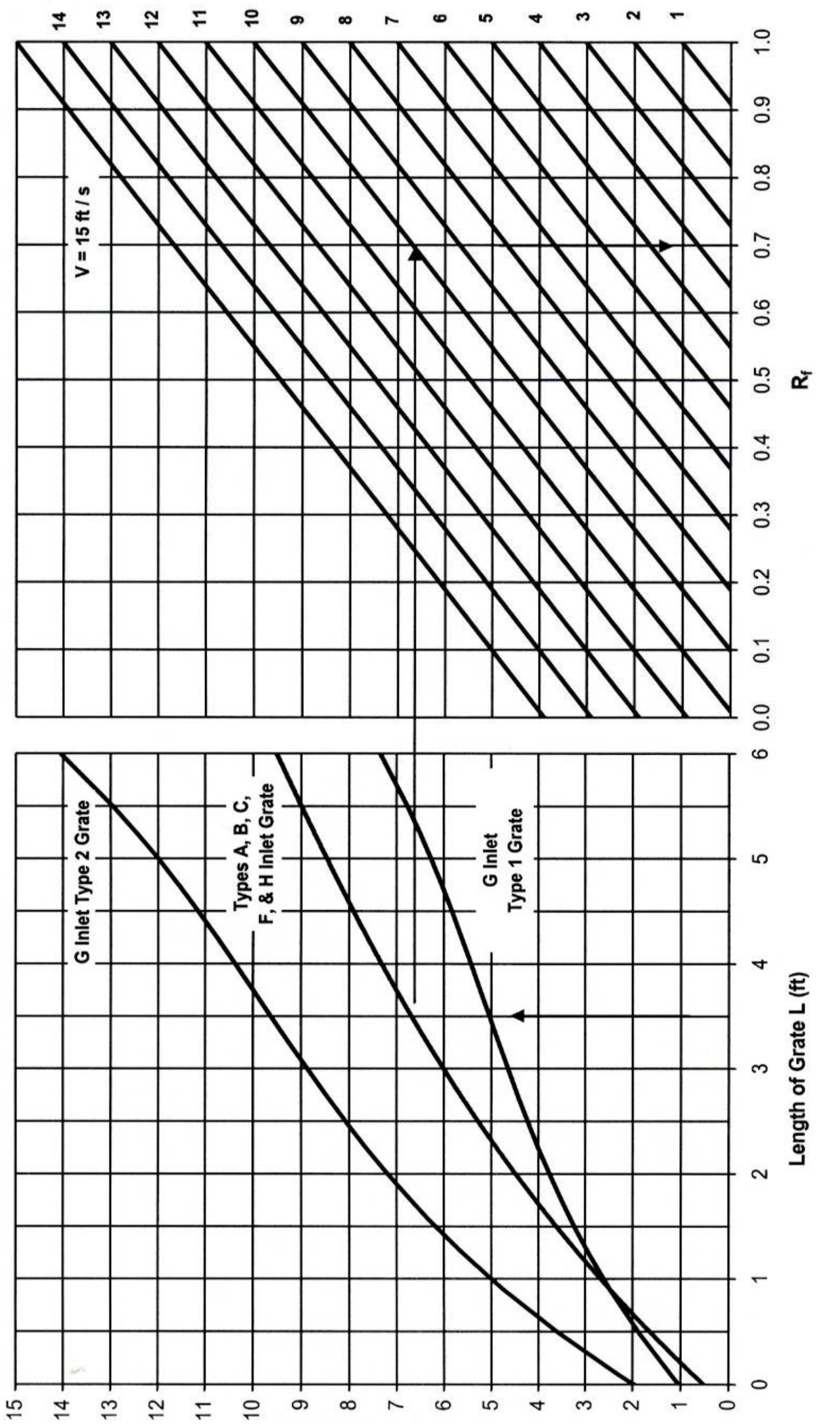
Chart 5-7

Grate Inlet Frontal Flow Interception Efficiency



Example: Find R_f for a type A inlet grate flowing at a velocity of 10 ft / s.

Solution: Type A inlet has a length of 42" or 3.5' R_f = 0.7

Splash-over velocity for G inlet Type 1 grate* $V_o = 0.46(2.22 + 4.03L - 0.65L^2 + 0.06L^3)$

Splash-over velocity for G inlet Type 2 grate** $V_o = 0.88(2.22 + 4.03L - 0.65L^2 + 0.06L^3)$

Splash-over velocity for types A, B, C, F, and H inlet grates*** $V_o = 0.51 + 2.34L - 0.20L^2 + 0.01L^3$

Ratio of frontal flow intercepted to total frontal flow $R_f = 1 - 0.09(V - V_o) \leq 1$

* The splash-over velocity curve for G inlet Type 1 grate is based upon an 54% reduction of the splash over velocity curve for the FHWA P-50 inlet grate.

**The splash-over velocity curve for G inlet Type 2 grate is based upon a 12% reduction of the splash over velocity curve for the FHWA P-50 inlet grate.

*** The 30° tilt bar grate is used for types A, B, C, F, and H inlets.

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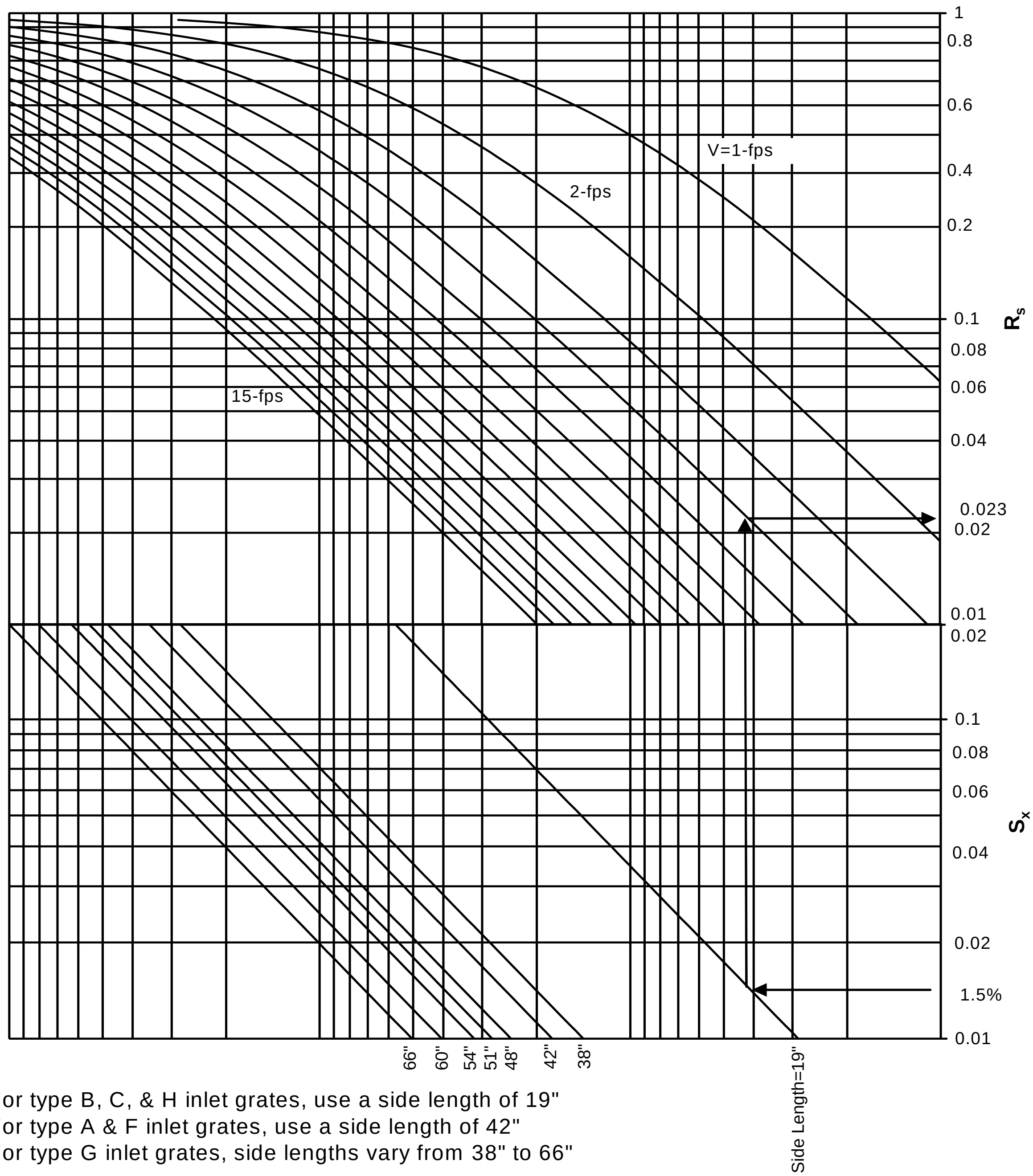
Chart 5-8

Grate Inlet Side Flow Interception Efficiency



For type B, C, & H inlet grates, use a side length of 19"
For type A & F inlet grates, use a side length of 42"
For type G inlet grates, side lengths vary from 38" to 66"

Ratio of side flow intercepted to total side flow

$$R_s = \frac{1}{1 + \frac{0.15 V^{1.8}}{S_x L^{2.3}}}$$

Example: Find Rs for a type C inlet grate on a slope of 1.5% moving at a velocity of 4-fps.
Solution: R_s=0.023

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5.3.4.5 CURB OPENING AND SLOTTED INLET CAPACITY ON GRADE

Curb opening and slotted inlets are less susceptible to clogging and offer little interference to traffic operation. They are a viable alternative to grates on flatter grades where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists. The method for determining the interception capacity of the curb opening inlet starts by calculating the length of opening required to intercept all of the flow approaching it. The efficiency of the actual curb opening is then determined by the use of a ratio of the actual length to the required length.

The length of the curb-opening inlet required for total interception of gutter flow (L_T) on a pavement section with a uniform cross slope is expressed by:

$$L_T = 0.6\, Q^{0.42}\ S_L^{0.3} \left(\frac{1}{n\, S_X} \right)^{0.6}$$

Where: S_L = longitudinal slope, ft/ft

S_X = cross slope, ft/ft

Q = gutter flow, ft^3/s

n = Mannings roughness coefficient

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left(1 - \frac{L}{L_T} \right)^{1.8}$$

Where: L = actual curb opening length, ft

For composite cross slopes, the length of inlet required for total interception can be found by the use of an equivalent cross slope, S_e in place of S_X. S_e is expressed by:

$$S_e = S_X + S_W^{'}\ E_o$$

Where: S'_W = a / (12 W) or = $S_W - S_X$ see Figure 5-1

W = depressed gutter width, ft

a = gutter depression, in

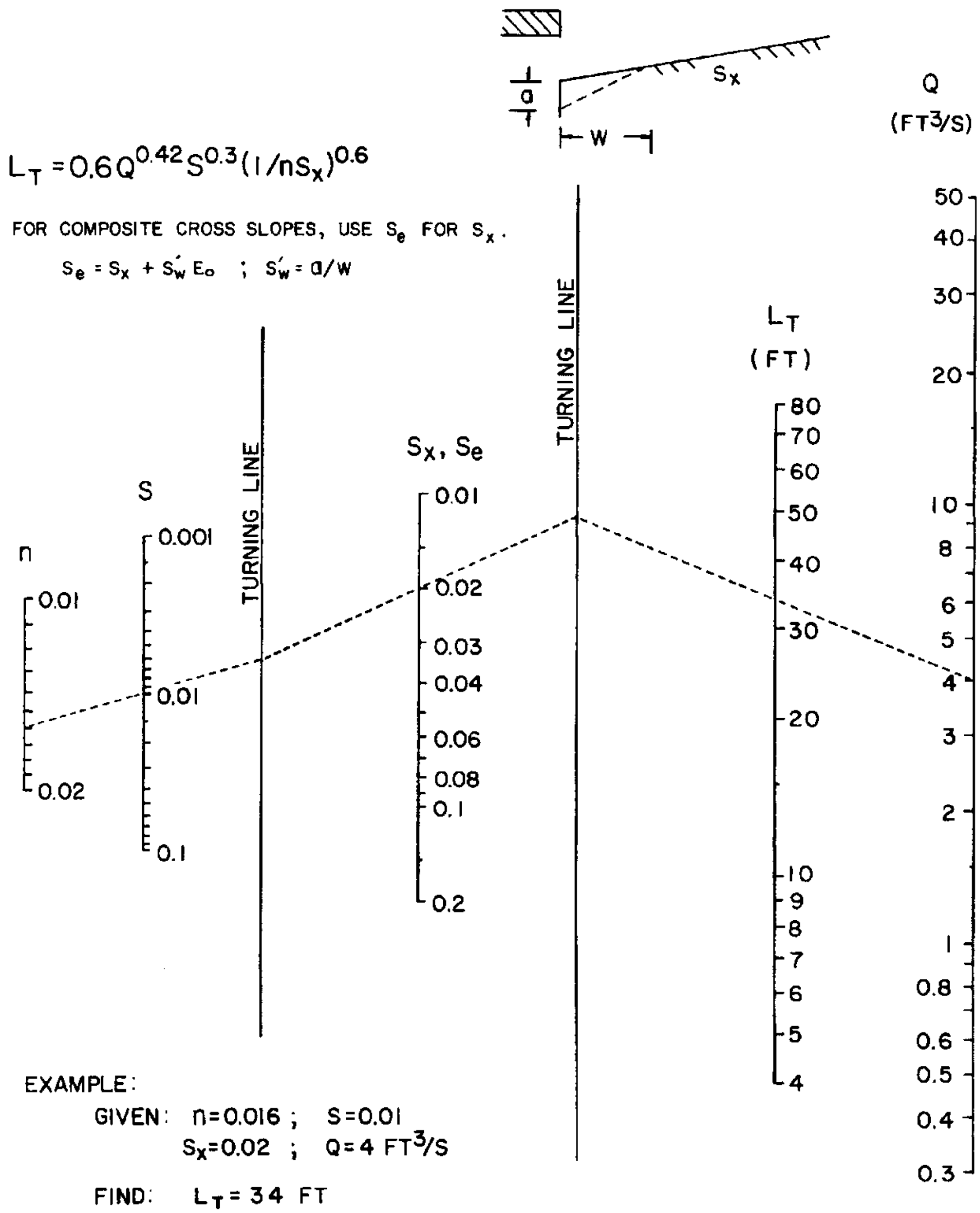
E_o = ratio of frontal flow to total gutter flow for a composite cross slope as defined in section 5.3.4.4

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Analysis of data from the FHWA tests of slotted inlets with slot widths greater than 1 ¾" indicates that the length of slotted inlet required for total interception can be computed with the same equation used for curb openings. Similarly, the same equation can be used to obtain the inlet efficiency for the selected length of inlet.

Chart 5-9 provides the required inlet length for total interception for curb opening and slotted inlets with reference to composite cross slopes. Chart 5-10 provides the inlet interception efficiency for curb opening and slotted inlets.

Chart 5-9

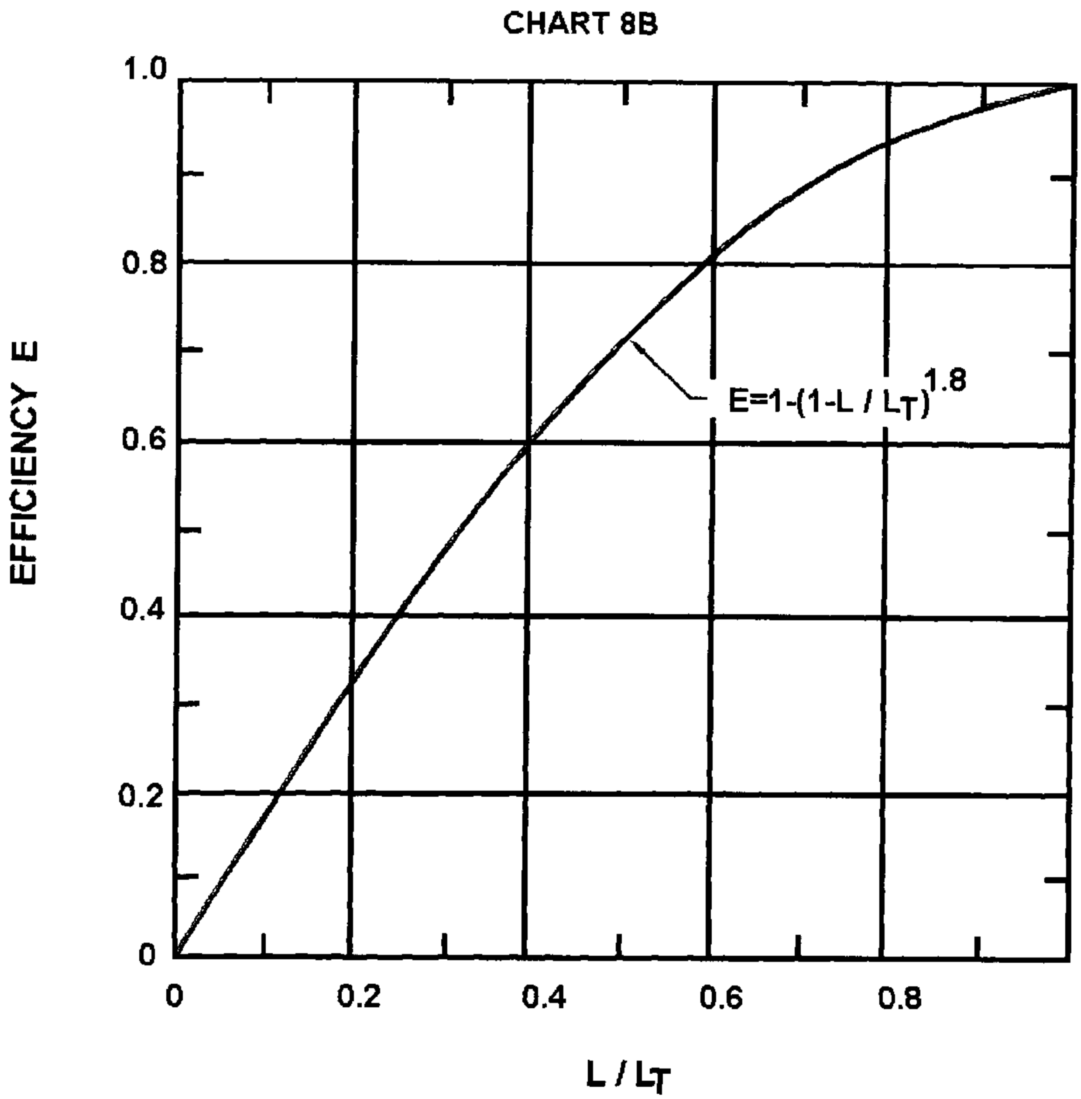
Curb Opening & Slotted Inlet Length for Total Interception



Source: Federal Highway Administration Hydraulic Engineering Circular-22

Chart 5-10

Curb Opening & Slotted Inlet Interception Efficiency



Source: Federal Highway Administration Hydraulic Engineering Circular-22

5.3.4.6 OVERLAND FLOW INLET CAPACITY

In situations where a side road (such as a driveway or commercial access road) intersects a major route, overland flow from the side road shall not be allowed to cross onto the major route. It shall be intercepted by an inlet and carried to a neighboring ditch, inlet or storm sewer. Since many of these side roads are not crowned to facilitate shallow concentrated flow conditions at the edges, wide sheet flow results. Thus, it is necessary to intercept flow over the entire width of the side road. Continuous inlets such as slotted or concrete box and grate inlets should be used across the entire width of the side road to capture the wide, shallow sheet flow. Short sag vertical curves located just off the mainline shoulder can be used to increase interception efficiency.

When slotted inlets are used in a transverse direction to capture overland flow, research has indicated that for water depths ranging from 0.38 inches to 0.56 inches, the 1, 1.75 and 2.5 inch wide slots can accommodate a flow of 0.03 $ft^3/s/ft$. This capacity is applicable for longitudinal slopes ranging from 0.5% to 9%. Slotted vane drains provide additional capacity due to the geometric properties of the vane. On longitudinal slopes of 0.5% to 6% they can accommodate a flow of 0.50 $ft^3/s/ft$.

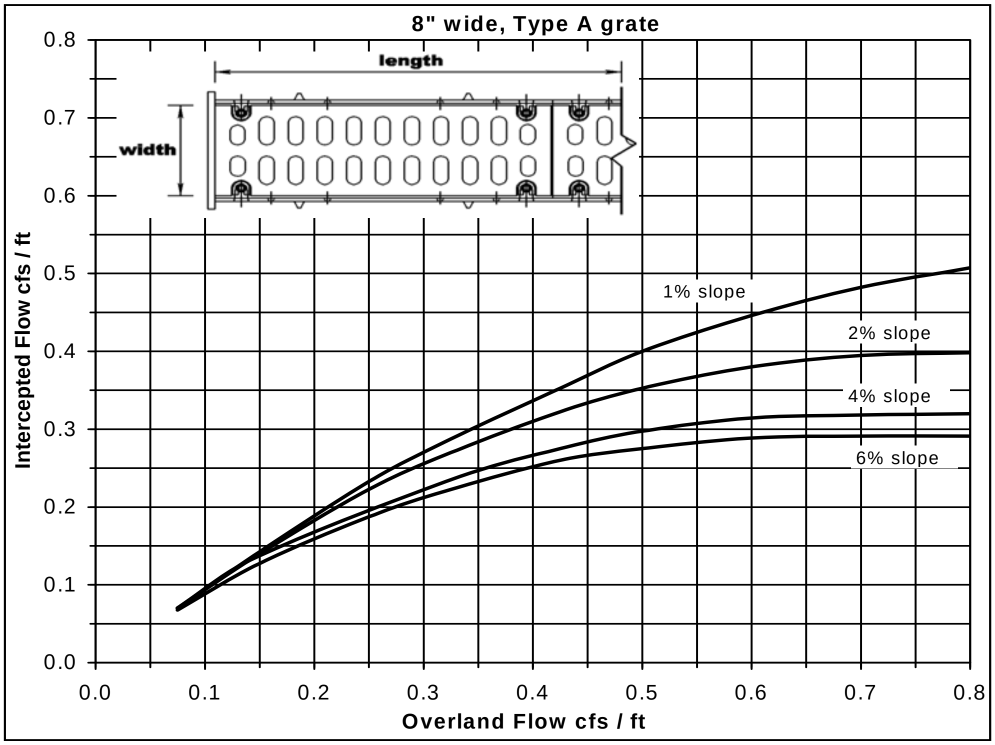
Capacity testing performed in 1976 by a major grate manufacturer provides data for interception and bypass of flow for various sizes of continuous concrete box and grate inlets. The testing was performed on two types of grates (Type A and Type C) for widths of 8 to 20 inches and a third type (Type P) for a width of 14 inches. The types vary by the size and configuration of the holes (see Section 5.3.4.1).

Chart 5-11 through Chart 5-17 provide illustrations and interception capacities for the types and sizes of grates tested for various longitudinal slopes. Type A and Type C grates have a 92% interception capacity for flows less than 0.60 $ft^3/s/ft$. Type P grates are intended for high pedestrian areas and only provide about 28% interception capacity for the same flow.

The flow rate entered on the x-axis of these charts is only that portion of overland flow that is directly approaching the inlet as frontal flow. The direction of the overland flow is perpendicular to the grate length shown on the schematic in each chart.

Chart 5-11

Concrete Box with Type A Grate (8”, 10”)



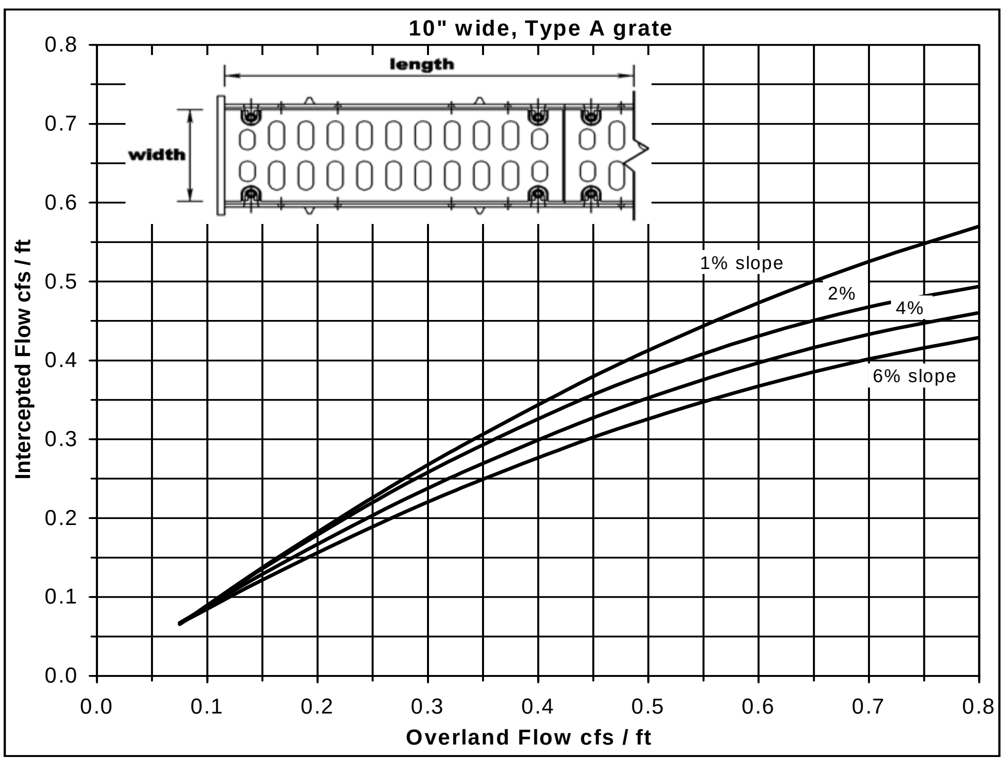


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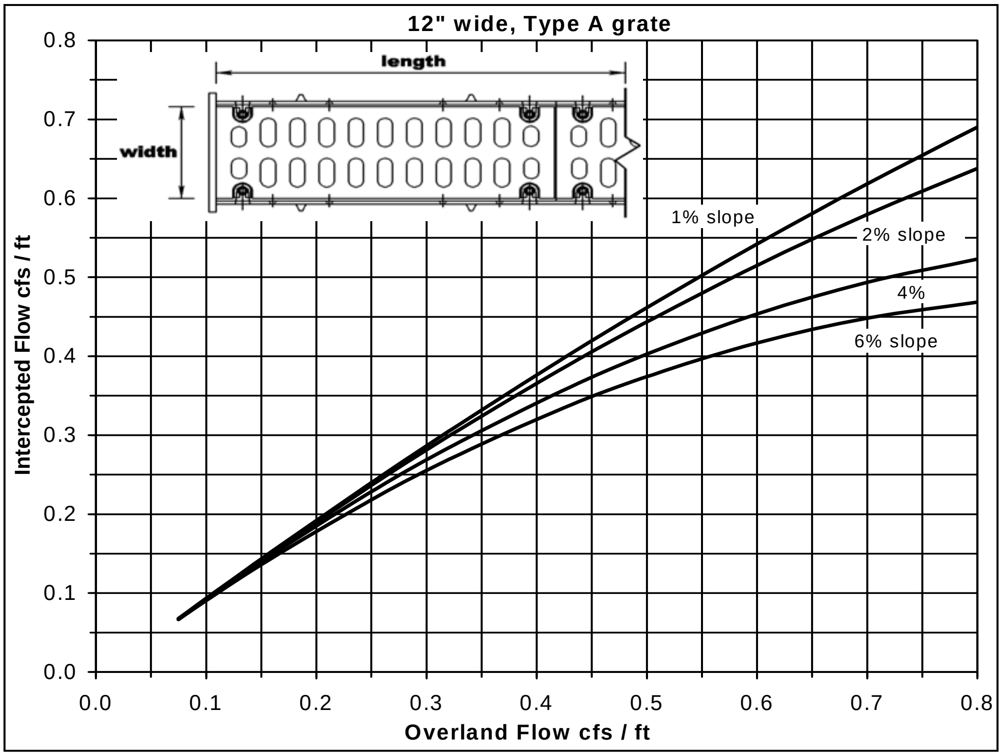
Chart 5-12

Concrete Box with Type A Grate (12”, 14”)



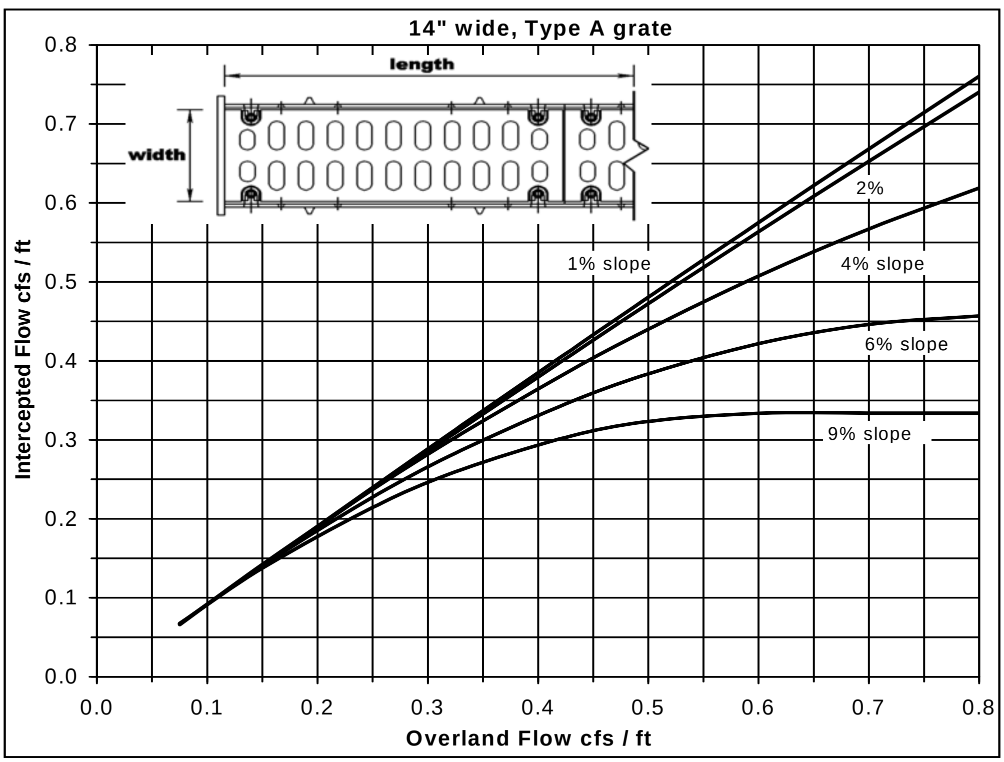


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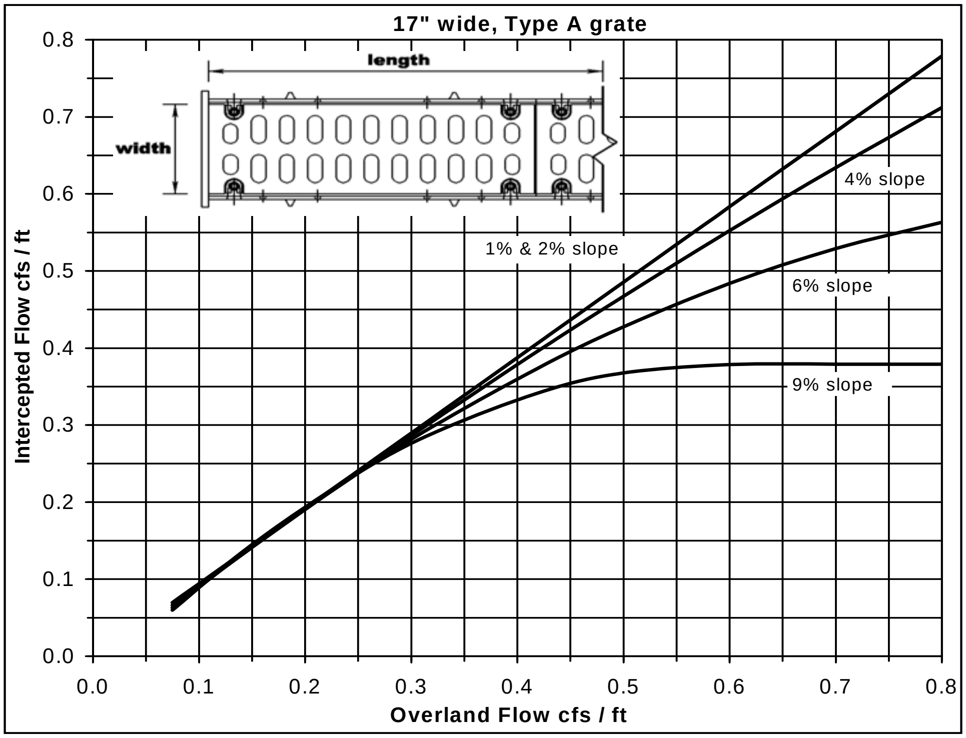
Chart 5-13

Concrete Box with Type A Grate (17", 20")



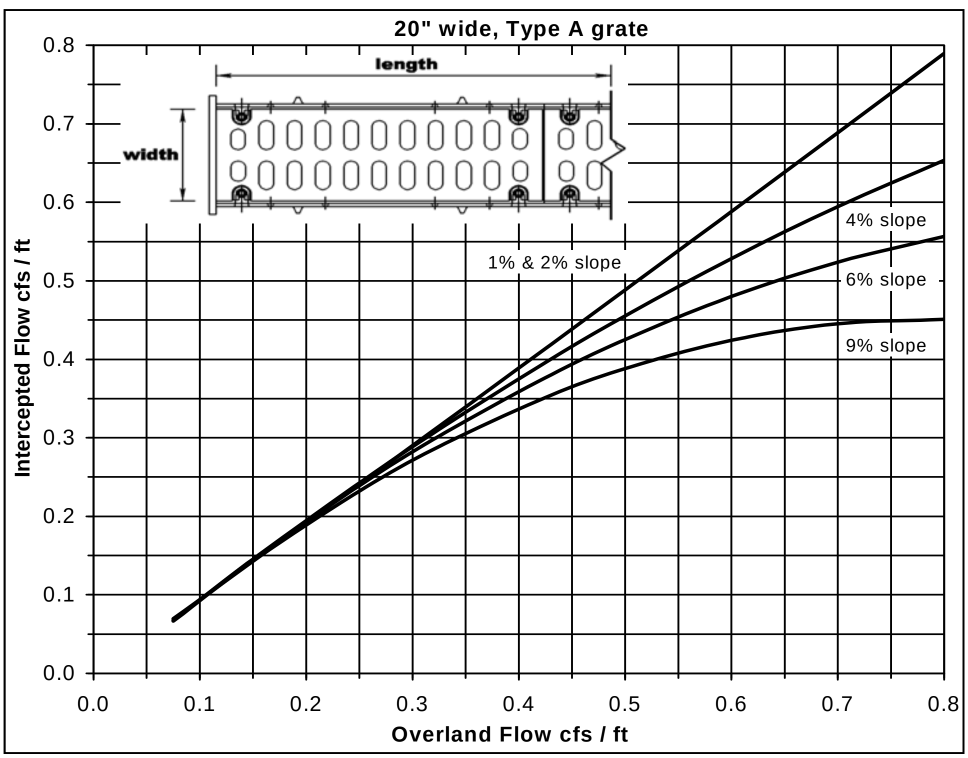


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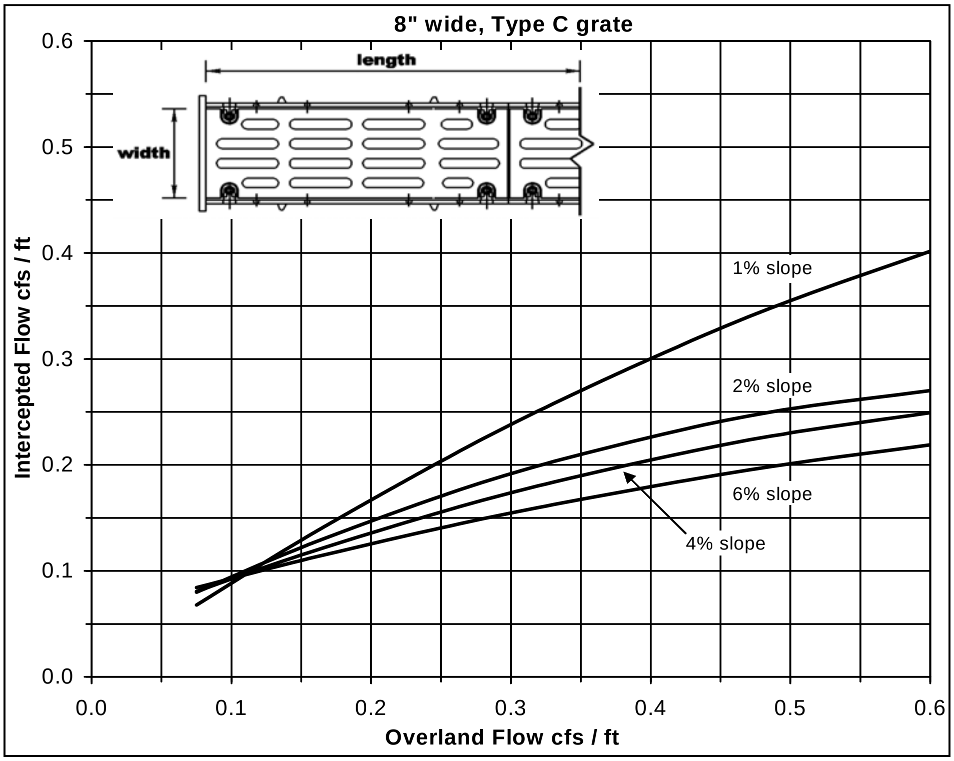
Chart 5-14

Concrete Box with Type C Grate (8", 10")



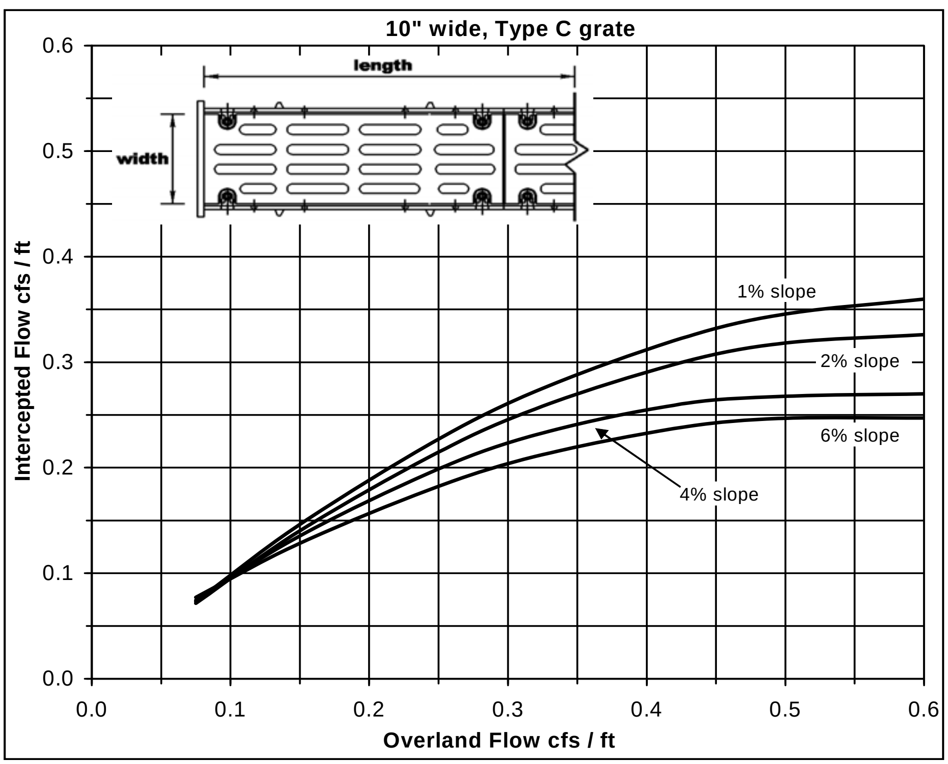


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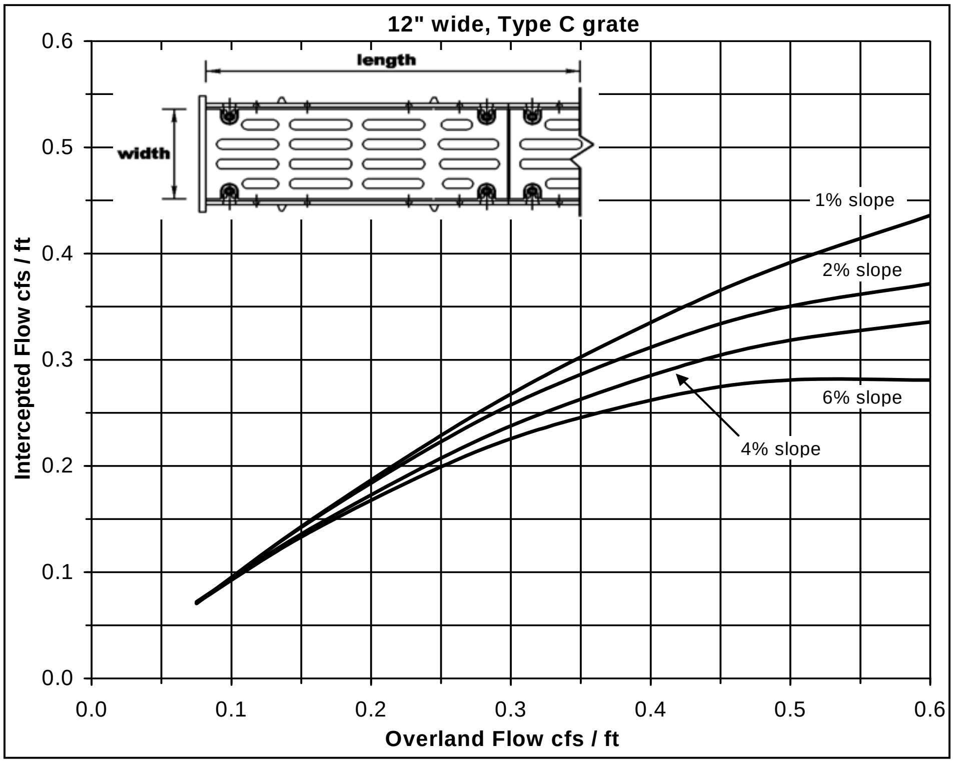
Chart 5-15

Concrete Box with Type C Grate (12", 14")



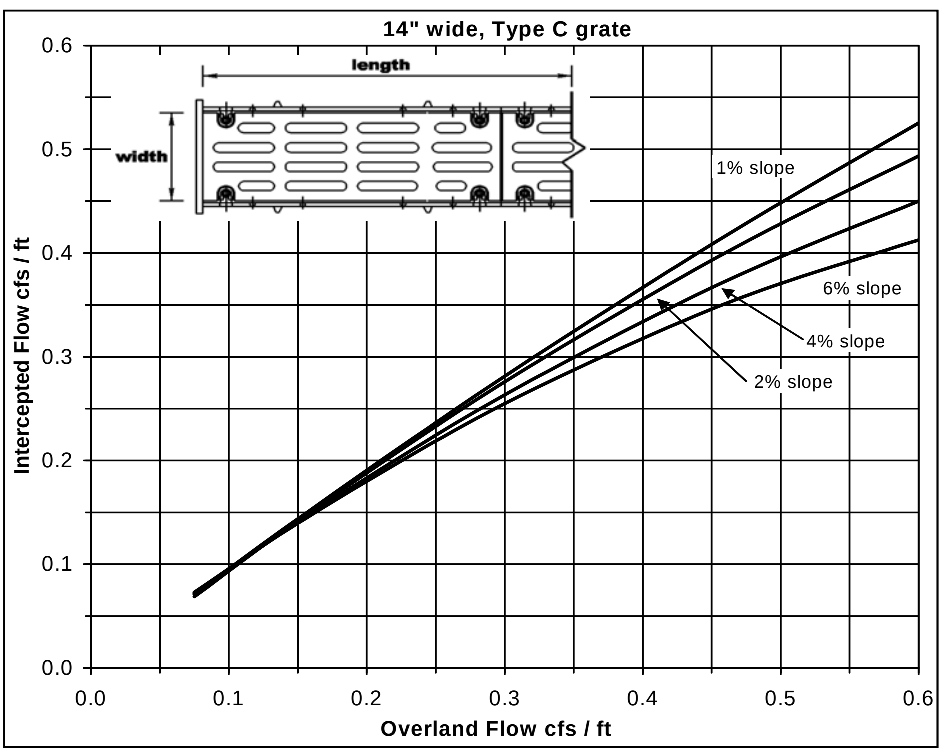


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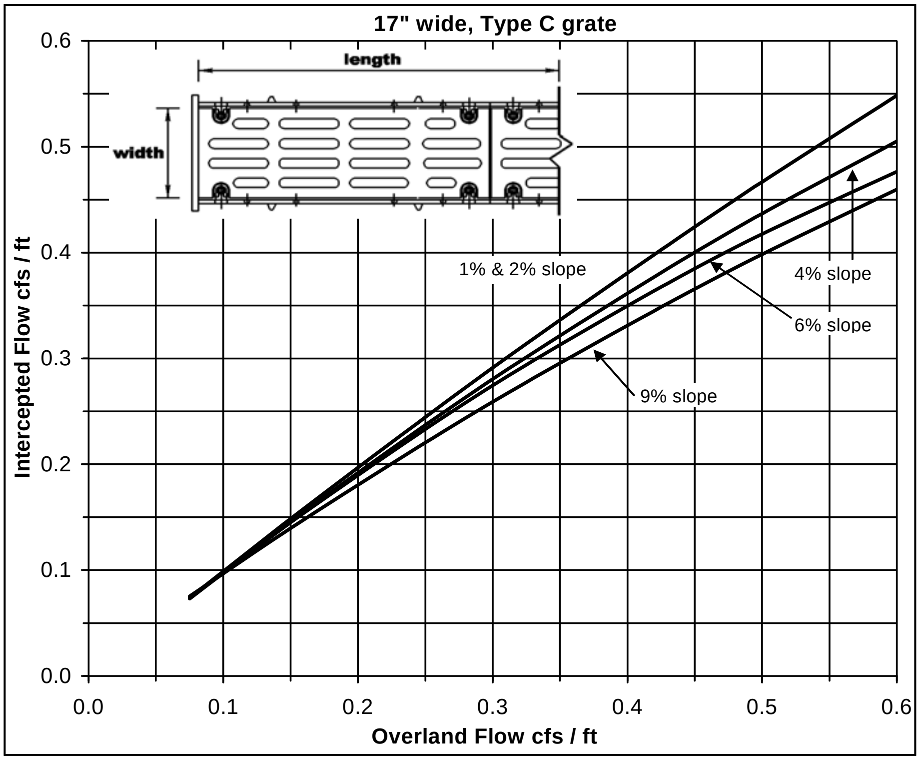
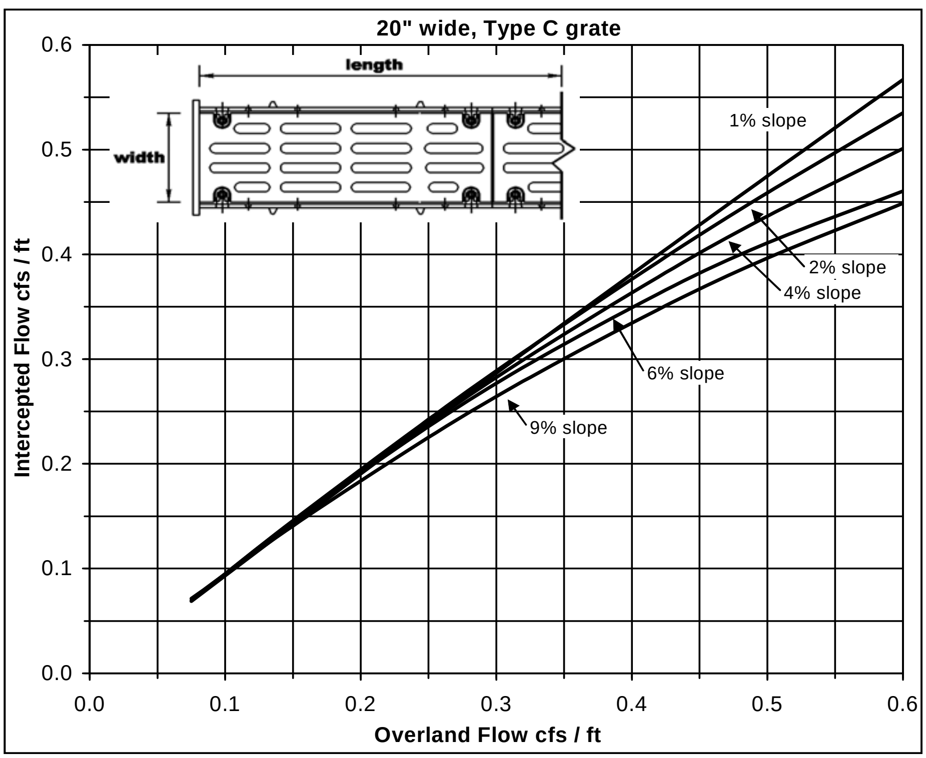
Chart 5-16

Concrete Box with Type C Grate (17”, 20”)

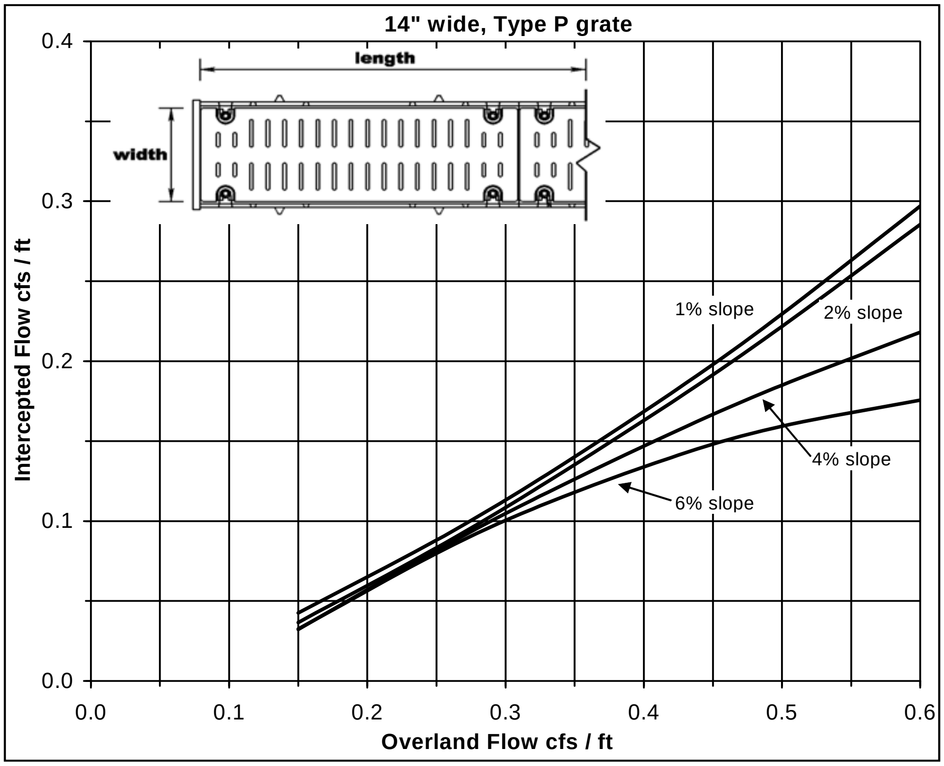




Created by the WVDOH Hydraulic and Drainage Unit

Chart 5-17

Concrete Box with Type P Grate (14”)



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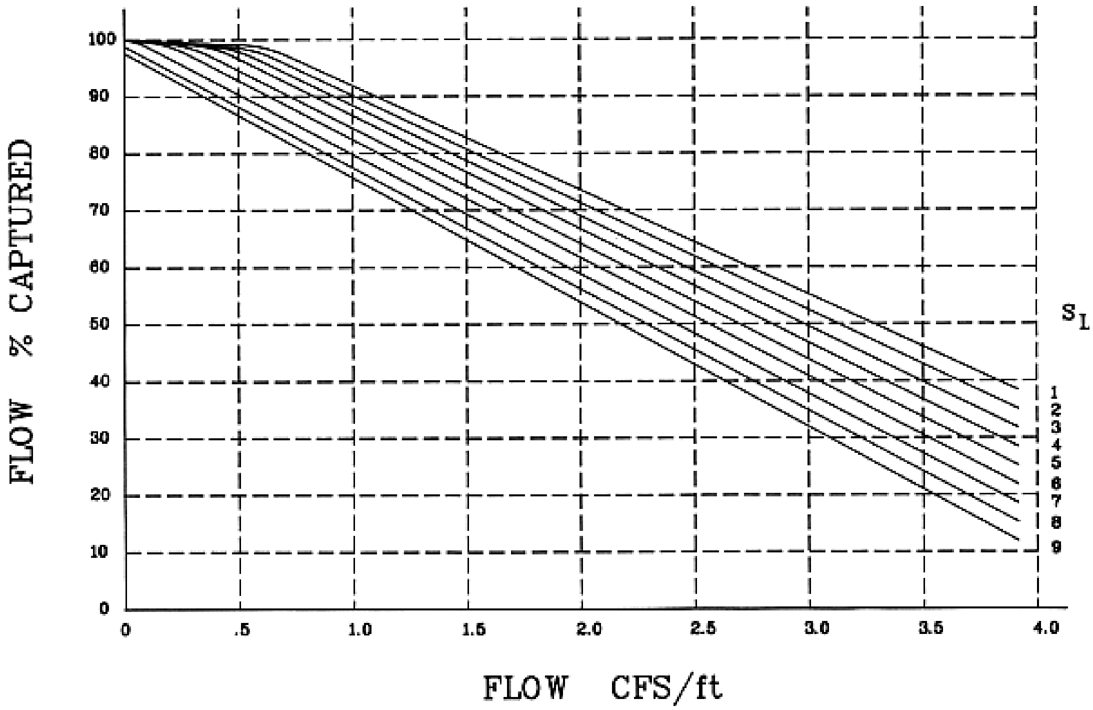
Continuous vane style inlet grates are also available for the cast in place concrete box drains. These provide additional capacity for flow over a longitudinal grade than the Type A or C grates. Testing was performed by the same major grate manufacturer for a 14” wide vane grate and is provided in Chart 5-18. Further research should be done for the inlet capacity of grate sizes that were not tested. If no information is available for the desired grate width, engineering judgment may be used, utilizing the data available as an estimate.

Chart 5-18

Concrete Box with Vane Style Grate (14”)



Source: Neenah Inlet Grate Capacities, 1987

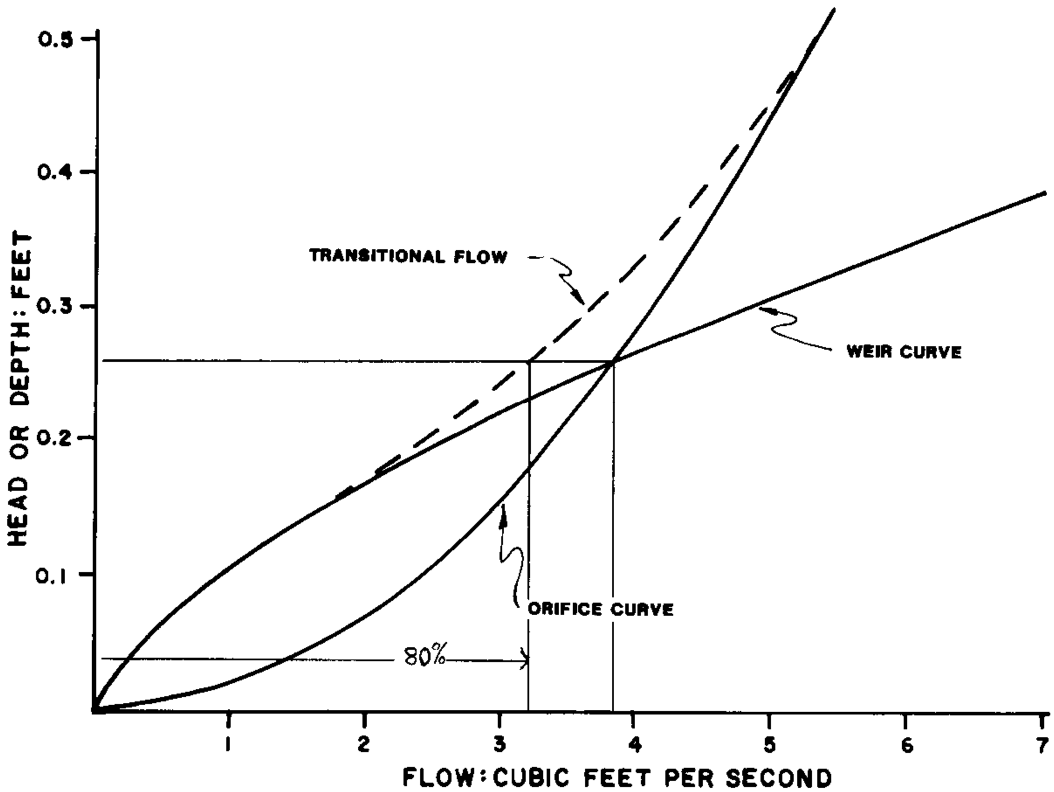
5.3.4.7 GRATE INLET CAPACITY IN A SAG LOCATION

A grate inlet in a sag location operates as a weir to depths that are dependent on the size of the grate and as an orifice at greater depths. Grates of larger dimension will operate as weirs to greater depths than smaller grates. The capacity controlling parameters for weir and orifice flow are the effective perimeter (or footage of the sides subject to flow) and the clear opening area, respectively.

The transition from weir to orifice flow results in an interception capacity that is less than that computed by either the weir or the orifice equation. According to research performed by a major grate manufacturer, this capacity can be approximated by drawing a curve between the lines representing the weir and orifice flow. When both weir and orifice flow rates are approximately the same, a vortex appears over the grate causing this reduction in capacity. A conservative estimate of 80% of capacity at the depth where this vortex occurs was used to approximate the curve for transitional flow.

Figure 5-4

Depth at 80% Capacity in a Sag



Source: Neenah Inlet Grate Capacities, 1987

For example, a 2' X 2' tilt bar grate inlet (Type B) in a curb and gutter section, in a sag location operates as a weir up to a depth of about 8 inches (0.68'). For this 30° tilt bar grate used by the WVDOH, the effective clear opening area that determines orifice flow capacity is 34% of the total area of the grate. This yields a clear opening area of 1.36 ft2 with orifice flow beginning at a depth of about 11.5 inches (0.89'). At a depth of 0.47 feet the weir and orifice flow were equal. Eighty percent of the capacity at this depth yields a flow rate of 3.99 ft3/s which gives a fixed point on the transitional flow line. Once drawn, the transitional flow line extends from a point before the limit of weir flow to a point after the limit of orifice flow for a conservative representation of flow behavior of a 2' X 2' tilt bar grate in a sag location.

The capacity of a grate inlet operating as a weir is:

$$Q_i = 3 P d^{1.5}$$

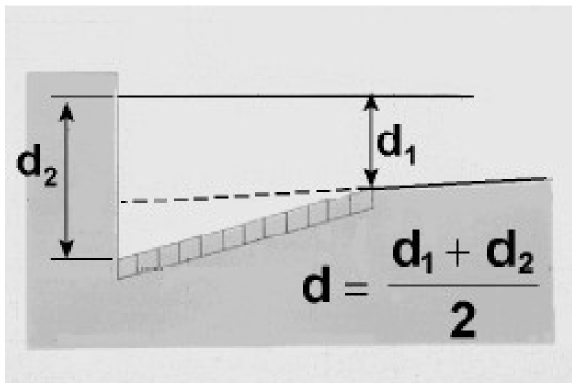
Where: P = effective perimeter of grate, ft

d = average depth across the grate, ft

(d_1 (adjacent to road) + d_2 (adjacent to curb)) / 2, ft

Figure 5-5

Average Depth Over a Grate



The capacity of a grate inlet operating as an orifice is:

$$Q_i = 0.67 A (2 g d)^{0.5}$$

Where: A = effective clear opening area of the grate, ft^2

g = gravitational acceleration, 32.2 ft/s^2

d = average depth across the grate, ft

Formulas for depths of flow adjacent to the road and adjacent to the curb for composite gutter sections can be found on Figure 5-1. Chart 5-19 through Chart 5-21 can be used to determine the grate inlet capacity of the WVDOH standard grate inlets in a sag location.

Chart 5-19

Type A Grate Inlet in a Sag Location

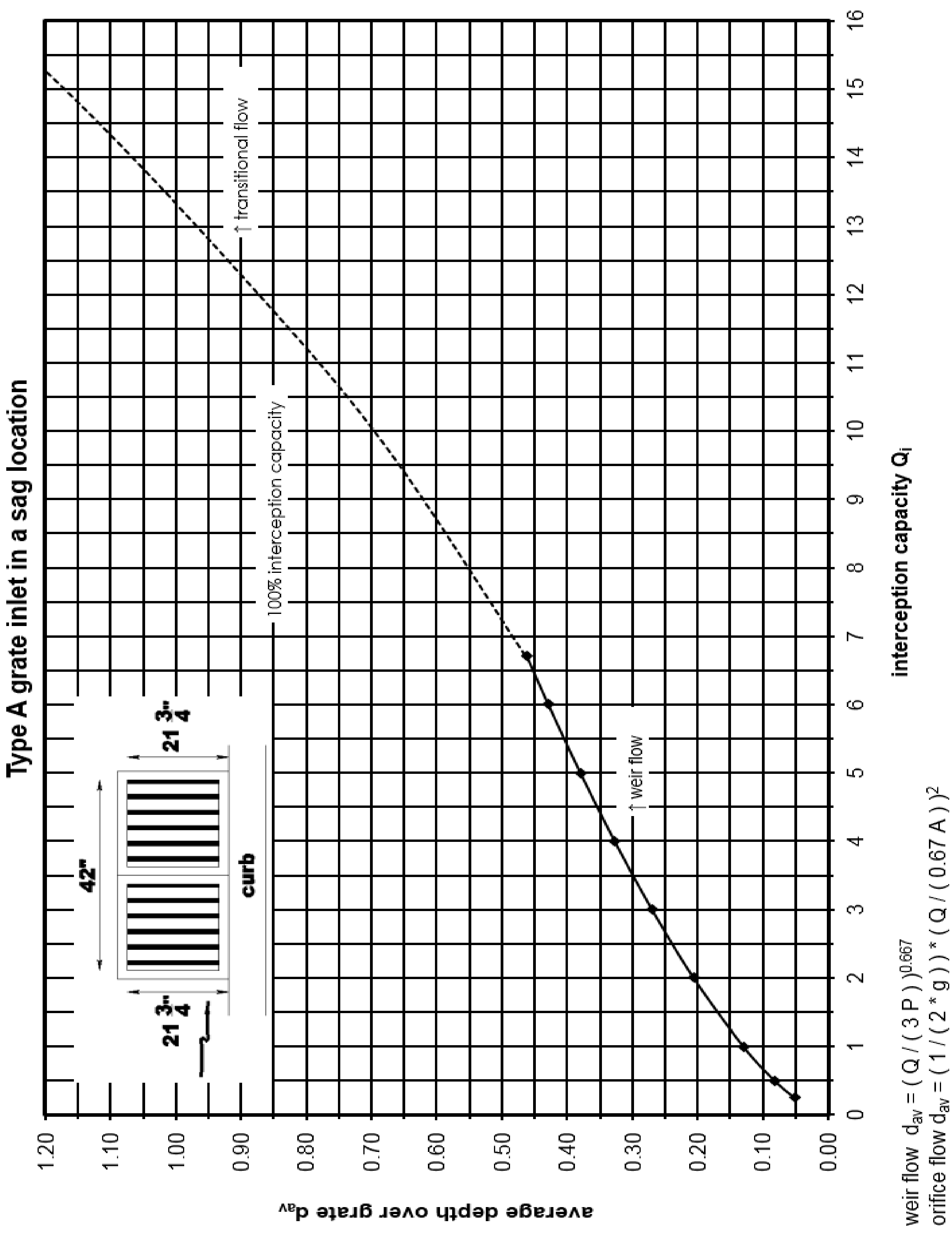


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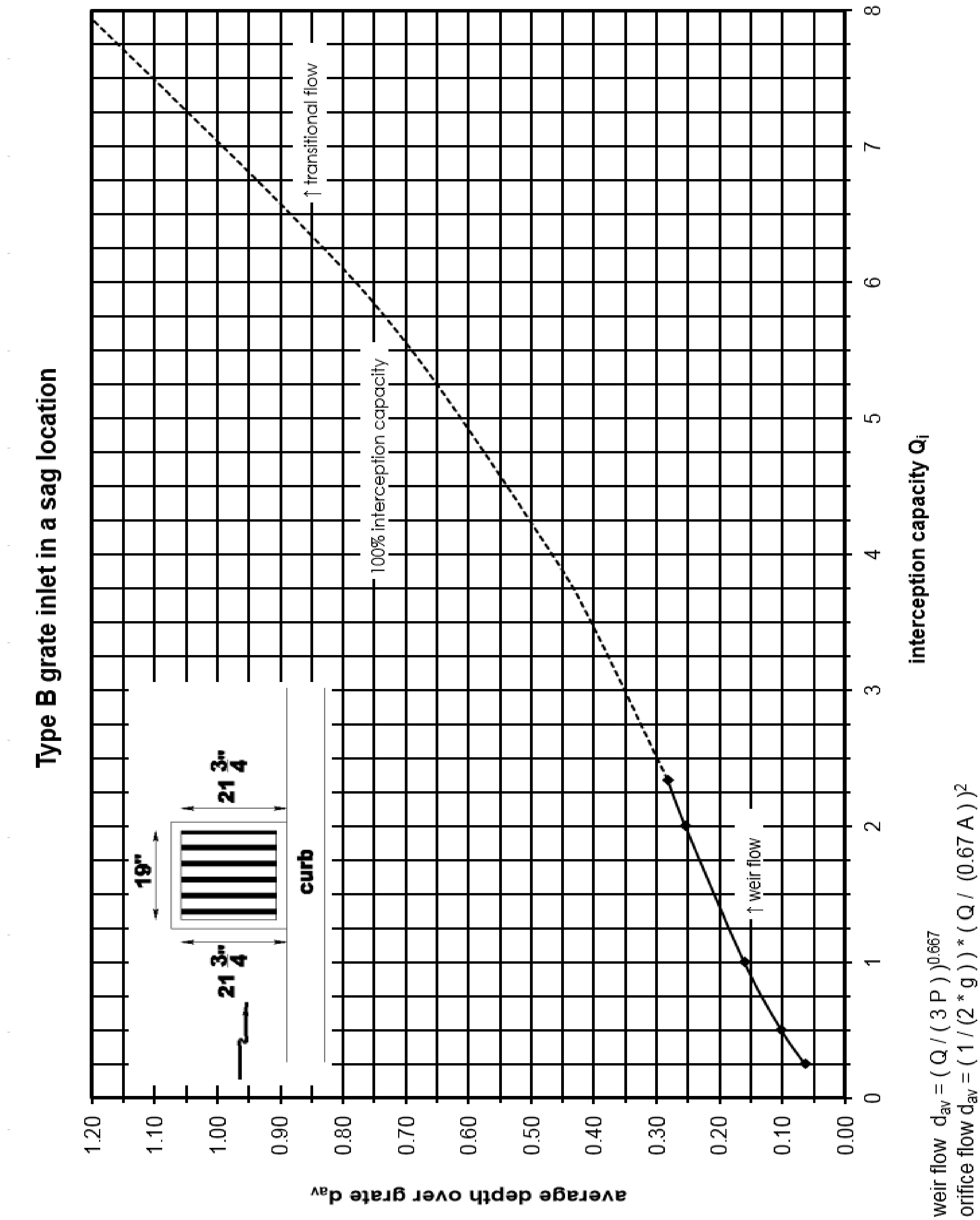
Chart 5-20

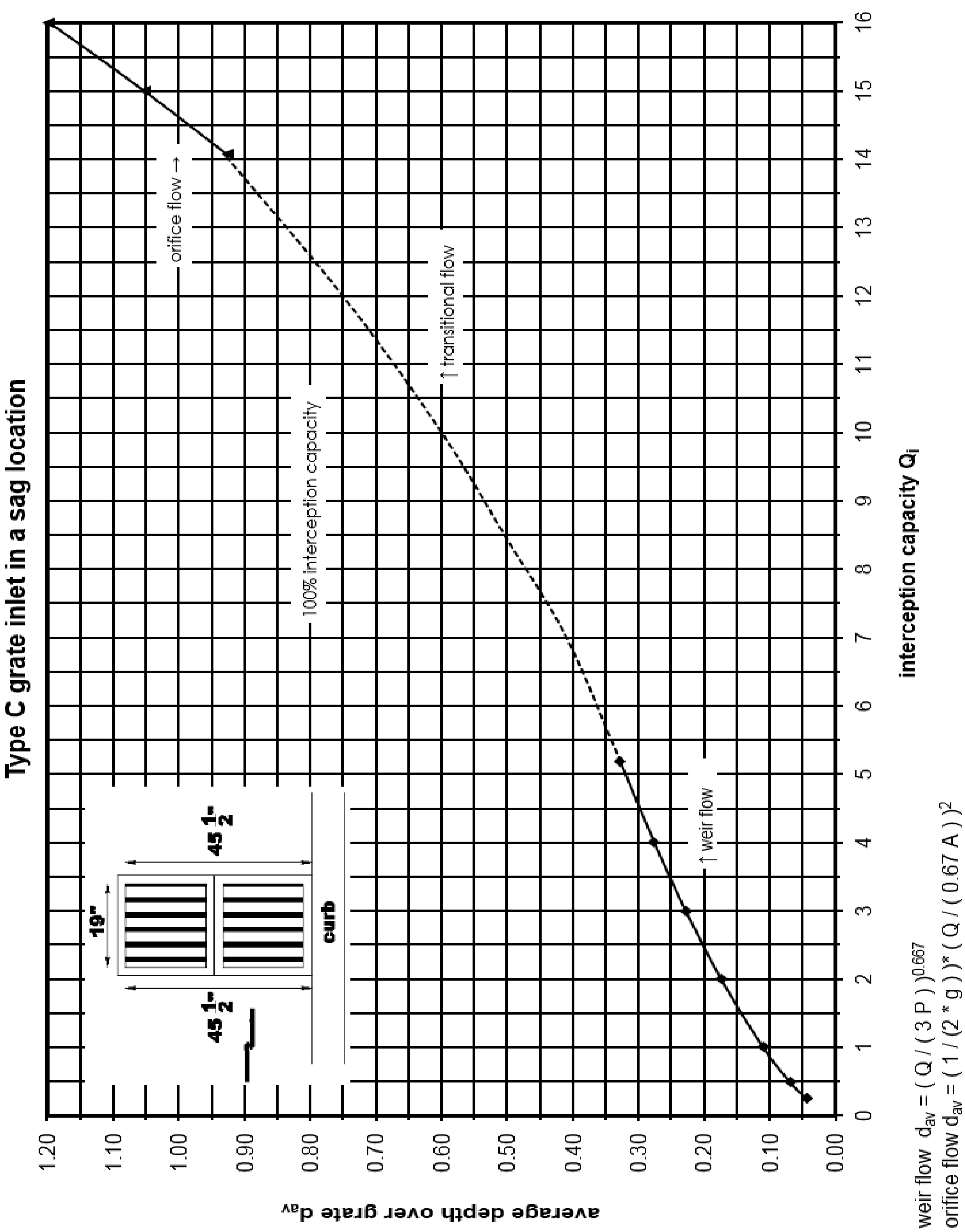
Type B Grate Inlet in a Sag Location



Created by the WVDOH Hydraulic and Drainage Unit

Chart 5-21

Type C Grate Inlet in a Sag Location



Created by the WVDOH Hydraulic and Drainage Unit

Equations that approximate the transitional flow line are provided for the purpose of spreadsheet development by users of this manual. They are provided in Table 5-4 through Table 5-6.

Table 5-4

Transitional Flow Line Equations for Type A, B, C Inlets

Line	Weir flow limit depth	d_{AV} vs. Interception Capacity	Orifice flow limit depth
A 100% capacity	0.46	$d_{AV} = 0.0028Q^2 + 0.0252Q + 0.1702$	1.52
B 100% capacity	0.28	$d_{AV} = 0.0137Q^2 + 0.0230Q + 0.1546$	1.23
C 100% capacity	0.34	$d_{AV} = 0.0028Q^2 + 0.0149Q + 0.1755$	0.93

5.3.4.8 CURB-OPENING INLET CAPACITY IN A SAG LOCATION

The capacity of a curb-opening inlet in a sag location depends on the projected water depth at the curb opening, the curb opening length, and the height of the curb opening. The inlet 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. At depths between 1.0 and 1.4 times the opening height, flow is in a transitional stage. If the opening is depressed, the depth of the depression is not added to the opening height to determine the limits of weir and orifice flow.

The capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_i = 2.3\left(L + 1.8\,W\right) d^{\,1.5}$$

Where: L = length of curb opening, ft

W = lateral width of depression, ft

d = depth at the curb measured from the cross slope

(i.e. projected depth $d = TS_x$), ft

WVDOH standard curb-opening inlets have a 4 inch opening and are depressed 2 inches. The lateral width of depression is 1 foot. For curb-opening lengths greater than 12 feet, the weir equation becomes:

$$Q_i = 3\,L\,d^{\,1.5}$$

The capacity of a curb-opening inlet operating as an orifice is:

$$Q_i = 0.67\ h\ L \left[2\ g \left(d - \left(\frac{h}{2} \right) \sin\theta \right) \right]^{0.5}$$

Where: h = orifice throat width, ft

L = length of curb opening, ft

d = depth at the curb, ft

θ = angle of inclination for the inlet throat, degrees

WVDOH standard curb-opening inlets have an inclined throat at an angle of 45° with a throat width of 4 inches. Chart 5-22 through Chart 5-26 can be used to determine the inlet capacity of the WVDOH standard curb opening inlets in a sag location. Based on the opening height, the depth limits for weir and orifice flow for the standard curb-opening inlets overlap; therefore, the transitional flow line was created using the same method as that of grate inlets.

Chart 5-22
Type D Curb-Opening Inlet in a Sag Location

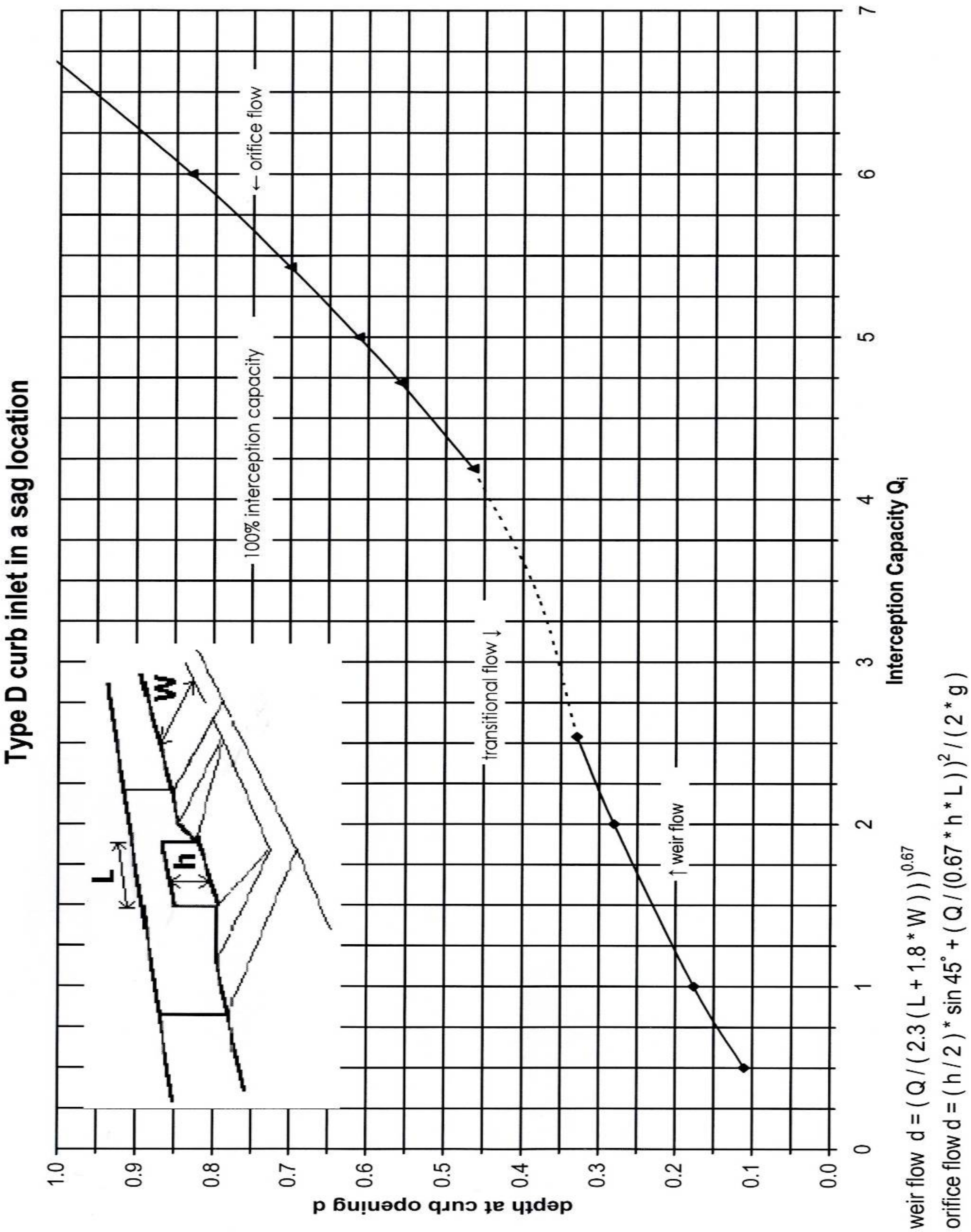


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Chart 5-23

Type E (6', 8') Curb-Opening Inlet in a Sag Location

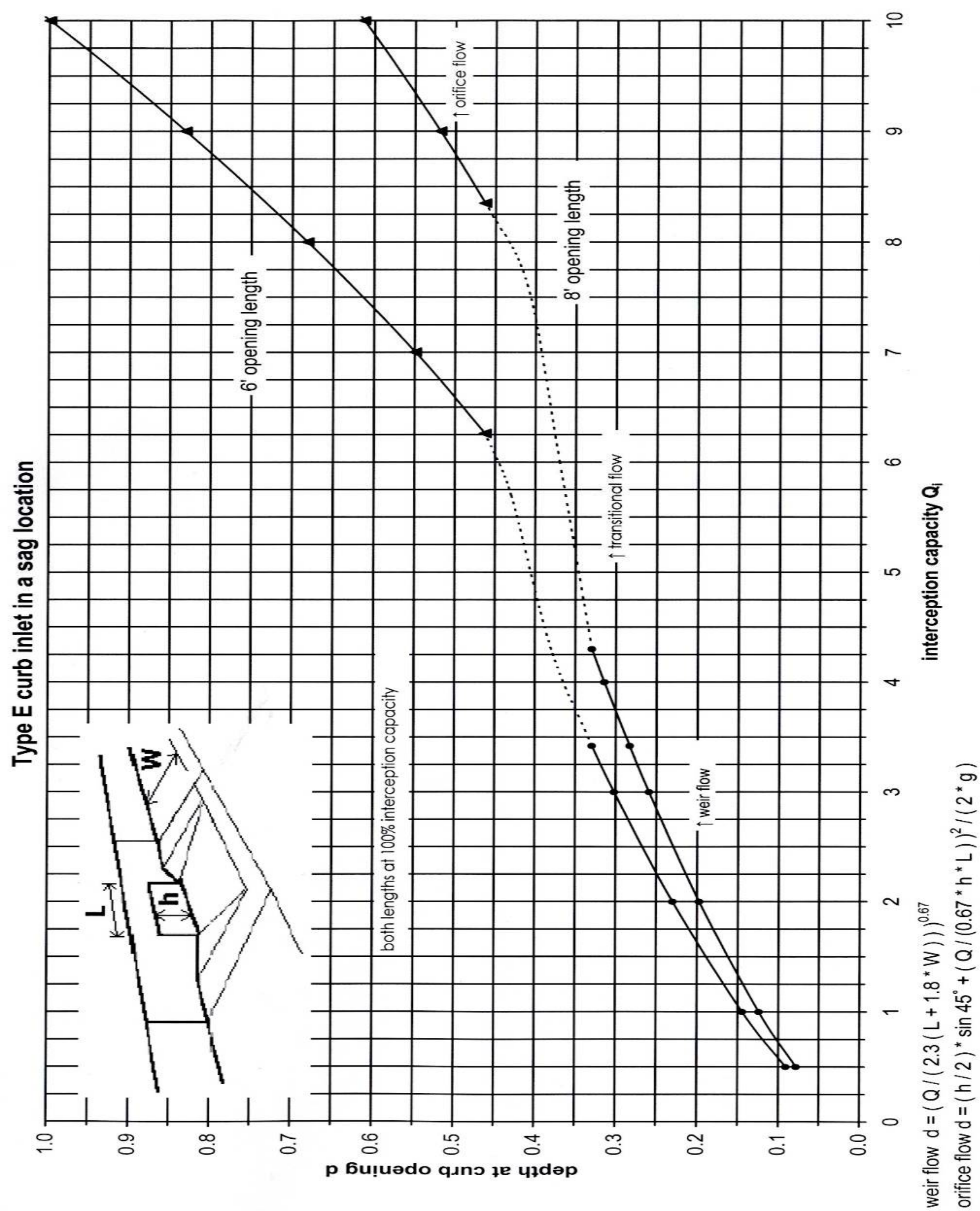

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Chart 5-24

Type E (10', 12') Curb-Opening Inlet in a Sag Location

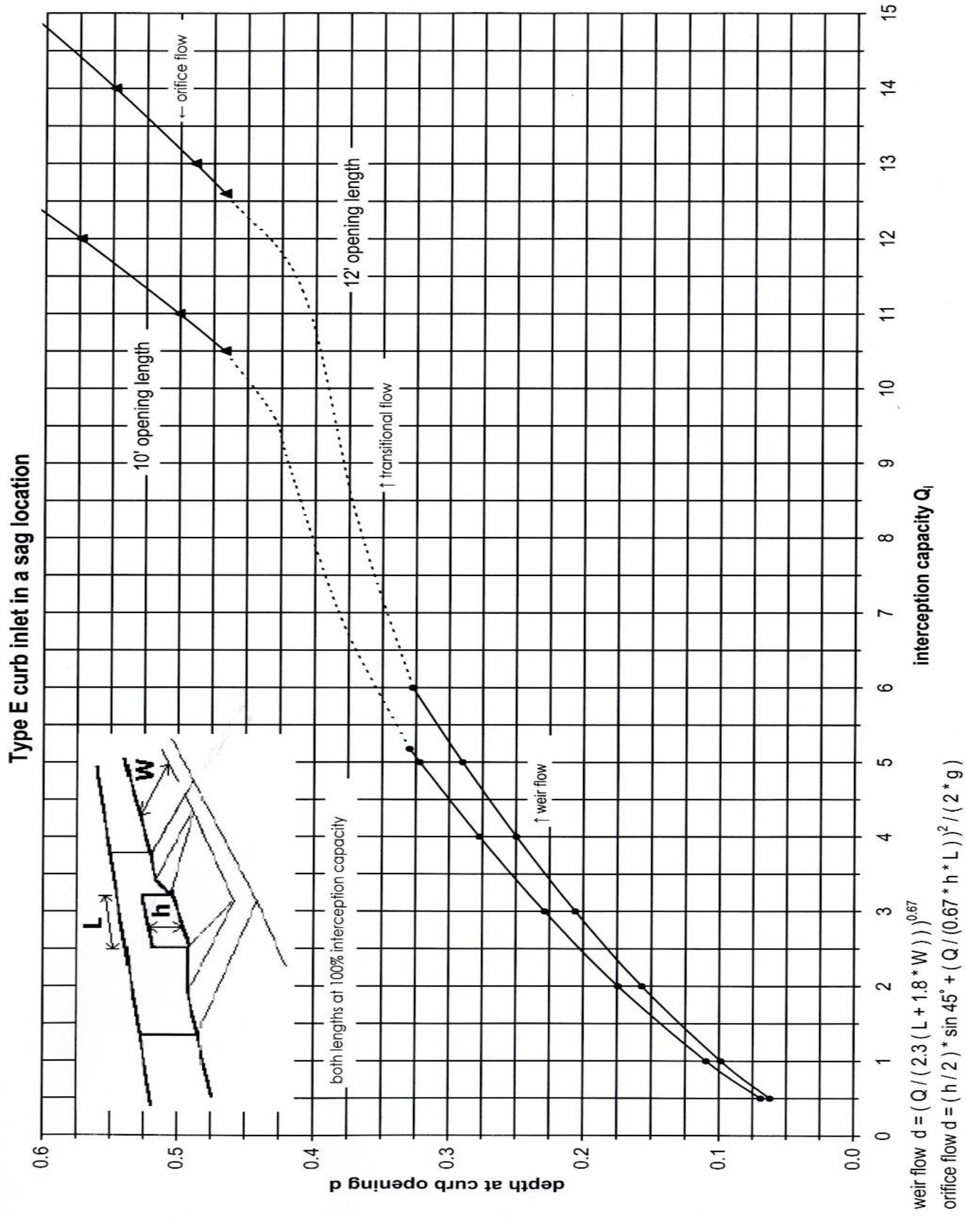

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Chart 5-25

Type E (14', 16') Curb-Opening Inlet in a Sag Location

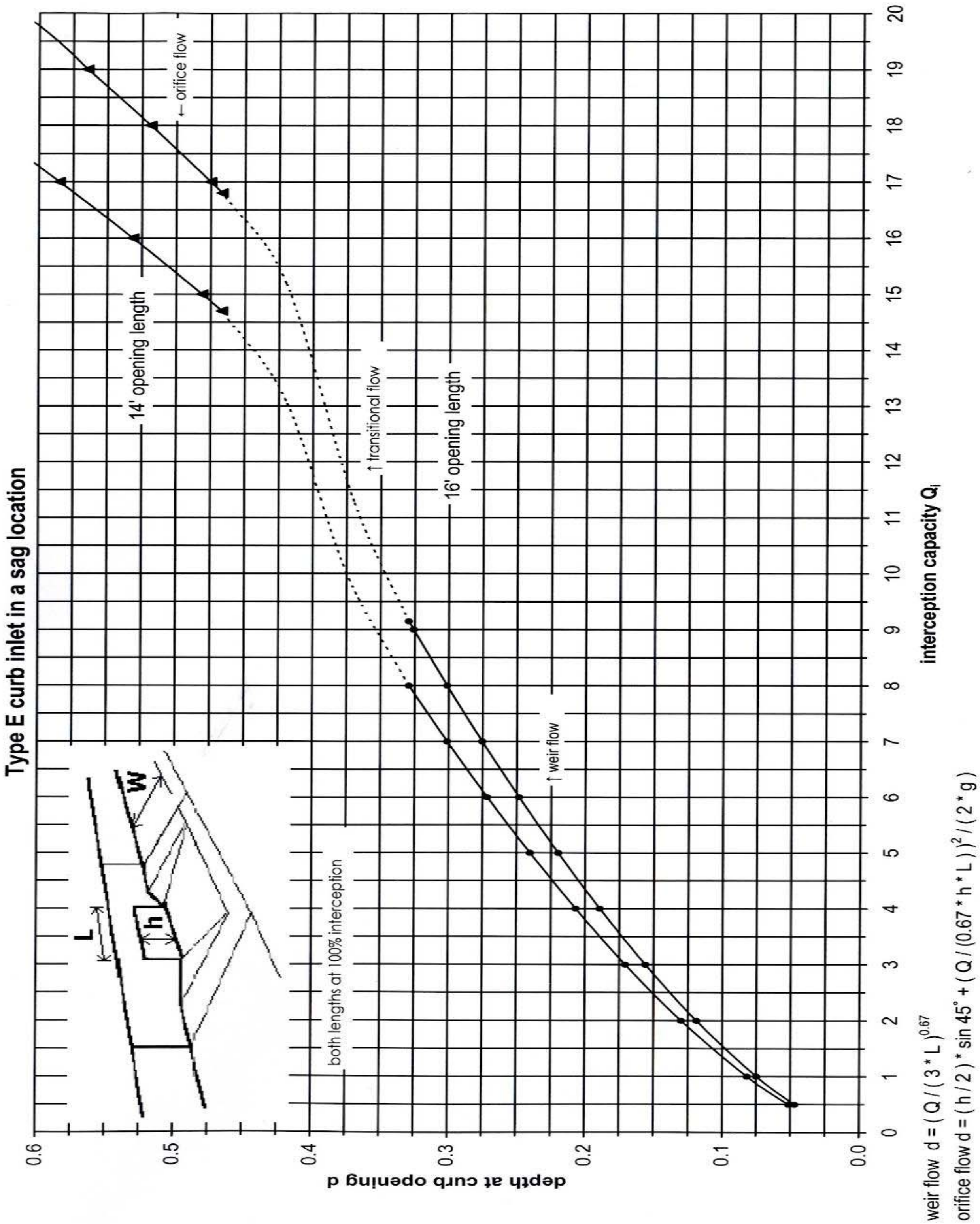

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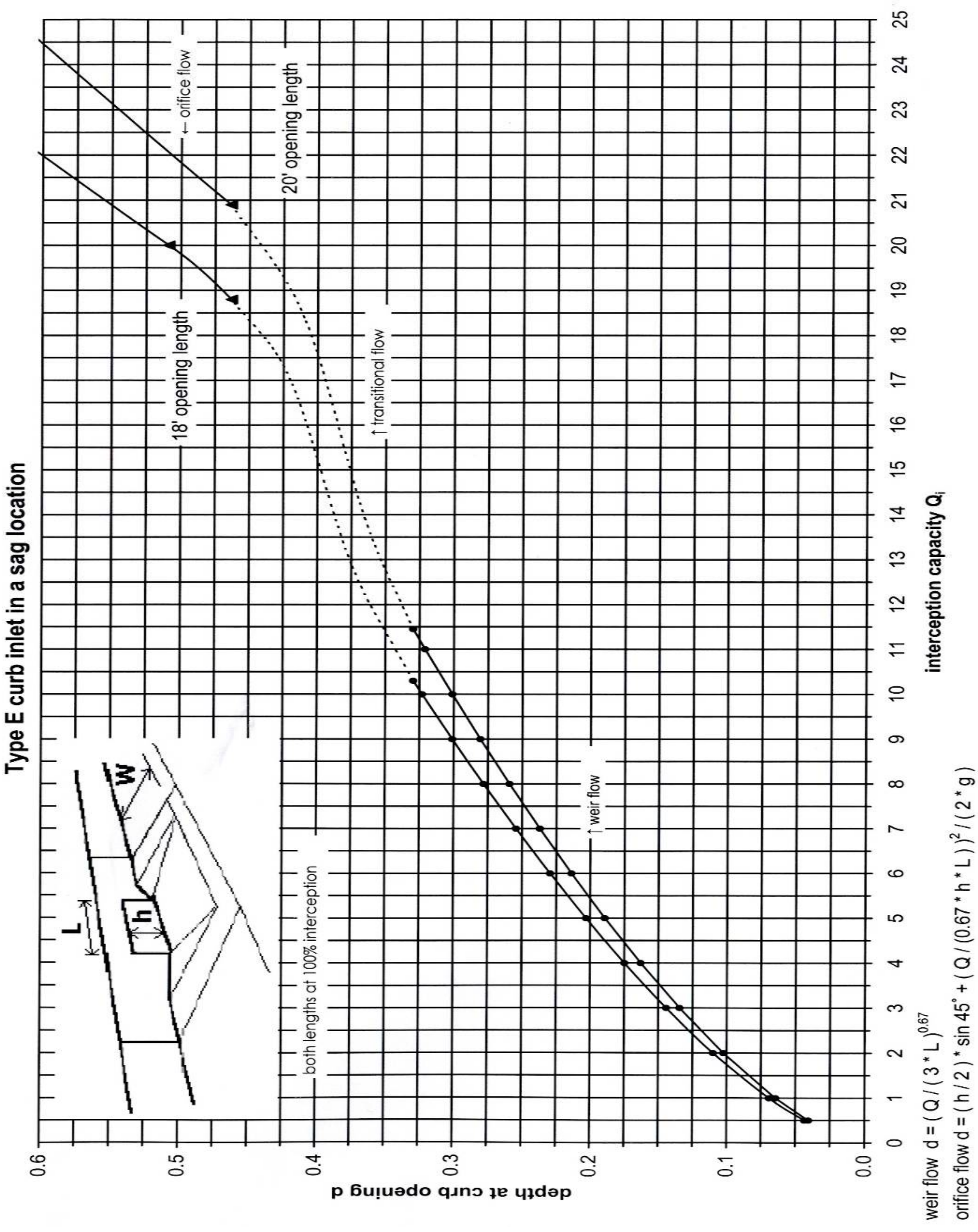
Chart 5-26

Type E (18', 20') Curb-Opening Inlet in a Sag Location



Created by the WVDOH Hydraulic and Drainage Unit

Table 5-5

Transitional Flow Line Equations for Type D & E Inlets

Line 100% capacity	Weir flow limit depth	Depth at the curb vs. Interception Capacity	Orifice flow limit depth
D	0.33	$d = 0.0295Q^2 - 0.1160Q + 0.4335$	0.47
E 6' opening	0.33	$d = 0.0073Q^3 - 0.108Q^2 + 0.5621Q - 0.6228$	0.47
E 8' opening	0.33	$d = 0.0047Q^3 - 0.0831Q^2 + 0.5051Q - 0.6771$	0.47
E 10' opening	0.33	$d = 0.0014Q^3 - 0.0318Q^2 + 0.2643Q - 0.3742$	0.47
E 12' opening	0.33	$d = 0.0012Q^3 - 0.0321Q^2 + 0.2895Q - 0.516$	0.47
E 14' opening	0.33	$d = 0.0007Q^3 - 0.0238Q^2 + 0.2777Q - 0.7328$	0.47
E 16' opening	0.33	$d = 0.0005Q^3 - 0.0196Q^2 + 0.2556Q - 0.7672$	0.47
E 18' opening	0.33	$d = 0.0004Q^3 - 0.016Q^2 + 0.2369Q - 0.8229$	0.47
E 20' opening	0.33	$d = 0.0002Q^3 - 0.0111Q^2 + 0.1806Q - 0.6394$	0.47

5.3.4.9 COMBINATION INLET CAPACITY IN A SAG LOCATION

The interception capacity of the curb opening and grate combination is essentially equal to that of the grate alone in weir flow. In orifice flow, the capacity is equal to that of the grate plus that of the curb opening. For orifice flow, the capacity of the grate depends on the average depth over the grate and the capacity of the curb opening depends on the projected water depth at the curb. In order to graph depth vs. capacity the depth must be common to both types of flow. For this manual the depth at the curb will be put in terms of the average depth over the grate. The use of this depth will require formulation of graphs for specific gutter widths and gutter slopes.

$$d_1 = d_2 + S_X\, W + a \qquad \text{(see Figure 5-1)}$$

Where: d_1 = depth at the curb, ft

d_2 = depth at the break in cross slope, ft

$$a = S_W\, W - S_X\, W \quad \therefore \quad d_1 = d_2 + S_W\, W \quad or \quad d_2 = d_1 - S_W\, W$$

Therefore the average depth over the grate (between d_1 and d_2) in terms of the depth at the curb (d_1) becomes:

$$d_{av} = d_1 - \frac{1}{2} S_W \, W \quad \therefore \quad d_1 = d_{av} + \frac{1}{2} S_W \, W$$

WVDOH standard combination inlets have a vertical throat with a throat width of 4 inches. Due to the vertical throat, the equation for orifice flow in a sag location for the curb-opening component does not depend on an angle of inclination. Substituting for the projected depth at the curb in the curb opening orifice flow equation gives the equation in terms of the average depth over the grate.

$$Q_i = 0.67 \, h \, L \left(2 \, g \, d_{av} + \frac{1}{2} S_W \, W \right)^{0.5}$$

Where: h = height of curb opening, ft

L = length of curb opening, ft

g = gravitational acceleration, 32.2 ft/s^2

S_W = cross slope of gutter, ft/ft

W = width of gutter, ft

This results in the equation for orifice flow for WVDOH standard combination inlets in terms of the average depth over the grate as:

$$Q_i = 0.67 \, A \left(2 \, g \, d_{av} \right)^{0.5} + 0.67 \, h \, L \left(2 \, g \, d_{av} + \frac{1}{2} S_W \, W \right)^{0.5}$$

Where: A = effective clear opening area of the grate, ft^2

h = height of curb opening, ft

L = length of curb opening, ft

g = gravitational acceleration, 32.2 ft/s^2

S_W = cross slope of gutter, ft/ft

W = width of gutter, ft

The limits for weir and orifice flow are determined in the same manner as that for the grate inlet capacity in a sag location (see Section 5.3.4.7). Chart 5-27 and Chart 5-28 can be used to determine the inlet capacity of standard WVDOH combination inlets in a sag location. Values for the capacity for other composite gutter sections may be obtained by using the equations.

Chart 5-27

Type F (2' wide, 6% gutter) Combination Inlet in a Sag Location

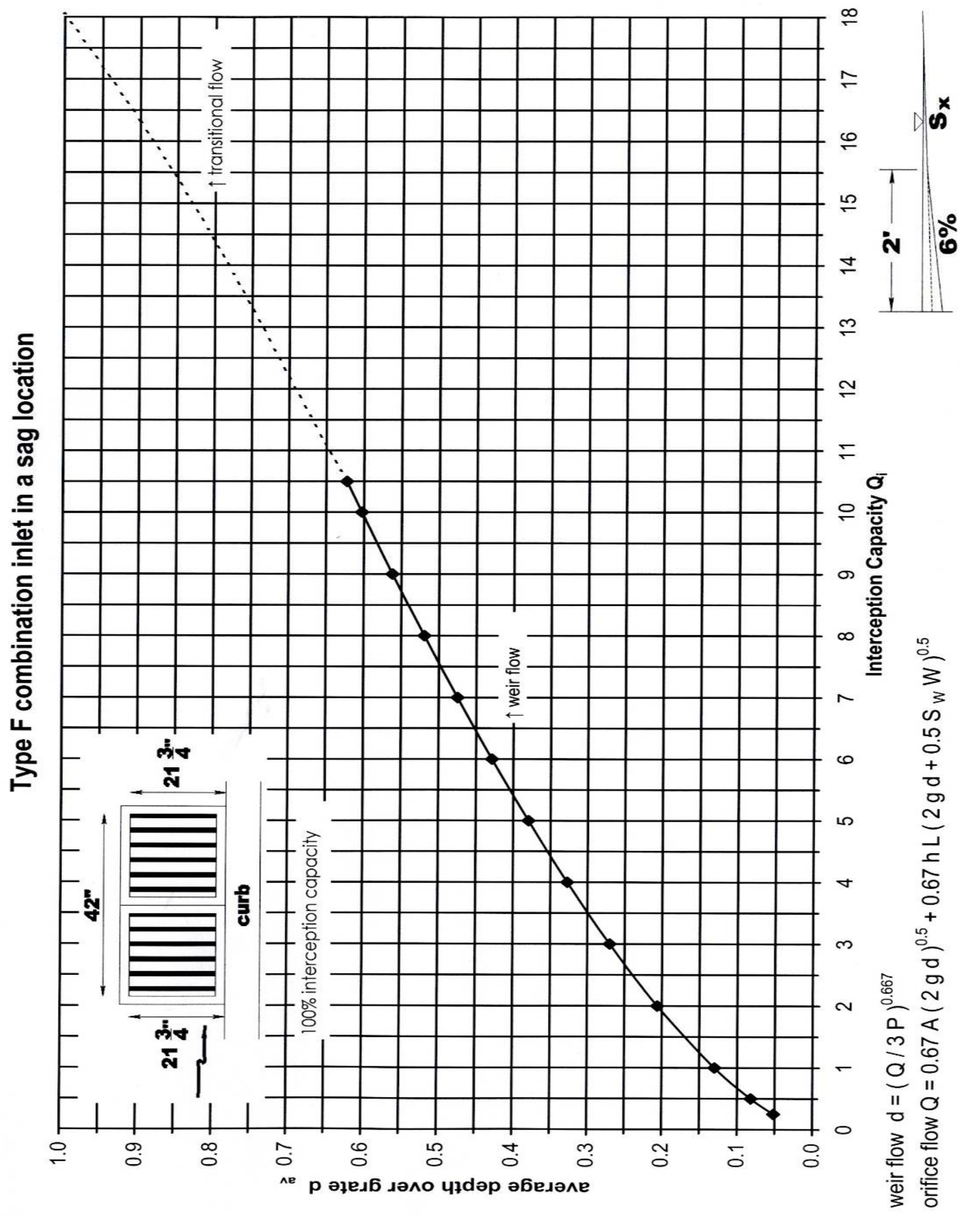


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Chart 5-28

Type H (2’ wide, 6% gutter) Combination Inlet in a Sag Location

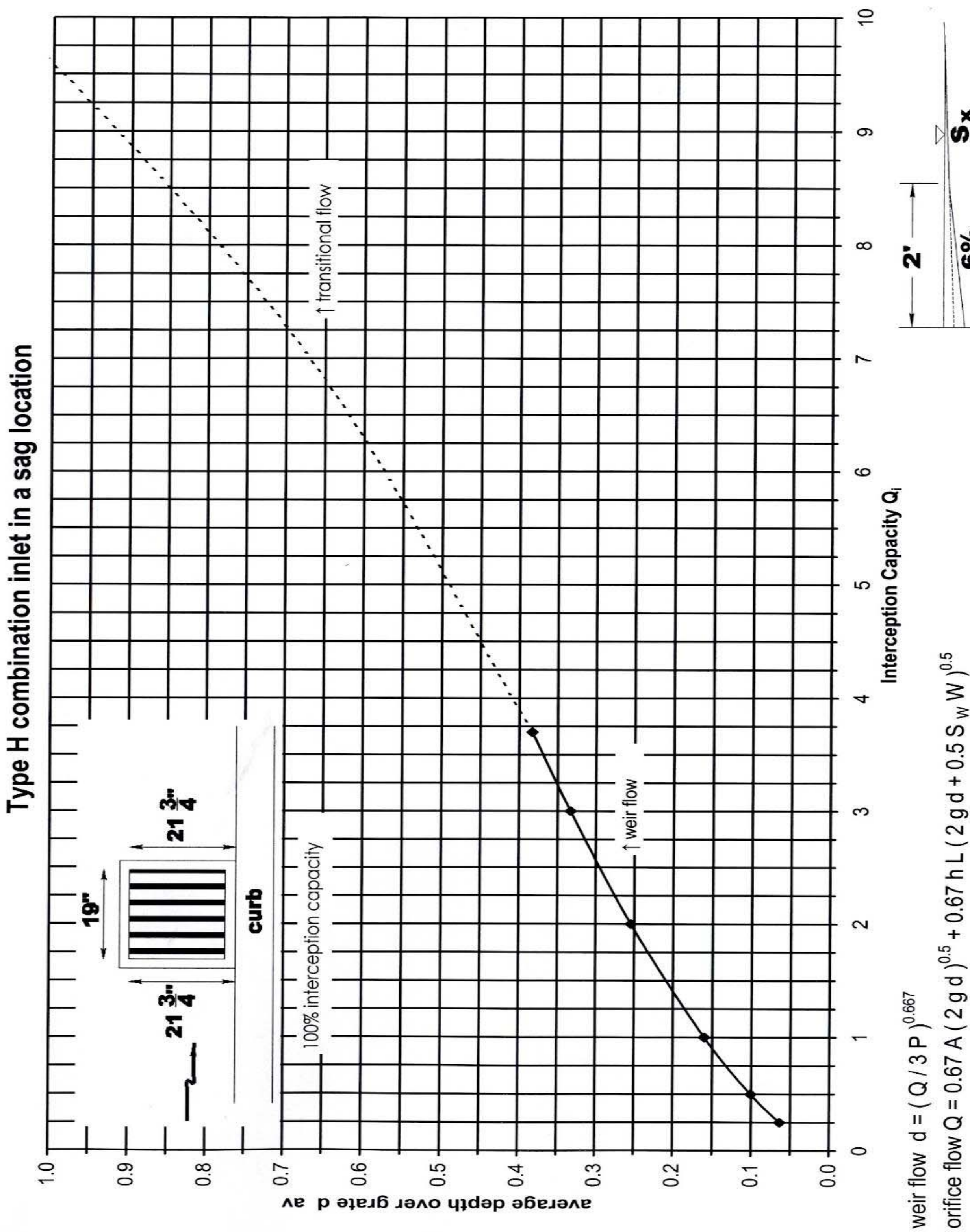

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Table 5-6

Transitional Flow Line Equations for Type F & H Inlets

Line 100% capacity	Weir flow limit depth	Depth at the curb vs. Interception Capacity	Orifice flow limit depth
F 100% capacity	0.62	$d = 0.0012Q^2 + 0.0146Q + 0.335$	1.79
H 100% capacity	0.38	$d = 0.0044Q^2 + 0.0462Q + 0.1475$	1.59

5.3.4.10 FLANKING INLETS AT SAG LOCATION

To properly drain sag vertical curves where significant ponding may occur, flanking inlets should be placed on each side of the inlet located at the low point of the sag. This is important when there is no outlet from a depressed area except through the storm drain system. The flanking inlets should be located so that they will receive all of the flow when the sag inlet is 100% clogged. They should do this without exceeding the design spread at the low point of the sag. Other inlets may be required between the flanking inlets and the sag inlet location. The flanking inlets are used as a safety factor and should not be included in the spacing design to intercept flow.

The design of a sag inlet begins by determining the depth for the allowable design spread. The allowable spread almost always results in weir flow into the inlets. It is important to define the depth common to the sag and the flanking inlets. In most cases this depth is the average depth over a grate since in weir flow the grate usually provides the flow capacity. The definition changes to the depth at the curb if a curb opening inlet is used as the sag or flanker inlet. If a mixture of a combination inlet at the sag and a curb opening inlet as a flanker is used the depth should be defined at the curb. This means the equation for the capacity of the combination inlet at the sag should be changed to use the depth at the curb instead of the average depth.

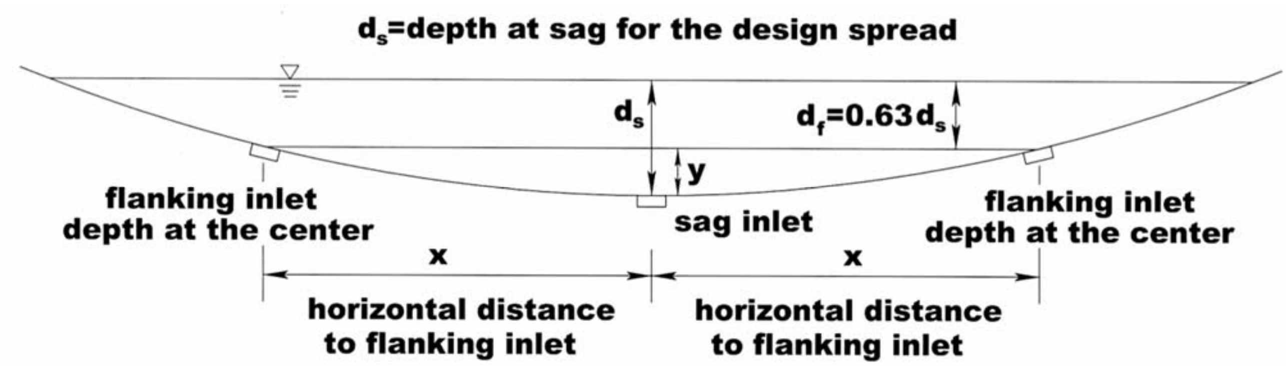
Once the capacity for the sag inlet is determined from the defined depth, half of this capacity is the amount the flanking inlet must accept. The depth over the flanking inlet for half of the sag inlet capacity is determined using the weir flow equation for the sag and flanking inlet. If the flanking inlets have the same effective perimeter as the sag inlet, the depth at the flanking inlet (d_f) is 0.63 times the depth as the sag inlet (d_s, see Figure 5-6). For example, this would be the case for the following sag and flanking inlet combinations:

- The same inlet type (and length for E's) at the sag and the flanking positions (A, B, C, D, E, F, H).

- Type F at the sag with Type A inlets at the flanking positions.
- Type H at the sag with Type B inlets at the flanking positions.

Figure 5-6

Flanking Inlets with Same Effective Perimeter as the Sag Inlet



The following illustrates how the factor is determined if the flanking inlets are the same as the sag inlet:

$$Q_i = 3 P d^{1.5}$$

Where: P = effective perimeter of grate (P_f – flanking, P_s – sag), ft

d = average depth across the grate, ft

$$Q_f = Q_s$$

$$3 P_f d_f^{1.5} = \frac{1}{2} \times 3 P_s d_s^{1.5}$$

$$d_f = \left(\frac{1}{2}\right)^{\frac{1}{1.5}} d_s$$

$$d_f = 0.63 d_s$$

Once the depths over the sag and flanking inlets are known, the vertical distance from the low point in the sag vertical curve to the flanking inlet location (y) can be determined. The horizontal distance to the flanking inlet from the sag inlet is then calculated by the following equation:

$$x = \sqrt{200 \ y \ K}$$

Where: y = vertical distance from the lowest point in the sag to the flanking inlet ($d_s - d_f$), ft

K = is the rate of vertical curvature, ft / %

If the units for K are defined as ft / ft/ft then the equation becomes:

$$x = \sqrt{2\ y\ K}$$

Table 5-7 shows the horizontal distance (x) to the flanking inlets if they have the same effective perimeter as the sag inlet. This distance is provided for various depths of ponding over the sag inlet (d_S) and various rates of vertical curvature (K) in ft/%.

AASHTO policy notes when considering the effects of drainage in a vertical curve design a minimum grade of 0.30 percent should be provided within 50 feet of the level point. This criterion corresponds to a K of 167 feet per percent change in grade.

Table 5-7

Flanking with the same Effective Perimeter as the Sag

Horizontal Distance (x) From the Low Point of a Sag Curve To Flanking Inlets Depth defined as the average depth or depth at the curb										
	K (ft/%)									
d_s (ft)	20	30	40	50	70	90	110	130	160	167
0.05	8.6	10.5	12.2	13.6	16.1	18.2	20.2	21.9	24.3	24.9
0.10	12.2	14.9	17.2	19.2	22.8	25.8	28.5	31.0	34.4	35.2
0.15	14.9	18.2	21.1	23.6	27.9	31.6	34.9	38.0	42.1	43.1
0.20	17.2	21.1	24.3	27.2	32.2	36.5	40.3	43.9	48.7	49.7
0.25	19.2	23.6	27.2	30.4	36.0	40.8	45.1	49.0	54.4	55.6
0.30	21.1	25.8	29.8	33.3	39.4	44.7	49.4	53.7	59.6	60.9
0.35	22.8	27.9	32.2	36.0	42.6	48.3	53.4	58.0	64.4	65.8
0.40	24.3	29.8	34.4	38.5	45.5	51.6	57.1	62.0	68.8	70.3
0.45	25.8	31.6	36.5	40.8	48.3	54.7	60.5	65.8	73.0	74.6
0.50	27.2	33.3	38.5	43.0	50.9	57.7	63.8	69.4	76.9	78.6

$$y = d_S - d_f \quad or \quad d_S - 0.63\, d_S$$

If the flanking inlets do not have the same effective perimeter as the sag inlet, another factor for the depth at the sag (d_S) is used to determine the depth at the flanking inlet (d_f). For example, this would be the case for the following sag and flanking inlet combinations:

- A type F or type A at the sag with type B's at the flanking positions.
- A type F or type A at the sag with type C's at the flanking positions.

- A type H at the sag with type C's at the flanking positions.

Table 5-8 and Table 5-9 are for inlets that do not have the same effective perimeter as the sag inlet. Horizontal distances for other combinations between the sag and flankers can be determined using the weir flow equation with the respective effective perimeters. The designer should be careful how the depth is defined when determining the distance between the sag and flanker inlets. This is especially true if curb opening inlets are used to flank a grate or combination inlet at the sag as the weir perimeter is much different for the curb opening inlets. The depth at the flanking inlet is taken at the middle of its longitudinal dimension. In the case of a curb opening inlet as a flanker, it may have a much longer longitudinal dimension than the sag inlet.

Table 5-8

Type F or A Inlet in the Sag with Type B Flanking

Horizontal Distance (x) From the Low Point of a Sag Curve To Flanking Inlets Depth defined as the average depth										
	K (ft/%)									
d_s (ft)	20	30	40	50	70	90	110	130	160	167
0.05	6.6	8.1	9.4	10.5	12.4	14.1	15.6	16.9	18.8	19.2
0.10	9.4	11.5	13.3	14.8	17.5	19.9	22.0	23.9	26.5	27.1
0.15	11.5	14.1	16.2	18.2	21.5	24.4	26.9	29.3	32.5	33.2
0.20	13.3	16.2	18.8	21.0	24.8	28.1	31.1	33.8	37.5	38.3
0.25	14.8	18.2	21.0	23.5	27.7	31.5	34.8	37.8	42.0	42.9
0.30	16.2	19.9	23.0	25.7	30.4	34.5	38.1	41.4	46.0	47.0
0.35	17.5	21.5	24.8	27.7	32.8	37.2	41.2	44.7	49.6	50.7
0.40	18.8	23.0	26.5	29.7	35.1	39.8	44.0	47.8	53.1	54.2
0.45	19.9	24.4	28.1	31.5	37.2	42.2	46.7	50.7	56.3	57.5
0.50	21.0	25.7	29.7	33.2	39.2	44.5	49.2	53.5	59.3	60.6

$$y = d_S - d_f \quad or \quad d_S - 0.78\, d_S$$

Table 5-9

Type F or A Inlet in the Sag with Type C Flanking

Horizontal Distance (x) From the Low Point of a Sag Curve To Flanking Inlets Depth defined as the average depth										
	K (ft/%)									
d_s (ft)	20	30	40	50	70	90	110	130	160	167
0.05	9.7	11.9	13.7	15.3	18.1	20.6	22.7	24.7	27.4	28.0
0.10	13.7	16.8	19.4	21.7	25.7	29.1	32.2	35.0	38.8	39.6
0.15	16.8	20.6	23.7	26.6	31.4	35.6	39.4	42.8	47.5	48.5
0.20	19.4	23.7	27.4	30.7	36.3	41.1	45.5	49.4	54.8	56.0
0.25	21.7	26.6	30.7	34.3	40.6	46.0	50.8	55.3	61.3	62.6
0.30	23.7	29.1	33.6	37.5	44.4	50.4	55.7	60.5	67.2	68.6
0.35	25.7	31.4	36.3	40.6	48.0	54.4	60.2	65.4	72.6	74.1
0.40	27.4	33.6	38.8	43.4	51.3	58.2	64.3	69.9	77.6	79.2
0.45	29.1	35.6	41.1	46.0	54.4	61.7	68.2	74.2	82.3	84.0
0.50	30.7	37.5	43.4	48.5	57.4	65.0	71.9	78.2	86.7	88.6

$$y = d_S - d_f \quad or \quad d_S - 0.53\, d_S$$

5.3.5 ACCESS STRUCTURES / MANHOLES

Manholes are utilized to provide access to continuous underground storm drains for inspection and cleanout. Where feasible, grate inlets may be used for access in lieu of manholes for the benefit of additional storm-water interception at minimal additional cost. Typical locations where manholes should be specified are:

- where two or more storm drains converge;
- at intermediate points along tangent sections;
- at a change in pipe size;
- where a change in alignment in excess of 10 degrees occurs; and
- where a change in grade in excess of 15 percent occurs.

All measures should be taken to avoid the placement of manholes within traffic lanes. When it is impossible to avoid locating a manhole in a traffic lane, care should be taken to ensure it is not in the normal wheel path.

5.3.5.1 HEIGHT

Pre-cast manhole sections are available for installation of pipes up to 60 inches in diameter. Larger sizes are limited by the pre-cast segment height of 8 feet. Manholes with an installation of larger pipe sizes will require field construction (cast-in-place or masonry), tee structures or a special pre-cast design. Special details must be shown in the contract plans.

5.3.5.2 SPACING

The maximum spacing of access structures whether manholes or inlets should not be exceeded. See Section 5.2.7.

5.3.6 STORM DRAINS

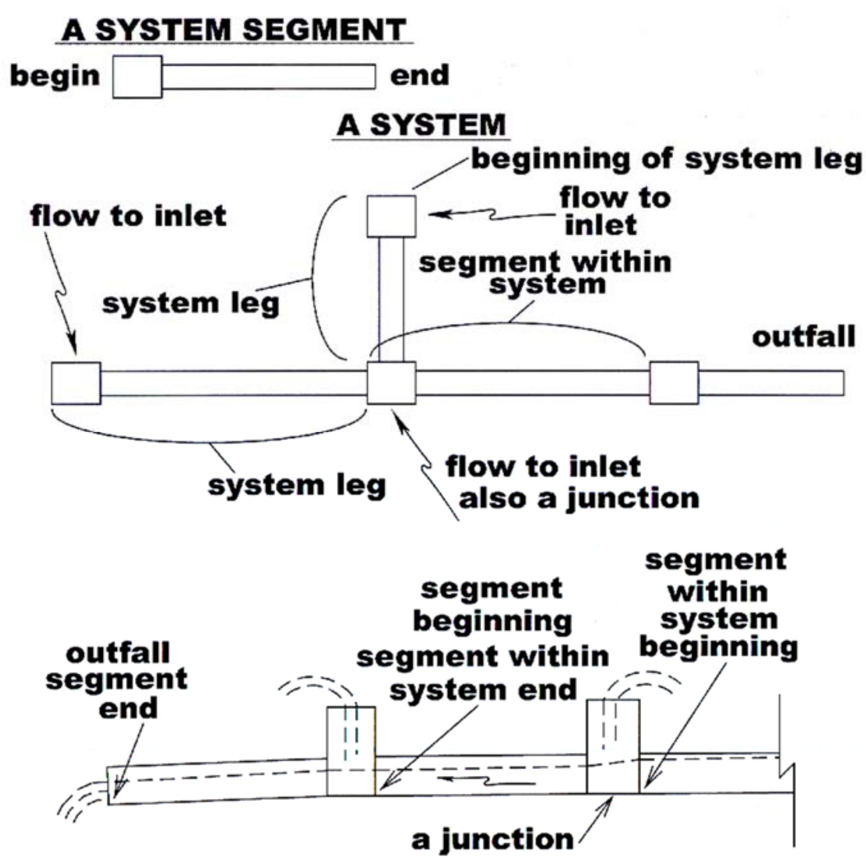
A storm drain is that portion of the highway drainage system which receives surface water through inlets and conveys the water through conduits to an outfall. A storm drain system generally discharges to a single outfall location. It may be a closed-conduit, open-conduit, or some combination of the two.

5.3.6.1 HYDRAULIC DESIGN

A chain of inlets connected by pipes is referred to as a system. Each segment in the system consists of an inlet and the pipe leaving that inlet. A leg in the system begins with an inlet and ends in a junction.

Figure 5-7

Hydraulic Design



After the preliminary locations of inlets, connecting pipes, and outfalls are placed, the size, slope, and amount of conveyed discharge is determined for each pipe in each segment of the system. This determination is made starting at the upstream segment proceeding downstream to the system outfall. The grade of the storm drain should approximate the roadway grade if the system parallels the roadway. When two segments or legs in a system converge at an inlet or access structure (junction) it is preferable to match the crown elevations of the inflow and outflow pipes. If the inflow and outflow inverts are the same and the accumulated inflow, or the reduction of slope for the outflow pipe should require an increase in the size of the outflow pipe then the flow entering the junction must increase in depth to fill the outflow pipe. The head loss in the junction may be more than the size increase for the outflow pipe because the inflow is slowed (loss of velocity head) in order to build up the required head for outflow. The result is a higher hydraulic grade line in the junction and possible siltation within the junction. Matching the crown elevations is more efficient because the design of the junction takes advantage of available head instead of the possibility of requiring additional head for the outflow. If an increase in the size of the outflow pipe is not necessary, the outflow pipe should be designed at least 0.1 feet lower than the lowest inflow pipe invert elevation to ensure flow through the junction after the construction of the system.

In most cases the Rational Method should be used to determine the conveyed discharge in each segment of the storm drain system. The rate of discharge at any point in the system is not equal to the sum of the flow rates of all the segments upstream of that point. For example, a fundamental rule of the Rational Method is that it assumes uniform rainfall intensity over the entire watershed. If the discharge rates were summed to the design point in a system and those rates where based on different time of concentrations, the rainfall intensities would be different for each rate. The flow path having the longest time of concentration to the design point will define the duration used to select the rainfall intensity value for the Rational Method. It is necessary to compute the incremental travel time through the system and accumulate this time as part of the flow path. Travel time through each segment of conduit is computed using the uniform flow velocity within the pipe of each segment. Refer to Chapter 4, Hydrology, to determine the rainfall intensity for the time of concentration.

In the case of a storm sewer system hydraulic design, the minimum T_c shall not apply. Since the travel time is summed as a design progresses through a system, applying the minimum of 5 minutes at the beginning artificially lowers the rainfall intensity when determining the overall time of concentration at the outlet.

Exceptions to the general application of the Rational Equation do exist. For example, a small relatively impervious area within a larger drainage area may have an independent discharge higher than that of the total area. This anomaly may occur because of a high runoff coefficient (C value) and high intensity resulting from a shorter time of concentration.

The design of a storm drain system should be based on open channel flow principles. The pipe should flow at a depth equal to 0.8 times the pipe diameter or equivalent at the design discharge. In situations where a substantial advantage in economics or constructability can be achieved, pressure flow may be allowed at the discretion of the Engineer. The hydraulic grade line methodology described in Sections 5.3.6.4 should be used to analyze pressure flow in storm drains. Manning's Equation is recommended for determining the initial size or capacity of the conduit. Hydraulic grade line calculations are then made to check the effects of tailwater conditions and energy losses through the system.

Refer to Section 5.2.3 for the design frequency of storm drain conduits.

5.3.6.2 HYDRAULIC CAPACITY

Manning's Equation is expressed by the following equation:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

In terms of discharge, the above formula becomes:

$$Q = VA = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

For storm drains flowing full, the above equations become:

$$V = \frac{0.059}{n} D^{2/3} S^{1/2} \qquad Q = \frac{0.463}{n} D^{8/3} S^{1/2}$$

Where: V = Mean velocity of flow, ft/s

S = Slope of the energy grade line, ft/ft

R = Hydraulic radius, ft

(flow area divided by the wetted perimeter *(A/P)*

n = Manning's roughness coefficient

P = Wetted perimeter, ft

Q = Discharge, ft^3/s

A = Cross-sectional area of flow, ft^2

D = Diameter of pipe, ft

Chart 5-30 can be used for determining the hydraulic capacity of storm drains flowing partially full. Chart 5-31 illustrates a sample calculation.

The Manning's Roughness coefficient for various pipe materials should be based on the recommendations in the latest version of Design Directive-503 (DD-503), Design of Alternate Pipe Materials, published by the West Virginia Division of Highways.

5.3.6.3 *MINIMUM VELOCITY, MINIMUM GRADES*

Minimum grades for storm drains should be such that the flow velocity is not less than 2 feet per second when the pipe is 15% full. For very flat grades the general practice is to design a system so that flow velocities will increase progressively throughout its length. The system should be checked to be sure there is sufficient velocity in all of the segments to prevent settling of particles.

Slopes that result in uniform flow velocities in excess of 10 feet per second for the design storm should be avoided due to the potential for abrasion. Concrete pipes on slopes in excess of 10 percent are not preferred due to the need for anchor blocks. In steeper terrain, large elevation differences can be accommodated by using access structures.

5.3.6.4 *HYDRAULIC GRADE LINE*

The hydraulic grade line (HGL) aids the designer in determining the acceptability of the proposed system by establishing the elevations to which the water will rise when the system is operating at the design discharge.

For a storm drain conduit flowing partially full under open channel flow conditions, the HGL is a line coinciding with the level of flowing water at any point in the conduit. For a storm drain conduit flowing full under pressure, the HGL will be above the crown of the conduit. Storm drain systems can alternate between pressure and open channel flow conditions from one section to another. Storm drains designed to operate under pressure flow conditions can cause surcharging and manhole lid displacement if the hydraulic grade line rises above the ground surface. A design should include a review with a storm event larger than the design storm which pressurizes the system. This larger storm is known as the check storm (see Chapter 4, Section 4.3.2) and it shall not be detrimental to nearby property. For example, surcharging due to pressurization from the check storm may exceed the allowable spread and close a roadway, but it should not overtop the curb and damage nearby property. Possible detrimental impacts to nearby property may require a more detailed design analysis and right of way take or easement acquisition.

Detailed HGL calculations usually begin with the tailwater elevations at the system outfall. If the proposed outfall is an existing storm drainage system, the HGL calculation must begin at the outlet end of the existing system.

5.3.6.5 TAILWATER

Evaluation of the hydraulic grade line begins at the system outfall with the tailwater elevation. If the pipe outfall is an existing stream or basin, the actual water surface elevation for the design storm should be determined as a first step. When estimating tailwater depth on the receiving stream, the designer should consider the joint or coincidental probability of two hydrologic events occurring at the same time. For the case of a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving or main stream. A short duration storm, which causes peak discharges on a small watershed, may not be critical for a large watershed. Also, it may safely be assumed that if the same storm causes peak discharges on both watersheds, the peaks will be out of phase. Table 5-10 can be used as an aid to evaluate the coincidental occurrence and joint probability.

Table 5-10

Joint Probability Analysis

Drainage Area Ratio	**Frequencies for Coincidental Occurrence**			
	10-Year Design		**100-Year Design**	
	Main Stream	**Tributary**	**Main Stream**	**Tributary**
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

This table (source: Urban Drainage Design Manual, HEC-22, FHWA 2001) provides a comparison of discharge frequencies of coincidental occurrence for a 10-year and 100-year design storm. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres, and the storm drainage system has a drainage area of 2 acres, the ratio of the receiving area to the storm drainage area is 200 to 2, which equals 100 to 1. From Table 5-10, considering a 10-year design storm occurring over both areas, the flow rate in the receiving stream will be equal to that of the 5-year storm when the drainage system flow rate reaches its 10-year flow at the outfall. Conversely, when the flow rate in the receiving stream reaches its 10-year flow rate, the flow rate from the storm drainage system will have fallen to the 5-year flow rate. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Once the flow rate is known for the receiving stream, the actual water surface elevation can be determined using Manning's equation if the receiving stream has steady, uniform flow. This water surface elevation is known as the normal depth. The tailwater is determined by comparing the observed or computed tailwater (from backwater or normal depth calculations) with the critical depth of flow at the outfall. Based on numerous calculations by the FHWA it was determined that for partly-full flow at the outfall a downstream extension of the full-flow hydraulic grade line pierces the vertical plane of the pipe outlet at a point halfway between critical depth (dc) and the top of the barrel (pipe diameter, D). This means the change in the flow state occurs within the pipe and the tailwater elevation at the outlet is (dc+D)/2 above the outlet invert. This is the value used to set the beginning of the hydraulic grade line at the outfall. If the observed or computed tailwater elevation exceeds (dc+D)/2, then it is used to set the beginning of hydraulic grade line. For more information on the tailwater elevation see Chapter 8, Section 8.4.10.

5.3.6.6 ENERGY LOSSES

Energy losses can be classified into major and minor losses. These losses are added to the elevation of the HGL at the end of a system segment to obtain the elevation of the HGL at the beginning of a system segment. Major head losses result from the energy required to overcome the friction in each segment pipe. Minor head losses are attributed to the energy required to overcome changes in momentum or turbulence at system outlets, inlet or access hole structures, transitions, and junctions.

Major head loss due to friction is a function of the pipe length and friction slope. Minor head losses are estimated as a function of the change in velocity or the velocity head. A transition is a location where the pipe or channel size changes however, these should be avoided and replaced with access holes. A junction is the connection of a lateral pipe to a larger trunkline without the use of an access hole structure. A junction loss is estimated using a form of the momentum equation.

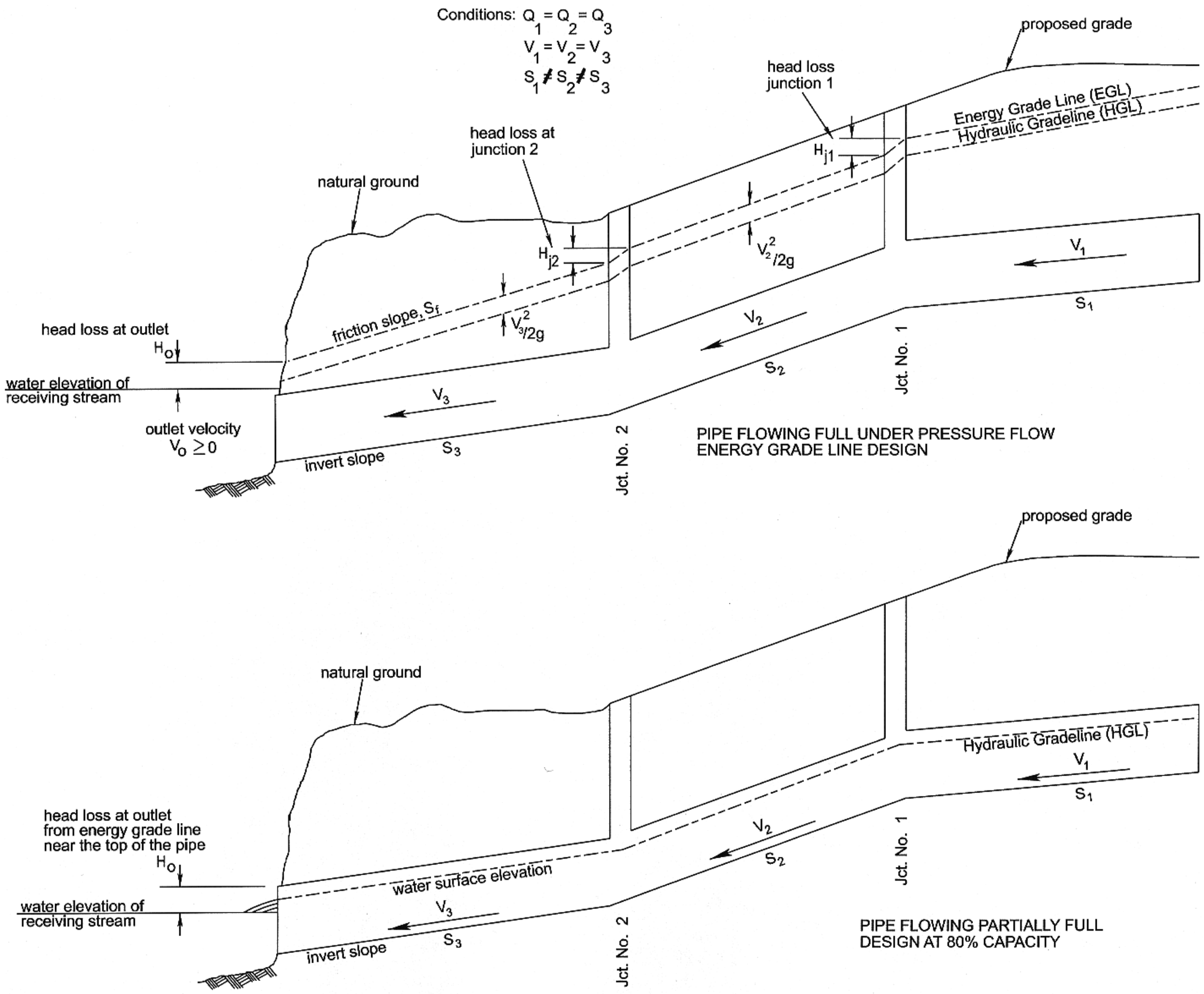
Transition and Junction losses are not covered in detail in this manual. Refer to HEC-22 for details on calculating these losses.

The design discharge and the effective pipe velocity should be used for computing the minor head losses. If the HGL is below the crown line of the conduit, partial-flow or normal velocity (based on the design discharge) should be used in computing the losses. If the HGL is above the crown line of the pipe, the full flow velocity (the design discharge divided by the cross sectional area of the conduit) should be used in computing the losses. Since it is not known whether the HGL will fall above or below the crown line of the pipe, the designer should first calculate the HGL assuming partial-flow or normal velocity. If the computed HGL is below the crown line, the assumption of normal velocity and the computed HGL is verified. If the computed HGL is above the crown line, then full flow velocity should be assumed and the HGL recalculated. Figure 5-8 illustrates energy losses and the hydraulic grade line in a storm drain system under pressure flow and in a system at an 80% design capacity.

A chain of inlets connected by pipes will be referred to as a system (see Figure 5-7). Each segment in the system consists of an inlet (segment beginning opposite of pipe connection) and the pipe leaving that inlet (segment end opposite of inlet connection). A location where more than one segment connects will be referred to as a junction. For the hydraulic design of a storm sewer, a leg in the system begins at an inlet and ends at a junction. For the determination of the hydraulic grade line in a storm sewer, a leg in the system begins at a junction and ends with an inlet. Refer to Section 5.4 and the step by step instructions for more detail.

Figure 5-8

Energy Losses in a Storm Drain System



5.3.6.7 *MAJOR LOSSES*

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is computed as follows:

$$H_f = S_f L$$

H_f = Friction loss, ft

S_f = Friction slope, ft/ft

L = Length of pipe, ft

The friction slope is also the slope of the hydraulic gradient for a particular pipe run. Since this design procedure assumes steady uniform flow in an open channel, the friction slope will match the pipe slope for part full flow. Pipe friction losses for full flow are computed as follows:

$$S_f = \frac{H_f}{L} = \left(\frac{Q\,n}{0.46\,D^{2.67}} \right)^2$$

Q = Rate of flow, ft^3/s

D = Storm drain diameter, ft

n = Manning's coefficient

5.3.6.8 MINOR LOSSES

Head loss due to flow entering and/or passing through an inlet or access hole structure is based on many factors. The method presented here for calculating minor losses is based on laboratory research by the FHWA. This method does not apply when the inflow pipe invert is above the water level (HGL) in the structure.

For the case where an inflow pipe invert is above the water level (HGL) within the structure, or the minor losses are being calculated for the first inlet in a leg of a system the outflow pipe shall be designed as a culvert. For a culvert, if this outflow pipe is flowing full or partly full under outlet control, the minor loss can be computed using the entrance loss coefficients in Chapter 8, Table 8-4. Adding this minor loss to the friction loss through the outlet pipe, the water surface elevation is obtained within the structure. For a culvert, if the outflow pipe is flowing under inlet control, the water depth in the structure should be computed using the inlet control equations or nomographs. Check the water surface elevation in the structure for both inlet and outlet control and use the higher of the two. Consult Chapter 8, Section 8.4.5 for guidance.

For cases where the inflow pipe invert is below the water level (HGL) in the access hole, the head loss is represented as being proportional to the velocity head of the segment pipe. Using K to represent the constant of proportionality, the total minor loss is approximated by the following equation:

$$H_m = K \left(\frac{V_S^2}{2\,g} \right) \qquad K = K_o\, C_d\, C_D\, C_Q\, C_P\, C_B$$

H_m = Minor loss (ft)

K = Overall minor loss coefficient

K_O = Initial head loss coefficient

C_d = Factor for flow depth (free surface flow only)

C_D = Factor for pipe diameter change (pressure flow only)

C_Q = Factor for relative flow

C_P = Plunging flow factor

C_B = Benching factor

V_S = Flow velocity in the segment pipe, ft/s

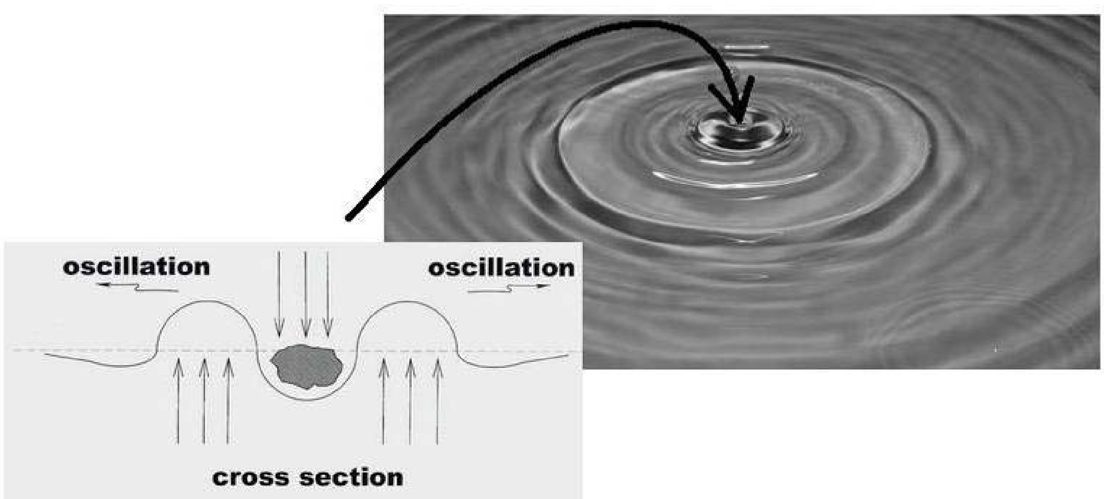
See Section 5.4 for definitions, equations, and illustrations for each of these minor loss factors.

5.3.6.9 THE STATE OF FLOW

The effect that gravity has on the state of flow is represented by a dimensionless ratio of the inertial forces to the gravitational forces. This ratio results in a number known as the Froude number (named for William Froude in 1868). This ratio may also be physically interpreted as the ratio between the average flow velocity to the speed of an elementary gravity wave (celerity) traveling over the water surface. A good example of a gravity wave is what results from throwing a rock into a lake with still water. When the surface water is displaced by the rock, gravity restores the surface toward equilibrium resulting in an oscillation about the equilibrium state. This oscillation causes waves that radiate in all directions.

Figure 5-9

Elementary Gravity Wave



Flow can exist as a subcritical state, a supercritical state, or a critical state in an open channel flow regime. Subcritical or tranquil flow occurs on mild slopes where flow is deep with a low velocity. The control section is always at the downstream end of the flow reach in this state. Supercritical flow occurs on steep slopes where the flow is shallow with a high velocity. In this state the control section is always at the upstream end of the flow reach.

The equation for the Froude number is as follows:

$$\frac{V}{\sqrt{g\ d_{md}}} \qquad d_{md} = \frac{A}{T}$$

Where: V = average velocity, ft/s

g = acceleration due to gravity, ft/s^2

d_{md} = hydraulic mean depth, ft (see section 8.4.5.1)

A = open channel flow area, ft^2

T = top width of flow, ft

$\sqrt{g\,d_{md}}$ = gravity wave celerity, ft/s

If this number is less than 1, gravitational forces are dominant and the flow is in a subcritical state. If this number is greater than 1 inertial forces are dominant and the flow is in a supercritical state. When this number is equal to one the flow is at the critical state and the hydraulic mean depth is termed the critical depth.

Major and minor head losses do not apply in a storm sewer system when the flow is in a supercritical state. This is because the effect of a disturbance in the flow cannot be transmitted upstream. For example: if the creek flowing into the lake has a velocity that is less than the wave celerity, the waves from the rock can travel upstream into the creek channel. This means the velocity of flow in the creek is in a subcritical state or the Froude number is less than one. If the creek velocity is greater than the wave celerity the waves from the rock cannot travel upstream into the channel and the creek is in a supercritical state or the Froude number is greater than one. For flow in a critical state the surface disturbances remain stationary in the flow. Another analogy would be running up an escalator moving in the downward direction. If the running person represents the gravity wave (or a disturbance in the flow), and the speed of the escalator is moving faster than the person can run, that person will not make it up the escalator.

In the case of the minor head losses, the flow must be in a supercritical state on both the inflow and outflow sides of the inlet or access hole. Comparing the pipe flow depth to the critical depth will reveal the state of flow for each pipe. Depths less than the critical depth are in a supercritical state and depths greater than the critical depth are in a subcritical state.

It is important to mention that this principle applies at the system outfall as well. The beginning of the HGL calculations will not start at the outfall if the flow state is supercritical. This is because the control section is upstream of the outfall. In this case, the HGL calculations start in the segment where subcritical flow begins and the elevation to begin with, is the critical depth.

5.3.6.10 TOTAL HEAD LOSSES

Total head loss is computed by adding the major loss due to friction to the minor losses due to each inlet or access hole structure.

$$H = H_f + H_m$$

5.3.6.11 MATERIALS AND ALTERNATES

Refer to DD-502, "Maximum Fill Height Tables," to determine appropriate pipe material, thickness and corrugation based on the depth of cover over the pipe.

See DD-503, "Design of Alternate Pipe Materials", for Manning's "n" values, and allowable pipe materials based on roadway classification, structural requirements, and corrosion and abrasion potential.

5.3.7 ECONOMIC CONSIDERATIONS

The designer should consider the following factors to get the maximum cost efficiency from the storm drain system:

- Location of utilities (under pavement or behind curb)
- Gutter widths
- Type and spacing of inlets
- Storm drain sizes and locations
- Type of curb or median
- Cross pipes from other inlets (size and depth)
- Culverts (size and depth)
- Debris

Where there are other means of maintenance access to storm drains or culverts, inlets should not be unreasonably large or deep to accommodate large pipes. Saddles and pipe stubs can be used to connect inlets to large or deep pipes at lower costs in such cases.

The cost of drainage is neither incidental nor minor on most roads. Careful attention to requirements for adequate drainage and protection of the highway from stormwater in all phases of location and design will prove to be effective in reducing costs in both construction and maintenance. Unless drainage is properly accommodated, maintenance costs will be unduly high.

5.3.8 *Storm Drainage in Karst areas*

The designer should be mindful of the outlet of flow from a storm sewer in karst areas. Outlet structures should be designed in such a way as to dissipate the flow over the largest area possible, thus creating an overland type of flow. Every attempt should be made to avoid the concentration and ponding of outlet flow in a karst area.

5.3.9 *Environmental Concerns*

Environmental concerns for storm drainage design projects generally pertain to erosion and sedimentation problems, which can be particularly visible on urban highway projects. Permanent and temporary erosion control features should be carefully designed, installed and maintained in sensitive surroundings.

Hazardous spills can be transported long distances in an enclosed storm drain system. Leaking pipe joints may also allow the spill to exfiltrate into strata containing precious groundwater supplies. The point of spill may not be a sensitive area, but the point of outfall such as a lake or river, may be sensitive. A 404 nationwide permit and a 401 water quality certification may be required if the point of outfall empties into waters of the U.S.

5.3.9.1 *Safety*

Safety of the public is always of paramount importance and is the basis for the design of all elements of a storm drainage system. Inlet design (type, location, and spacing), pavement drainage design (cross-slope, spread, curb and gutter), and storm drain design, are all predicated on safely discharging storm water to the outlet.

5.3.9.2 ADA CONSIDERATIONS

State governments are required to comply with the Americans with Disabilities Act (ADA) for the design of public facilities. Inlet type, location, and spacing should be considered with regard to the safety of pedestrians and disabled persons. Drainage inlets should be placed sufficiently away from pedestrian crosswalks so that people with disabilities do not encounter excessive water for the design event when traversing such locations.

5.3.9.3 BICYCLE SAFETY

Inlet type, location, and spacing should be considered with regard to the safety of bicyclists. Inlets should generally not be located within bicycle lanes. Water should not be allowed to spread into designated bicycle lanes. Pavement depressions to accommodate grate and curb inlets should preferably be located outside bicycle lanes to prevent bicyclists from swerving to avoid them. Grate inlets should be bicycle safe unless located on highways where bicycles are not permitted.

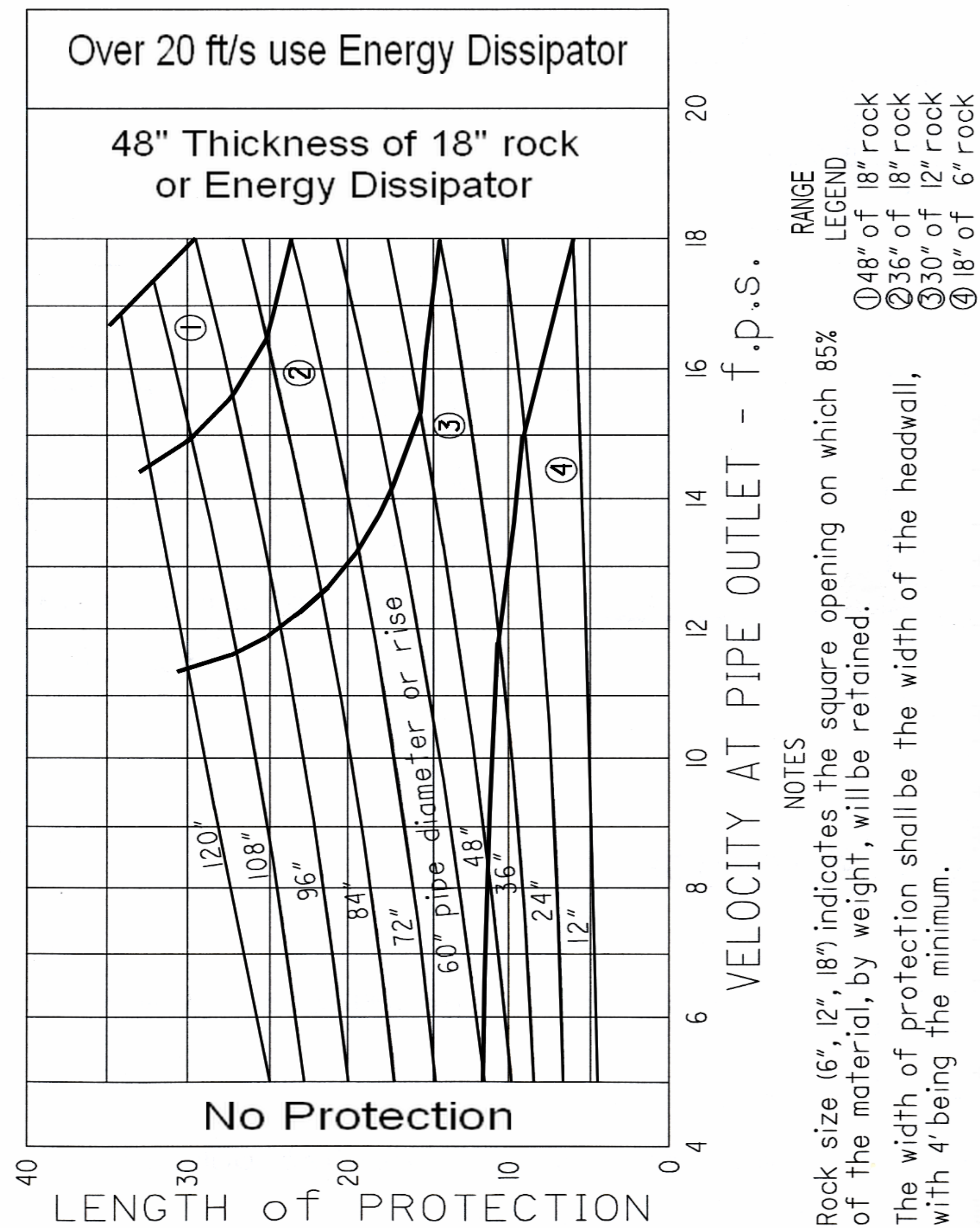
5.3.10 OUTLET PROTECTION

The three most common forms of rock lined outlet protection are dumped rock gutter, select embankment, and a rock lined scour basin. Rock lining is designated as "Dumped Rock Gutter" if the bottom width of the outlet bed is less than 8 feet. If the bottom width of the outlet bed is 8 feet or more, the rock lined protection is designated as "Select Embankment". Both dumped rock gutter and select embankment shall not be placed within a stream with a defined bed, bank, and ordinary high water markings. These linings are for protecting the outlet of a storm sewer system or culvert that drains surface runoff which does not create a defined channel. Dumped rock gutter and select embankment should be used as shown in the Standard Details Book, Volume I, Sheet DR8, Sheet 2 of 4. The length of blanket needed for dumped rock gutter or select embankment protection can be determined by the method outlined in the FHWA Hydraulic Engineering Circular 14 or the following chart provided by the Ohio Department of Transportation. This chart was formulated from an ODOT funded study through the University of Cincinnati called Design Criteria for Erosion Protection at the Outlet of Culverts by L.M. Laushey.

It is important to note that this specification for rock type differs from the WVDOH specification for dump rock gutter. The 85% requirement for the rock diameter should be explained on the general notes sheet.

Chart 5-29

Length of Blanket for Outlet Protection



Source: ODOT Location and Design Manual Vol. 2, Drainage Design, January 2007

The allowable velocities for the outlet bed material are listed in Table 5-11 (AASHTO Model Drainage Manual, 1991 Edition). V_o is the storm sewer system or culvert outlet velocity in feet/second.

- If V_o < allowable outlet bed material velocity, no protection is needed
- If V_o > allowable outlet bed material velocity, use dumped rock gutter or select embankment
- If V_o > 20 fps, use rock lined basin or energy dissipator (see HEC-14)

Table 5-11

Allowable Velocities of Streambed Material

SOIL TEXTURE	ALLOWABLE VELOCITIES (ft/s)
Fine sand and sandy loam (A-3)	2.5
Silt soils (A-4, A-5, A-6)	3.0
Silt or clayey gravel and sand (A-2)	3.5
Clayey soils (A-6, A-7)	4.0
Clay, fine gravel	5.0
Cobbles	5.5
Shale	6.0

5.4 INSTRUCTIONS, DESIGN FORMS, AND PROGRAMS

This section contains step by step procedures and computation forms for the designer's use. Nomographs, charts, computation forms, step-by-step procedures, and example problems pertaining to all aspects of storm drain system design can also be found in HEC-22. The Federal Highway Administration has developed the HYDRAIN computer program package that includes the computer program HYDRA, which can be used to check the design adequacy and analyze the performance of a storm drain system under inflow conditions. The FHWA HY-22 computer program can be used to compute gutter and inlet capacities and determine the spacing of inlets. Other programs may be used with approval from the engineer.

A Step By Step Guide to Form 5-1 INLET SPACING

STEP 1: Mark the location of inlets which are necessary without considering any specific drainage area such as the locations described in Section 5.2.5.

STEP 2: Start at a high point in the vertical alignment at one end (if possible) and work towards the sag curve or low point. Begin at the next high point and work backwards toward the same sag or low point. An inlet design for the sag curve or low point with flanking inlets will be required.

STEP 3, DRAINAGE AREA: The purpose of this step is to estimate the drainage area that will create an amount of runoff to cause the design spread to occur in the gutter, thus requiring the placement of an inlet. Since the width of the drainage area is known (typically the roadway from the curb to the centerline), the length is the parameter to determine. The closer the estimate is to the length that creates the design spread, the less iteration needed to achieve it. A good first trial length is between 300 and 500 feet. If there is a possibility of any runoff that will overtop the curb and run onto the roadway be sure to take this area into account, thus shortening your length estimate. Where practical, runoff from areas beyond the curb should be intercepted before it reaches it. Record the drainage area in column 3.

STEP 4, INLET ID: Record the location of the inlet by number and station in columns 1 and 2. Identify the curb and gutter type in the remarks column. A sketch of the gutter cross section should also be presented in the margin. If the system will be small in size the next line could be used for the sketch.

STEP 5, TIME OF CONCENTRATION: Compute the total time of concentration by calculating the sheet flow and shallow concentrated flow segments. See Chapter 4 Section 4.4.2.5 for guidance on determining T_c. Record this information in columns four, five and six. The minimum time of concentration is 5 minutes.

STEP 6, RAINFALL INTENSITY: Using the time of concentration, determine the rainfall intensity from the Intensity-Duration-Frequency curve obtained from the Precipitation Frequency Data Server website or by selecting the appropriate rainfall intensity zone for West Virginia. This will be the intensity for a <u>ten year</u> return period or recurrence interval. See Chapter 4 Section 4.4.2.7 for guidance. Record this information in column seven.

STEP 7, RUNOFF COEFFICIENT: Determine the runoff coefficient C, which represents the drainage area for the inlet. See Table 4-4 of Chapter 4 and record this value in column eight.

Step 8, FLOW: Determine the flow in the gutter using the Rational Method and record this information in column nine. See chapter four Section 4.4.2 for guidance.

STEP 9, GUTTER SECTION PROPERTIES: Record the gutter section properties in columns ten through fifteen. The longitudinal gutter slope should take into account any superelevation. If your gutter section is not a composite one, enter the same value for columns eleven and twelve. The inlet type should be noted according to those shown in the Standard Details Book Volume 1 (type A, B, C, D, E, F, or H).

STEP 10: For the first inlet in a series, enter 0 into column sixteen and the same value from column 9 into column 17 as there will not be any bypass flow.

STEP 11, GUTTER DISCHARGE: Determine the flow spread by Chart 5-4, Chart 5-5, Chart 5-6 or the equations in Figure 5-1 and enter the value into column eighteen. Determine the depth of flow at the curb by Chart 5-4, Chart 5-5, Chart 5-6 or the equations in Figure 5-1 and enter the value into column 19. Compare the calculated spread with the allowable spread (see Section 5.3.3) and the calculated depth of flow with the curb height. If the calculated spread is near the allowable spread and the depth at the curb is less than the curb height, continue to Step 12. Else, expand or contract the drainage area up to the inlet to increase or decrease the spread. Repeat Steps 3 through 11 until appropriate spread and depth at the curb values are obtained.

STEP 12A, INLET CAPACITY: Calculate the inlet capacity or intercepted flow by the inlet (Q_i) and enter the value into column 20. For grate and combination inlets on grade use Chart 5-4, Chart 5-5, Chart 5-6 or the equations in Section 5.3.4.4 to determine the frontal flow / total gutter flow ratio E_o, and the side flow / total gutter flow ratio Q_s/Q. The inlet efficiency E is determined using the frontal flow intercepted / total frontal flow ratio R_f, and the side flow intercepted / total side flow ratio R_s. These values are taken from Chart 5-7and Chart 5-8, respectively. For curb opening and slotted inlets on grade Chart 5-9, Chart 5-10, and the equations in Section 5.3.4.5 will provide the inlet efficiency. The interception flow Q_i is then equal to the efficiency E times the flow in the gutter Q for the inlet type.

STEP 12B: Determine the inlet bypass flow (Q_B) by subtracting the intercepted flow from the total gutter flow (column 17 - column 20) and enter this value into column 21. Record this value into column 16 for the next inlet location down the grade.

STEP 14, CONTINUE: Proceed with the design for the next inlet down the grade by repeating steps 3 – 13 (skipping step 10). Determine the total gutter flow for the next inlet by adding the bypass from the previous inlet to the flow in the gutter and record this value in column 17. Continue for each subsequent inlet down the grade to the low point.

Form 5-1

Inlet Spacing Design Form

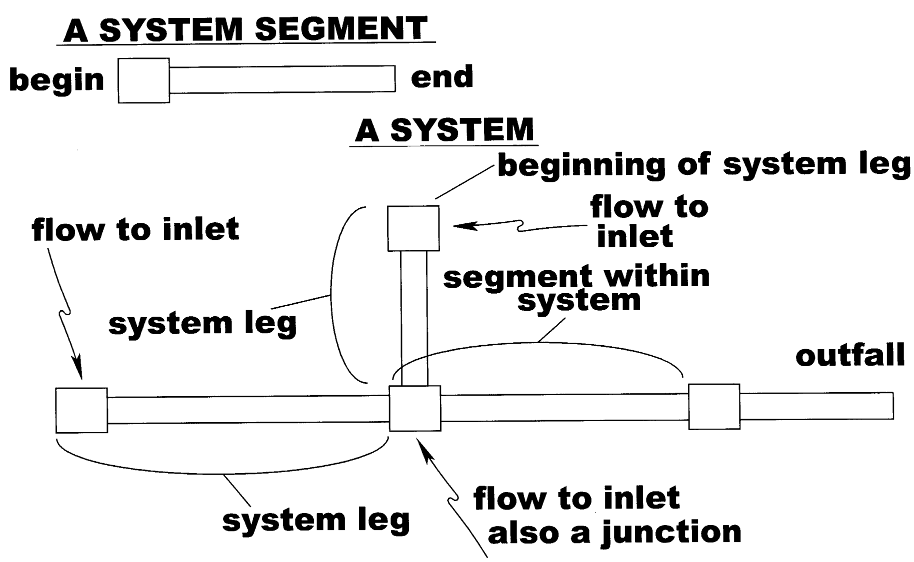
DRAINAGE COMPUTATION FORM 5-1
INLET SPACING

PROJECT NAME ______ PROJECT # ______ SHEET ____ OF ____

INLET ID.		FLOW IN THE GUTTER							GUTTER SECTION PROPERTIES						GUTTER DISCHARGE				INLET CAPACITY		REMARKS
			TIME OF CONCENTRATION																		
INLET NUMBER	STATION	DRAINAGE AREA	SHEET FLOW	SHALLOW CONCEN. FLOW	TOTAL	RAINFALL INTENSITY	RUNOFF COEFICIENT	= C I A	LONGITUDINAL GUTTER SLOPE	GUTTER CROSS SLOPE	ROADWAY CROSS SLOPE	GUTTER WIDTH	INLET TYPE	INLET DIMENSIONS	PREV IOUS INLET BYPASS FLOW	TOTAL GUTTER FLOW $Q_{10} + Q_B$	GUTTER FLOW SPREAD	GUTTER FLOW DEPTH	INTERCEPTED FLOW	INLET BY PASS FLOW	
		A	t_t	t_t	T_c	i	C	Q_{10}	S_L	S_W	S_X	W			Q_B	Q	T	d	Q_i	Q_B	
		acres	min	min	min	in/hr		cfs	ft/ft	ft/ft	ft/ft	feet		feet x feet	cfs	cfs	feet	feet	cfs	cfs	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	

A Step By Step Guide to Form 5-2 STORM SEWER, HYDRAULIC DESIGN

A chain of inlets connected by pipes will be referred to as a system. Each segment in the system consists of an inlet and the pipe leaving that inlet. Each line on Form 5-2 represents one segment. The FROM POINT in column 1 represents the invert of the inlet (beginning of segment) and the TO POINT in column 2 represents the invert at the end of the pipe leaving the inlet (end of segment). A leg in the system begins with an inlet and ends in a junction. The first segment in a leg in the system (no segment preceding it) should have a blank line above it on Form 5-2 (excluding the use of line 1). A separate form shall be used for each system. When laying out a system the design should stay as close to the surface as possible while considering the minimum cover for each pipe. If an increase in the size of the outflow pipe is not necessary, the outflow pipe should be designed at least 0.1 feet lower than the lowest inflow pipe invert elevation to ensure flow through the junction after the construction of the system. See Section 5.3.6.1 for guidance.

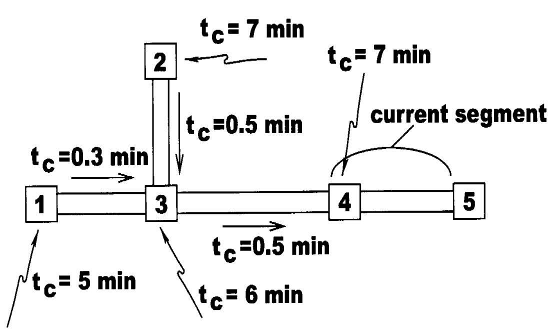


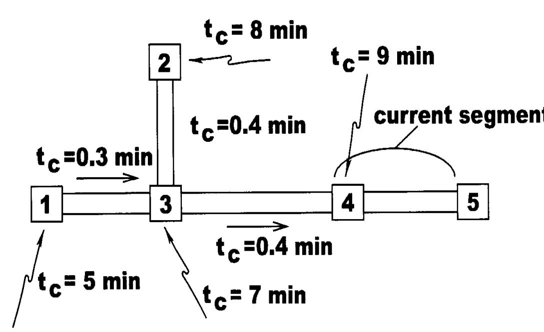
STEP 1, PIPE ID: Start at the beginning of a system leg on line 1. Fill in column 1 (FROM POINT) with the inlet label from the plan sheet. Fill in column 2 (TO POINT) with the proceeding inlet label from the plan sheet. All legs attached to the next segment shall be designed before continuing on to a segment within the system (legs 2-3 and 1-3 in the diagram on the next page).

STEP 2A, PIPE FLOW: Calculate the pipe flow at the invert of the inlet (FROM POINT) using the Rational Method. Fill in the amount of drainage area (in acres) that contributes flow to the inlet opening in column 3. Fill in the weighted runoff coefficient (C) for that drainage area in column 4. Multiply the drainage area and the weighted runoff coefficient and place this value in column 5. This represents the increment CA value at the current segment. Sum the increment CA values for the preceding segments and the current segment and place this value in column 6. This value represents the accumulated product of the drainage area and runoff coefficient for the system up to the current segment.

STEP 2B: Calculate the flow time of concentration at the beginning of the segment and fill in column 7. For a segment at the start of a leg in the system, this value is the time of concentration to the inlet opening. For a segment within the system or within a leg, this value is the larger of the longest overall time of concentration for the system from the previous segment (column 21 from the previous line) vs. the time of concentration of flow to the inlet opening. The purpose of this comparison is to obtain the longest flow time of concentration to the current segment. In the case of a storm sewer system hydraulic design, the minimum T_c value of 5 minutes shall not apply. Since the travel time is summed as a design progresses through a system, applying the minimum of 5 minutes at the beginning artificially lowers the rainfall intensity when determining the overall time of concentration at the system outlet.

The following diagrams illustrate a couple of examples:

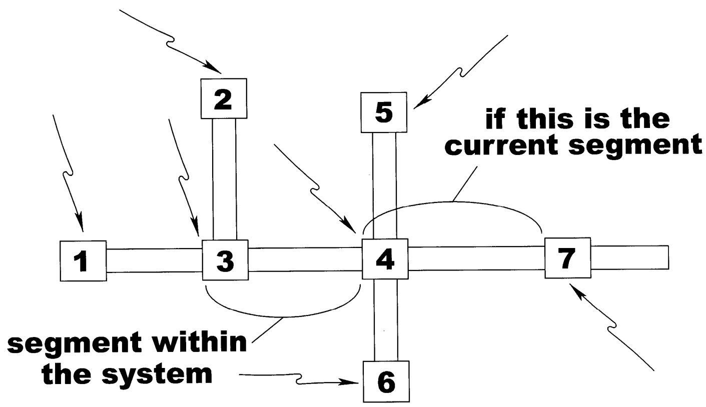




For the example on the left, the longest overall time of concentration for the system at the current segment would be 8 minutes (segment 2-3-4) and the time of concentration of flow to the inlet opening would be 7 minutes (in 4), therefore the answer is 8 minutes. For the example on the right, the longest overall time of concentration for the system at the current segment would be 8.8 minutes (segment 2-3-4) and the time of concentration of flow to the inlet opening would be 9 minutes (in 4), therefore the answer is 9 minutes.

For a situation where the current segment is preceded by a segment within the system and one or more system legs, compare the longest overall time of concentration for the system from the previous segment within the system vs. each of the legs vs. the time to the inlet opening and record the longest value.

The following diagram illustrates this comparison:



Compare the column 21 values for segment 3-4, segment 5-4, and segment 6-4 to the time of concentration into inlet 4 and record the largest value in column7 for the current segment.

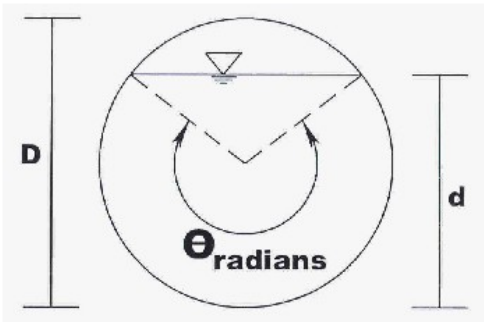
STEP 2C: Determine the rainfall intensity for a 10-year return period for the time of concentration in column 7 from Chart 4-2, 4-3, or 4-4 and place this value in column 8. Calculate the pipe flow at the invert of the inlet (FROM POINT) for a 10-year return period by multiplying the values in column 6 and column 8 and place the answer in column 9.

STEP 3, PIPE PROPERTIES: Record the pipe properties for the segment by filling in columns 10 through 15 with the appropriate data. As stated in Section 5.3.6.1 the pipe should flow at a depth no greater than 0.8 times the pipe diameter or equivalent at the design discharge, thus column 15 is provided for a check against the calculated flow depth.

STEP 4A, PIPE FLOW PROPERTIES: Calculate the pipe flow normal depth using Mannings equation and fill in column 16. If you re-arrange Mannings equation with a constant on one side, you can iteratively solve for the depth of flow if the slope, roughness, and flow are known. You can also solve for a pipe diameter if you set the flow depth at 0.8 times the diameter (0.8D) with the slope, roughness, and flow as known parameters.

$$Q = \frac{1.486}{n} A\, R^{\frac{2}{3}}\, S^{\frac{1}{2}} \qquad 0.6730 = \frac{1}{Q\,n} A\, R^{\frac{2}{3}}\, S^{\frac{1}{2}}$$

Here are some useful formulas for the iteration:



$$\theta_{Radians} = 2\,ar\cos\left(1 - 2\,\frac{d}{D}\right)$$

$$WettedPerimeter = 0.5\,D\,\theta_{radians}$$

$$Flow\ Area = \frac{D^2}{8}\left(\theta_{radians} - \sin\theta_{radians}\right)$$

STEP 4B: Calculate the pipe flow average velocity and critical depth and fill in columns 17 and 18. Average velocity is simply the flow in the pipe divided by the flow area in the pipe. Critical depth describes the flow depth that minimizes the specific energy (head) for the open channel flow within the pipe (see section 8.4.5.1). It is important to examine the open channel flow regime as it can exist in a state of subcritical or supercritical flow based on the effect of gravity. This is important because the flow state determines the effect of head loss on a storm sewer system. The effect of gravity on the state of flow is represented by a dimensionless ratio of the inertial flow forces to the gravitational forces which is known as the Froude Number.

$$\frac{V}{\sqrt{g\ d_{md}}} \qquad d_{md} = \frac{A}{T}$$

Where: V = average velocity, ft/s

g = acceleration due to gravity, ft/s^2

d_{md} = hydraulic mean depth, ft

A = open channel flow area, ft^2

T = top width of flow, ft

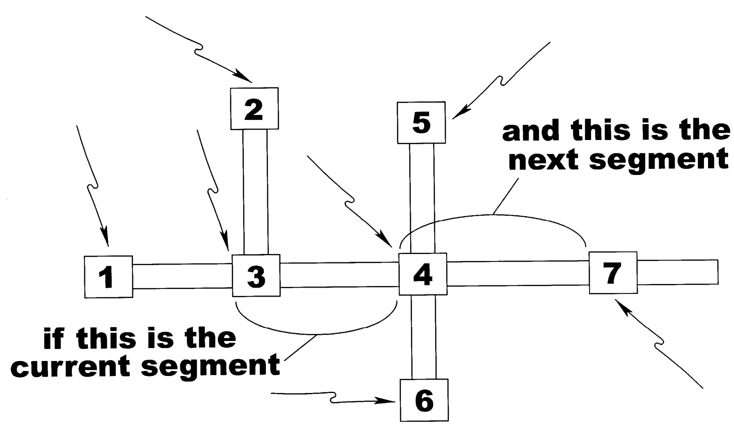
If this number is less than 1 the gravitational forces are dominant and the flow is in a subcritical state, characterized by deep slow flow. If this number is greater than 1 the inertial forces are dominant and the flow is in a supercritical state, characterized by shallow fast flow. The hydraulic mean depth that causes the Froude Number to be equal to 1 is the critical depth of flow. Major (pipe friction) and minor (access hole or inlet) head losses do not apply when the flow is in a supercritical state. In the case of the minor losses the flow must be supercritical on both the inflow and outflow sides of the inlet. Comparing the calculated pipe flow normal depth to the critical depth will reveal the state of flow. Normal depths less than the critical depth are in a supercritical state and normal depths greater than the critical depth are in a subcritical state. See section 8.4.5.1 and 5.3.6.9 for more information. Fill in column 19 with the state of flow.

STEP 5, SYSTEM FLOW TIME: Calculate the system flow time up to the end of the current segment and fill in columns 20 and 21. The pipe flow time is simply the length of the pipe (column 13) divided by the velocity of flow through the pipe (column 17). The longest overall time of concentration is the time of concentration at the beginning of the segment (column 7) plus the pipe flow time (column 20). This number will be compared to the time of concentration of flow to the inlet opening for the next segment <u>within the system</u>.

STEP 7, SELF CLEANING: Calculate the minimum velocity needed for self cleaning of the pipe and fill in columns 22 and 23. This velocity should not be less than 2 feet per second when the pipe is flowing 15% full. If this number is less than 2 feet per second, make the required changes and re-design the segment on the next line of Form DR-5A (do not skip a line).

STEP 8, CONTINUE: Continue to the next segment. If there is more than one leg attached to the next segment, then the following line(s) on the form (blank line preceding each) shall be the other legs attached to the next segment. Once all the attached legs are designed continue on to the next segment within the system. In the diagram to the right, legs 5-4 and 6-4 shall precede the next segment.

Fill in column 1 with the inlet label from the plan sheet. Fill in column 2 with the proceeding inlet label from the plan sheet for the next segment. Repeat steps 2 through 8 until the system outfall is reached.



Form 5-2

Storm Sewer, Hydraulic Design Form

DRAINAGE COMPUTATION FORM 5-2
STORM SEWER, HYDRAULIC DESIGN

PROJECT NAME____________
PROJECT NUMBER____________

DESIGNER__________ DATE______
REVIEWER__________ DATE______

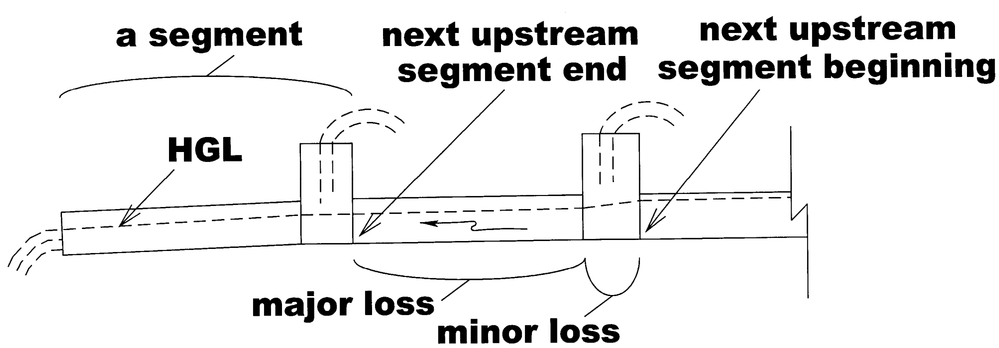
SHEET______ OF______

LINE	PIPE ID.		PIPE FLOW (AT THE FROM POINT)		CA					PIPE PROPERTIES	INVERT ELEVATIONS					PIPE FLOW PROPERTIES				SYSTEM FLOW TIME		SELF CLEANING	
	FROM POINT	TO POINT	FLOW TO INLET DRAINAGE AREA	RUNOFF COEFFICIENT (WEIGHTED?)	INCREMENT	ACCUMULATED	TIME OF CONCENTRATION	RAINFALL INTENSITY	RUNOFF = C I A	LONGITUDINAL PIPE SLOPE	UPPER END FROM POINT	LOWER END TO POINT	PIPE LENGTH	PIPE DIAMETER	80% OF PIPE DIAMETER	PIPE FLOW DEPTH	PIPE FLOW VELOCITY	PIPE FLOW CRITICAL DEPTH	SUBCRITICAL OR SUPERCRITICAL FLOW	PIPE FLOW TIME	LONGEST OVERALL TIME OF CONCENTRATION	15% OF PIPE DIAMETER	PIPE FLOW VELOCITY
			A	C	CA	CA	t_C	i_{10}	Q_{10}	S				D	0.8D	d	V	d_{CR}	SUB SUP		t_C	0.15D	$V_{15\%}$
			acres		acres	acres	min	in / hr	cfs	ft / ft	feet	feet	feet	feet	feet	feet	ft / s	feet		min	min	feet	ft / s
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1																							
2																							
3																							
4																							
5																							
6																							
7																							
8																							
9																							
10																							
11																							
12																							
13																							
14																							
15																							

A Step By Step Guide to Form 5-3 and Form 5-4

STORM SEWER, HYDRAULIC GRADE LINE

A chain of inlets connected by pipes will be referred to as a system. Each segment in the system consists of an inlet and the pipe leaving that inlet and each line on Form 5-3 represents one segment. A location where more than one segment connects will be referred to as a junction. For this process, a leg in the system begins at a junction and ends with an inlet (backwards from hydraulic design process). The diagram on the next page illustrates the terminology used in this process. Each segment at a leg in the system (one with no segment preceding it) should have a blank line below it on Form 5-3. A separate form should be used for each system.



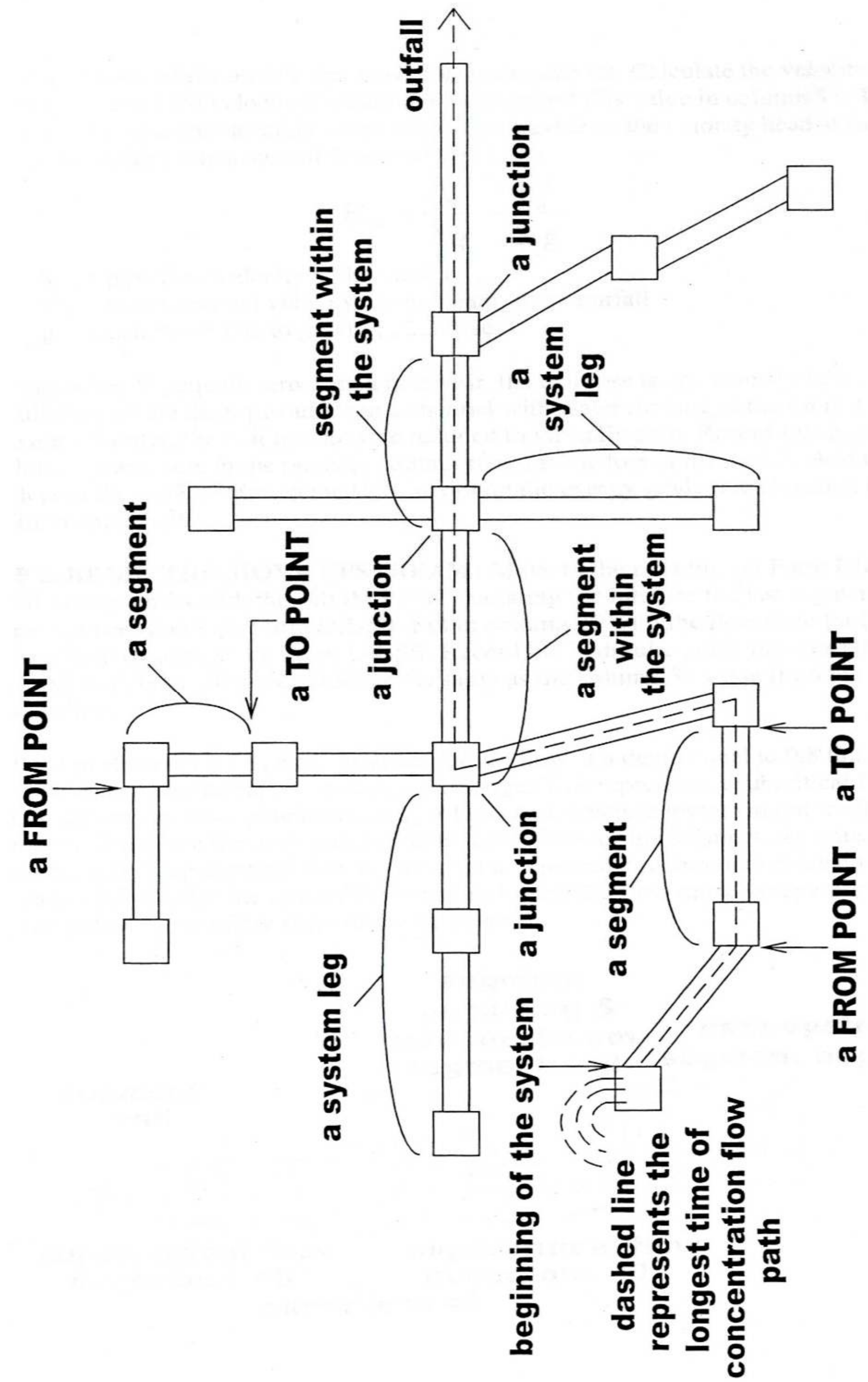
Hydraulic grade line computations begin at the outfall of the system and are calculated upstream taking each junction into consideration.

Energy losses in a system are classified as major (flow friction loss) and minor (access hole or inlet loss). Each segment is referred to by its respective inlet and the total loss is calculated up to the inflow side of the inlet.

STEP 1, SYSTEM OUTFALL: Start at the system outfall and fill in column 24 with a label notating the end of the pipe. You want to start with the same label for the TO POINT from the last segment of the system from Form 5-2. This process will work the connected segments backwards to a system leg or the beginning of the system through a junction. Fill in column 32 with the velocity of the flow at the outfall. This is the flow velocity at the end of the last segment of the system in column 17 on Form 5-2. A storm sewer is assumed to be under outlet control when determining the hydraulic grade line (HGL) for the system. The starting HGL elevation for the outfall is placed in column 37 and it is determined in the same manner as the tailwater elevation for a culvert design in an outlet control situation (see Section 8.4.10).

Figure 5-10

A System Illustration



In the case of the outfall pipe outlet being submerged, the tailwater or HGL is the observed or known water surface elevation at the outfall. In the case of the outfall pipe flowing partly full or unsubmerged, the tailwater or HGL is determined by comparing the observed or normal depth in the outlet channel and the average of the critical depth and the pipe diameter. The larger of these two values is the tailwater or HGL at the outfall. Record this value in column 37.

Record the crown of the outfall pipe elevation in column 38. Calculate the velocity head for the outlet using the velocity in column 32 and record this value in column 33. The exit loss from a storm drain outfall is a function of the change in the velocity head at the outlet. For a sudden expansion of flow the exit loss is:

$$H_o = \frac{V_O^2}{2g} - \frac{V_d^2}{2g}$$

V_o = pipe flow velocity at the outfall

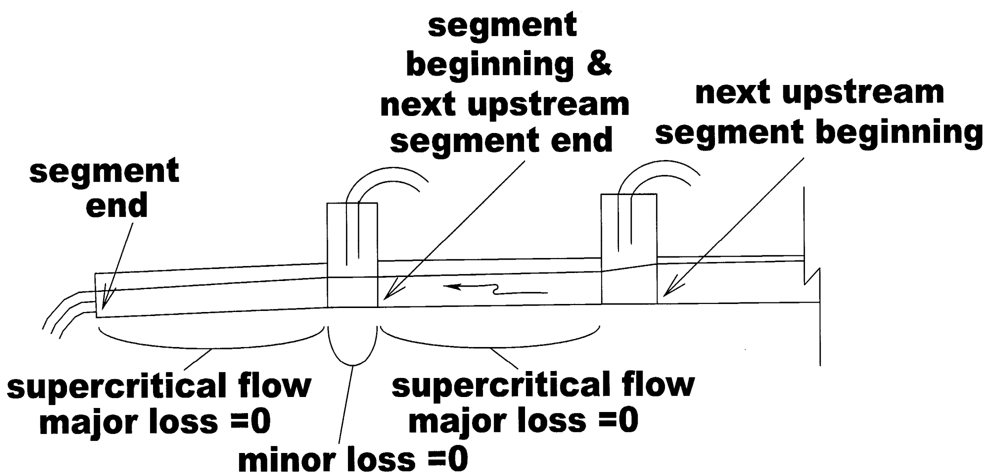
V_d = outlet channel velocity downstream of the outfall

g = acceleration due to gravity, 32.2 ft/s^2

Note that when V_d equals zero as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with water moving in the same direction as the outfall water, the exit loss may be reduced to virtually zero. Record this head loss in column 36 and note in the remarks column that it is the loss at the outfall. Add this head loss at the outfall value to the HGL to obtain the energy grade line elevation (EGL) and fill in column 39.

STEP 2, BEGIN THE MOVE UPSTREAM: Move to the next line on Form 5-3 and fill in column 24 with the TO POINT notating the same outfall end of the last segment in the system from column 2 on Form 5-2. Fill in column 25 with the flow state for the segment from column 19 on Form 5-2. Record the hydraulic grade line elevation for the outfall in column 26 which shall be the same as the column 37 value from the previous line.

As stated in Section 5.3.6.1, a storm sewer should flow at a depth equal to 0.8 times the pipe diameter or equivalent at the design discharge. This represents a subcritical flow state. In subcritical flow, pipe losses (major loss) and structure losses (minor loss) are summed to determine the upstream hydraulic grade line for the segment. As stated in Section 5.3.6.9 if supercritical flow is present, the pipe and structure losses are not carried upstream. However for the case of the structure losses, the flow must be supercritical on both the inflow and outflow sides of the structure.



Therefore, if supercritical flow exists within the current segment pipe, the major loss is zero and column 31 shall be zero. If supercritical flow exists within the preceding segment pipe, the major loss for the preceding segment is zero and the minor loss for the structure in-between them shall also be zero.

STEP 3, MAJOR LOSSES: Record the diameter, flow, and length for the segment in columns 27, 28, and 29 from the information on Form 5-2. If column 25 denotes SUB calculate the friction loss H_f and fill in column 31. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope S_f will match the pipe slope for partly full flow. In this case, the pipe slope from column 10 on Form 5-2 will be the same as column 30 on Form 5-3. The major loss is calculated by this equation:

$$H_f = S_f * pipe\ \ length$$

Major losses for full pipe flow result in a different friction slope thus they are calculated by this equation:

$$H_f = S_f * length = \left(\frac{Q\,n}{0.46\,D^{2.67}} \right)^2 * length$$

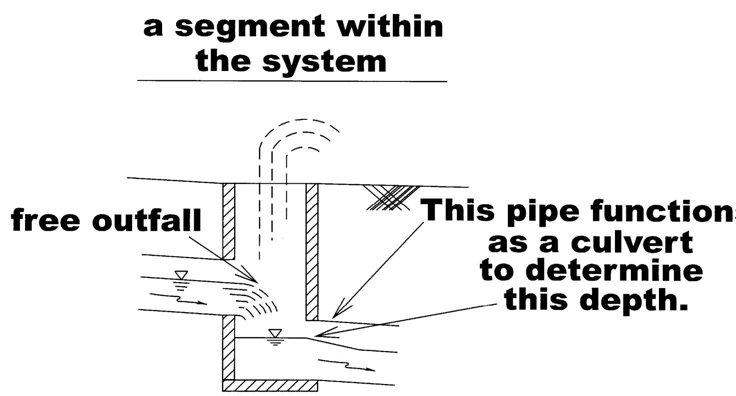
Q = pipe flow

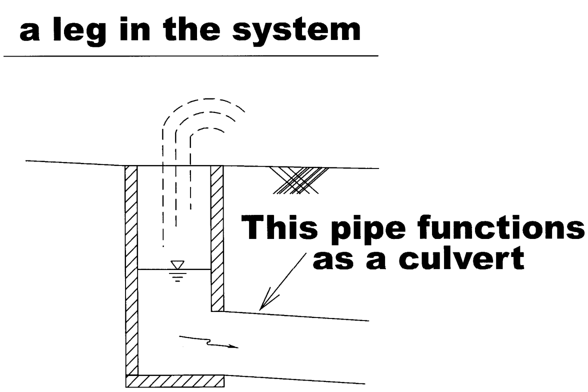
n = Manning's roughness coefficient

D = pipe diameter

If the flow state in column 25 denotes SUP (supercritical) record a zero in column 31.

STEP 4, MINOR LOSSES: The energy loss equation for flow through an access hole or inlet structure is based on laboratory research. The equation will not apply when the invert of the inflow pipe is above the water level in the structure or if there is no inflow pipe present and the inlet opening is the only source of the inflow (as occurs in a leg of the system).





In both situations the outflow pipe or current segment pipe will function as a culvert. In the case of the invert of the inflow being above the water level, the hydraulic and energy grade line calculations will start over, thus treating it as a system outfall.

For a culvert design, if the segment pipe is flowing full or partly full under outlet control, the minor loss in the structure can be calculated by setting K equal to K_e in the equation below (see Table 8-4, Section 8.4.5). If the segment pipe is flowing under inlet control, the minor loss in the structure shall be computed using the inlet control equations (see Section 8.4.5) or Charts 8-1 or 8-4 from Chapter 8. The higher water depth in the structure due to either inlet or outlet control will govern.

The energy loss that occurs as flow moves through a structure is represented as being proportional to the velocity head of the outlet pipe from the structure.

Therefore the velocity head for the segment pipe being analyzed will be multiplied by a coefficient to represent the minor loss. Record the velocity for the segment from column 17 on Form 5-2 into column 32 on Form 5-3 and calculate and record the velocity head in column 33. The equation for the minor loss through the structure is as follows:

$$H_{aho} = K\left(\frac{V_S^2}{2\,g}\right)$$

H_{aho} = minor loss for the access hole or inlet

V_S = velocity for the segment pipe

K = minor loss coefficient

g = acceleration due to gravity, 32.2 ft/s^2

Determination of the K value is based on experimental study which resulted in a six term equation. Each term represents a different scenario that the flow may experience as it moves through the structure.

$$K = K_o\ C_d\ C_D\ C_Q\ C_P\ C_B$$

K_O = initial head loss coefficient

C_d = factor for flow depth (free surface flow only)

C_D = factor for pipe diameter change (pressure flow only)

C_Q = factor for relative flow

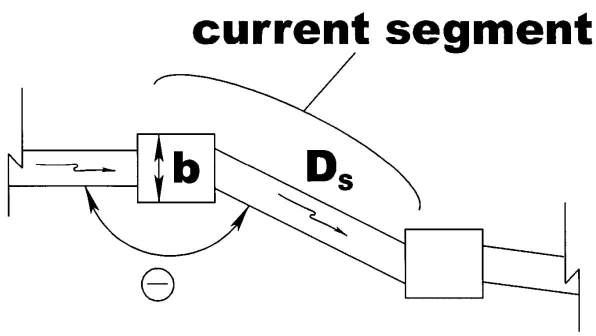
C_P = plunging flow factor

C_B = benching factor (energy gain)

The minor loss coefficient will be calculated in the following steps using Form 5-4. Fill in column 40 with the same TO POINT from column 24 on Form 5-3. **<u>Fill in column 41 with the NEXT UPSTREAM SEGMENT flow state from column 19 on Form 5-2</u>.** Remember, the flow must be supercritical on <u>BOTH</u> the inflow and outflow sides of the structure to neglect the minor losses through it. If column 25 and column 41 denote SUP then record a ZERO in column 57 and columns 34 & 35 for the current segment.

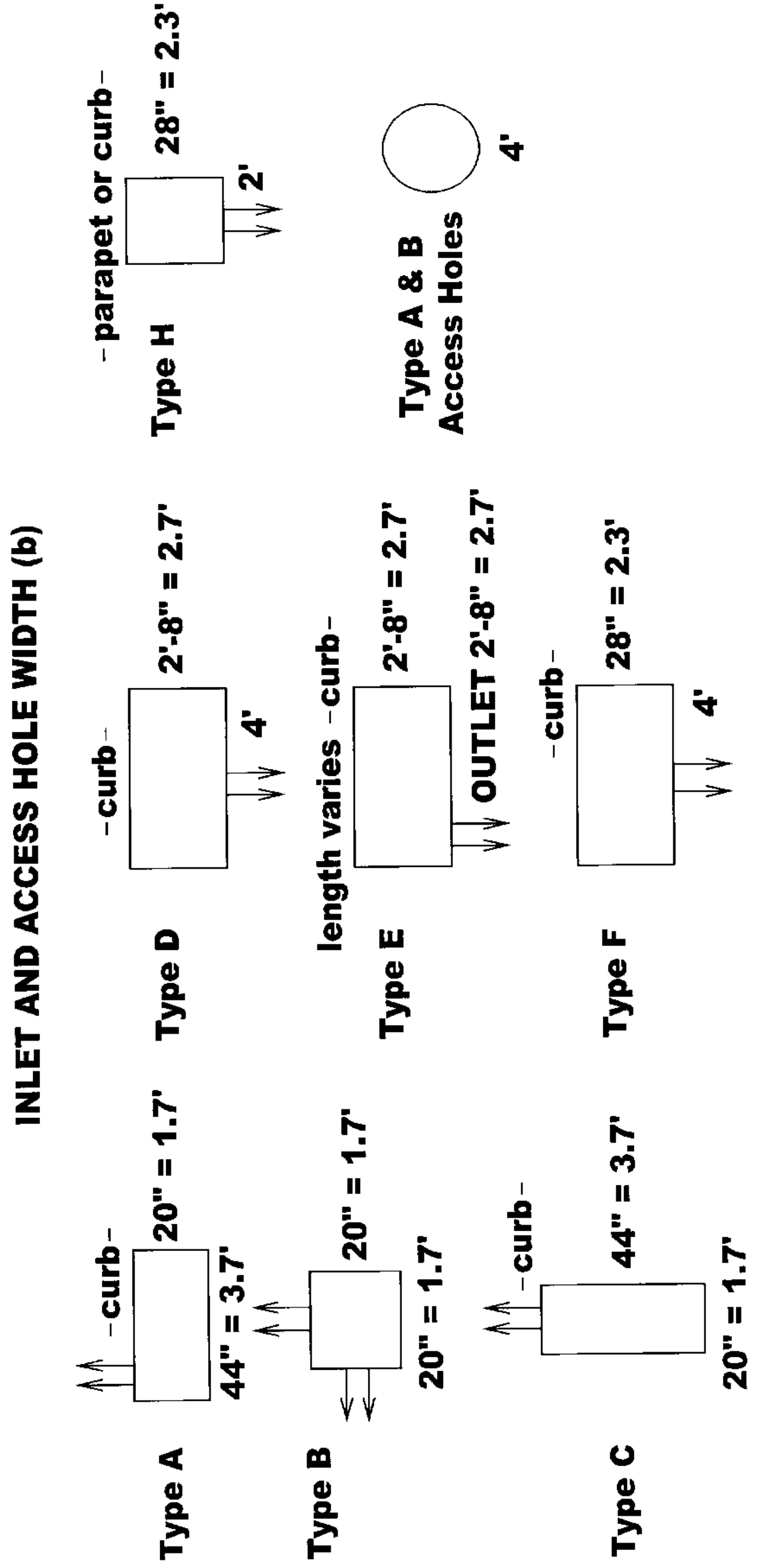
STEP 4A, INITIAL HEAD LOSS: The initial head loss coefficient is estimated as a function of the relative structure size and the angle of deflection between the next upstream segment pipe and the current segment pipe (in degrees).

$$K_o = 0.1\left(\frac{b}{D_S}\right)(1-\sin\theta) + 1.4\left(\frac{b}{D_S}\right)^{0.15}\sin\theta$$



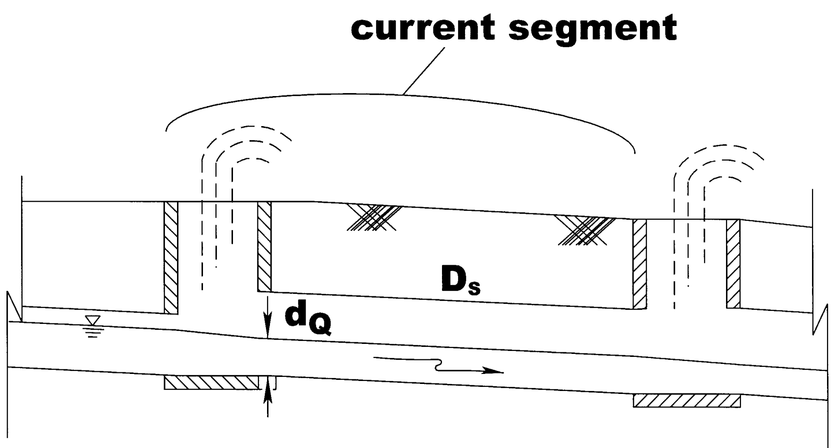
b - inlet width or access hole diameter
θ - change in flow angle
D_s- segment pipe diameter

Record the inlet width or access hole diameter in column 42 (see next page), the change in flow angle between the current segment and the next upstream segment in column 43, and the current segment pipe diameter in column 44. Calculate and record the initial head loss coefficient in column 45.


INLET AND ACCESS HOLE WIDTH (b)
Type A
-curb-
20" = 1.7'
44" = 3.7'
Type B
20" = 1.7'
20" = 1.7'
Type C
-curb-
44" = 3.7'
20" = 1.7'
Type D
-curb-
2'-8" = 2.7'
4'
Type E
length varies -curb-
2'-8" = 2.7'
OUTLET 2'-8" = 2.7'
Type F
-curb-
28" = 2.3'
4'
Type H
-parapet or curb-
28" = 2.3'
2'
Type A & B
Access Holes
4'

STEP 4B, FACTOR FOR FLOW DEPTH: This factor is significant <u>only in cases of free surface flow</u> or low pressures when the water depth in the structure is less than or equal to 3.2 times the segment pipe diameter. In cases where the water depth is greater, this factor is set equal to 1. To determine the applicability of this factor, the water depth is approximated as the level of the hydraulic grade line at the upstream end of the segment pipe.

$$C_d = 0.5\left(\frac{d_Q}{D_S}\right)^{0.6}$$



d_Q- water depth in inlet or access hole

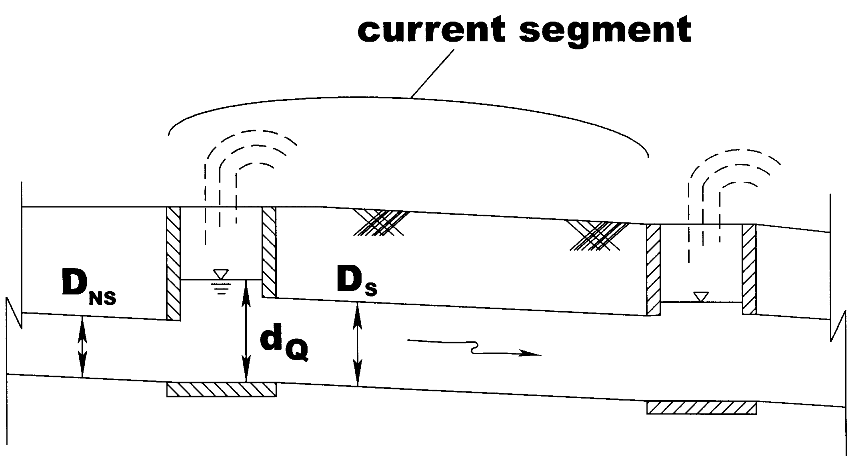
D_s- segment pipe diameter

$d_Q \leq 3.2\ D_s$

Record water depth or flow depth in the inlet or access hole in column 46. Calculate and record the free surface factor for flow depth in column 47.

STEP 4C, FACTOR FOR A PIPE DIAMETER CHANGE: A change in head loss due to differences in segment pipe diameters is <u>only significant in pressure flow situations</u> when the water depth in the structure is greater than or equal to 3.2 times the segment pipe diameter. In cases where the water depth is less, this factor is set equal to 1.

$$C_D = \left(\frac{D_S}{D_{NS}} \right)^3$$



d_Q- water depth in inlet or access hole

D_s- segment pipe diameter

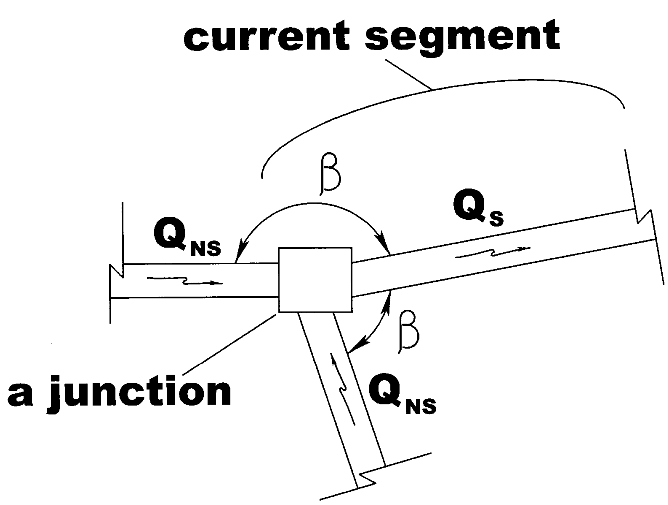
D_{NS}- next upstream segment pipe diameter

$d_Q \geq 3.2\ D_s$

Record the next upstream segment pipe diameter in column 48. Calculate and record the pressure flow factor for a pipe diameter change in column 49. Assume that the d_Q requirement is met and determine the minor loss for column 49. Go back and check this d_Q requirement once the water surface elevation (HGL) has been determined for column 37. If this d_Q requirement is not met then change this factor to 1 prompting a new water surface elevation (HGL) calculation for column 37.

STEP 4D, FACTOR FOR RELATIVE FLOW: This factor is a function of the angle of the incoming flow as well as the percentage of flow entering the structure from the next upstream segment in the analysis path. This factor is only applied to a junction where there are 3 or more pipes connected to the structure at approximately the same elevation. Engineering judgment will have to be used to determine the meaning of the word approximately. This judgment should be based on the vertical geometry of the entire proposed system. For example, If only one junction in the system exhibits a noticeable difference between inflowing pipes then that junction should not have a relative flow loss factor.

$$C_Q = (1 - 2\sin\beta)\left(1 - \frac{Q_{NS}}{Q_S}\right)^{0.75} + 1$$



Q_S - segment pipe flow

Q_{NS} - next upstream segment pipe flow

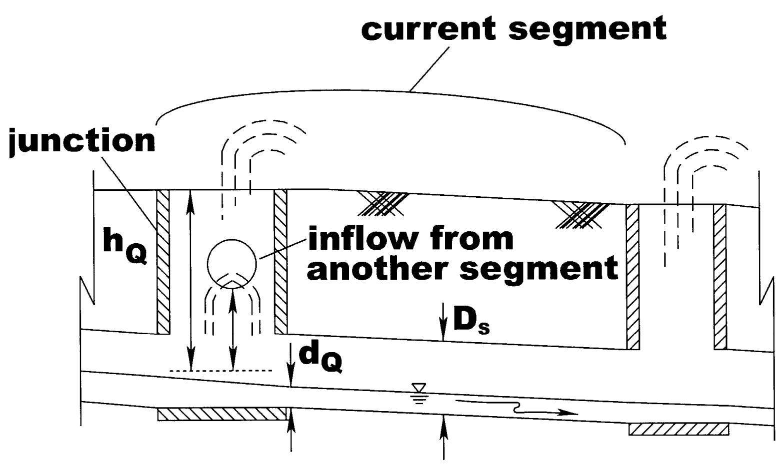
β - angle between analyzed segments

Record the current segment pipe flow in column 50, the next upstream segment pipe flow in column 51 (also from column 9 on Form 5-2), and the angle between the analyzed segments in column 52 (in degrees). Calculate and record the factor for relative flow in column 53.

STEP 4E, PLUNGING FLOW FACTOR: This factor is applied only when a higher flow plunges into a structure that has an inflow and outflow in the bottom of the structure. The higher flow must fall a distance that is greater than the water depth in the inlet or access hole. The source of the higher flow could be from the next upstream segment along another analysis path at a junction, or from a curb and/or grate opening from above. The distance of fall is the vertical length from:

- The invert of the next upstream segment along another analysis path at a junction to the center of the current segment pipe.
- The structures' curb and/or grate opening to the center of the current segment pipe.
- Two important facts to note in determining this loss are:
- The water depth in the inlet or access hole is approximated as the level of the hydraulic grade line at the upstream end of the segment pipe.
- If both sources of plunging flow are present in the same structure they will both add to the turbulence and head losses. In this case the average plunging height between the two shall be used to determine the factor.

$$C_P = 1 + 0.2\left(\frac{h_Q}{D_S}\right)\left(\frac{h_Q - d_Q}{D_S}\right)$$



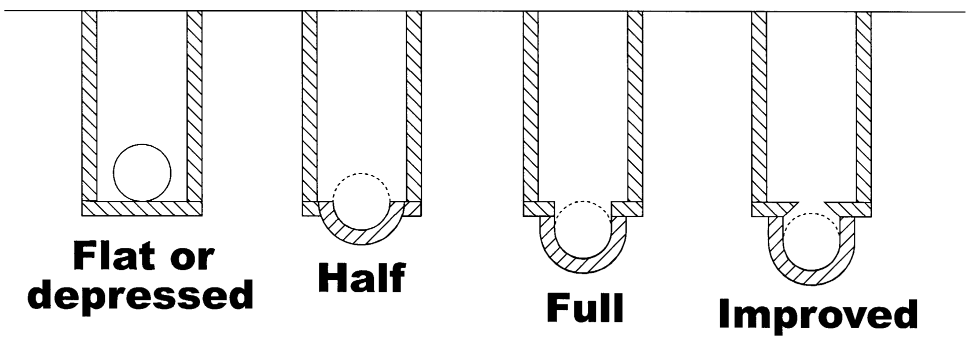
d_Q- water depth in inlet or access hole
D_s- segment pipe diameter
h_Q- vertical distance of plunging flow

Record the vertical distance of plunging flow in column 54. Calculate and record the plunging flow factor in column 55.

STEP 4F, STRUCTURE BENCHING FACTOR: Benching provides an improvement for the transmission of flow through a structure. It tends to direct flow through the inlet or access hole, resulting in a <u>reduction of head loss.</u> The factor is obtained from the following table for flow depths within the structure respective to the segment pipe diameter. Refer to the diagram for the plunging flow factor for an illustration of d_Q and D_S. If the d_Q requirement is not met then this factor is equal to one.

Bench Type	Submerged (pressure flow)	Unsubmerged (free surface flow)
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved Bench	0.40	0.02

Types of structure benching



for pressure flow $d_Q \geq 3.2\ D_s$

for free surface flow $d_Q \leq D_s$

Record the benching factor in column 56 from the table above.

STEP 4G, MINOR LOSS COEFFICIENT: Multiply the values in columns 45, 47 (if applicable), 49 (if applicable), 53, 55, & 56 to obtain the minor loss coefficient K and record it in columns 57 on Form 5-4 and 34 on Form 5-3. Determine the access hole or inlet loss for the current segment by multiplying columns 34 and 33 and record this value in column 35.

STEP 5, TOTAL HEAD LOSS: The total loss is simply the addition of the major and minor losses for the current segment. Add the values in columns 31 and 35 and record this value in column 36. This energy loss will determine the hydraulic grade line elevation from the outlet end of the pipe (TO POINT) to where flow enters the structure (FROM POINT) for the current segment.

STEP 6, WATER SURFACE ELEVATION (HGL): Add the head or energy loss from column 36 to the pipe outlet water surface elevation from column 26 and record this value in column 37. This represents the hydraulic grade line elevation for the beginning of the current segment.

STEP 7, STRUCTURE TOP ELEVATION: Record the top cover elevation for the access hole or the top of grate elevation for the inlet in column 38. This step provides the check for a flow surcharge from the system. If column 37 is higher in elevation than column 38, water is purging from the system for the design storm.

STEP 8, ENERGY GRADE LINE: The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit that carries water. Total energy includes elevation head, velocity head and pressure head. The energy equation states that the energy head at any cross section within the system must equal that in any other downstream section plus the intervening losses. The EGL for the beginning of the current segment is the HGL plus the velocity head at the end of the next upstream segment. Therefore once column 33 is determined for the next line on Form 5-3 (or the next upstream segment) you can return to column 39 and add that value to column 37 and record the EGL.

STEP 9, REPEAT AND CONTINUE TO MOVE UPSTREAM: Move to the next line on Form 5-3 and fill in column 24 with the TO POINT for the next upstream segment. Fill in column 25 with the flow state for the segment from column 19 on Form 5-2. Record the hydraulic grade line elevation for the outfall in column 26 which shall be the same as the column 37 value from the previous line.

Return to STEP 3 and STEP 4 and determine the losses in the segment.

Continue through STEP 5 to STEP 8 and repeat STEP 9 until you reach the end of a leg in the system or the beginning of the system. It is important to note that the segment at the start of a leg is called the beginning of the system when the time of concentration from this leg to the outfall is the longest for any path in the system.

Form 5-3

Storm Sewer, Hydraulic Grade Line Form

DRAINAGE COMPUTATION FORM 5-3
STORM SEWER, HYDRAULIC GRADE LINE

PROJECT NAME ____________________
PROJECT NUMBER ____________________

DESIGNER_______ DATE_______
REVIEWER_______ DATE_______

SHEET_____ OF _____

LINE	INLET LABEL	SUBCRITICAL OR SUPERCRITICAL CURRENT SEGMENT	PIPE OUTLET WATER SURFACE ELVEV.	MAJOR LOSS: SEGMENT PIPE DIAMETER	MAJOR LOSS: SEGMENT PIPE FLOW	MAJOR LOSS: SEGMENT PIPE LENGTH	MAJOR LOSS: FRICTION SLOPE	MAJOR LOSS: FRICTION LOSS	MINOR LOSS: FLOW VELOCITY FOR SEMENT PIPE	MINOR LOSS: SEGMENT PIPE VELOCITY HEAD	MINOR LOSS: MINOR LOSS COEFFICIENT	MINOR LOSS: ACCESS HOLE OR INLET LOSS	TOTAL HEAD LOSS	WATER SURFACE ELEV. (HGL)	STRUCTURE TOP ELEVATION (SURCHARGE CHECK)	ENERGY GRADE LINE ELEV. (EGL)	REMARKS
		SUB SUP	feet	D_S feet	Q_S cfs	L_S feet	S_f ft / ft	H_f feet	V_S ft / s	$V_S^2 / 2g$ ft / s	K	H_{aho} feet	H feet	feet	feet	feet	
	24	25	26	27	28	29	30	31	32	33	34 & 57	35	36	37	38	39	
1																	
2																	
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	
11																	
12																	
13																	
14																	
15																	

Form 5-4

Storm Sewer, Minor Losses Form

DRAINAGE COMPUTATION FORM 5-4
STORM SEWER, MINOR LOSSES

PROJECT NAME ____________________
PROJECT NUMBER ____________________

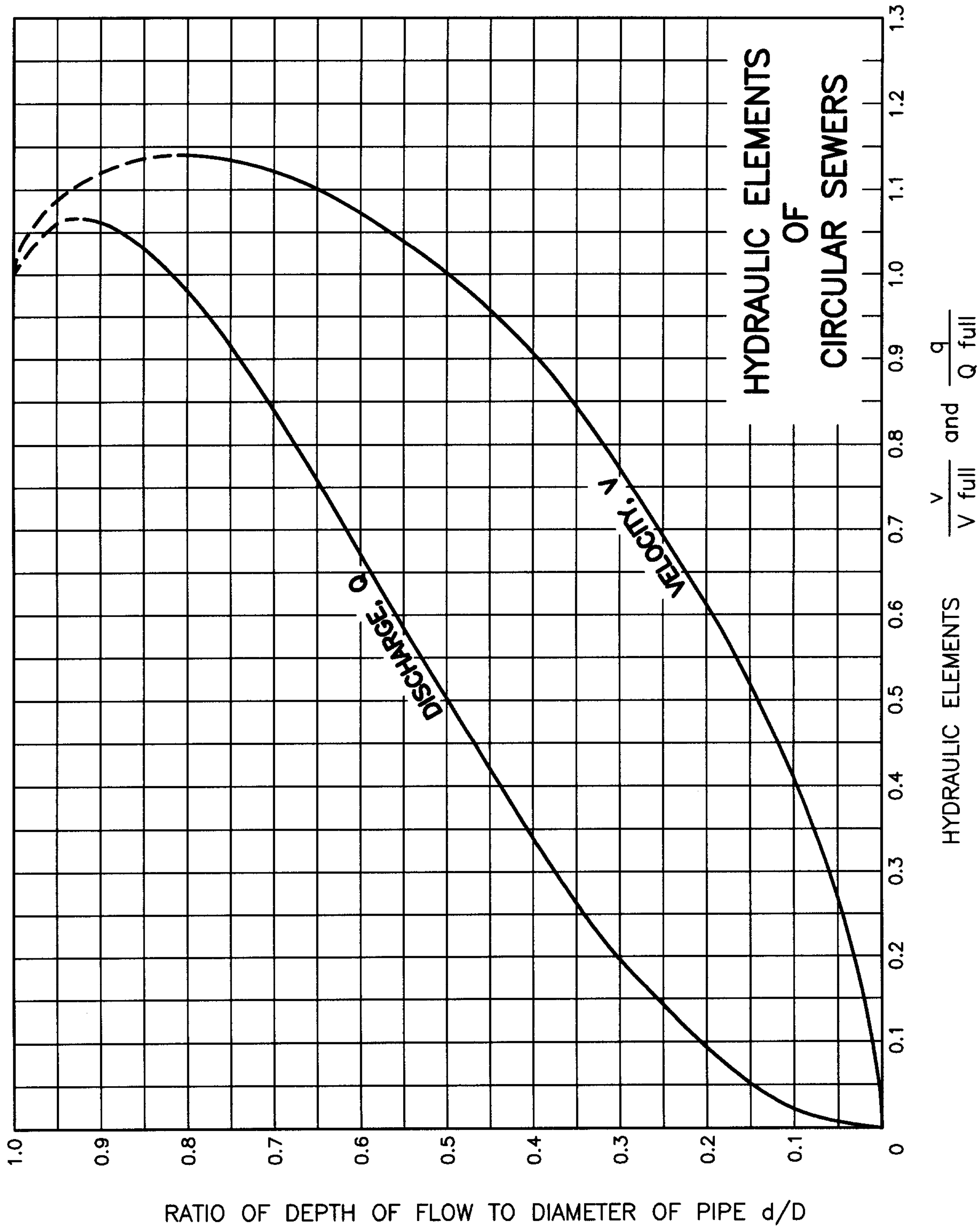
DESIGNER_______ DATE______
REVIEWER_______ DATE______

SHEET______ OF ______

			INITIAL HEAD LOSS				FLOW DEPTH LOSS		PIPE DIAMETER CHANGE LOSS		RELATIVE FLOW LOSS				PLUNGING FLOW LOSS				
LINE	INLET LABEL	SUBCRITICAL OR SUPERCRITICAL NEXT UPSTREAM SEGMENT	INLET WIDTH OR ACCESS HOLE DIAMETER	CHANGE IN FLOW ANGLE	SEGMENT PIPE DIAMETER	INITIAL HEAD LOSS COEFF.	FLOW DEPTH IN ACC. HOLE OR INLET	FREE SURFACE FACTOR FOR FLOW DEPTH	NEXT UPSTREAM SEGMENT PIPE DIAMETER	PRESSURE FLOW FACTOR FOR PIPE DIAMETER	SEGMENT PIPE FLOW	NEXT UPSTREAM SEGMENT PIPE FLOW	ANGLE BETWEEN ANALYZED SEGMENTS	FACTOR FOR RELATIVE FLOW	DISTANCE OF PLUNGING FLOW	PLUNGING FLOW FACTOR	BENCHING FACTOR (ENERGY GAIN)	MINOR LOSS COEFFICIENT	REMARKS
		SUB SUP	b feet	Θ deg	D_S feet	K_O	d_Q feet	C_d	D_{NS} feet	C_D	Q_S cfs	Q_{NS} cfs	β deg	C_Q	h_Q feet	C_p	C_B	K	
	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	
1																			
2																			
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			
11																			
12																			
13																			
14																			
15																			

Chart 5-30

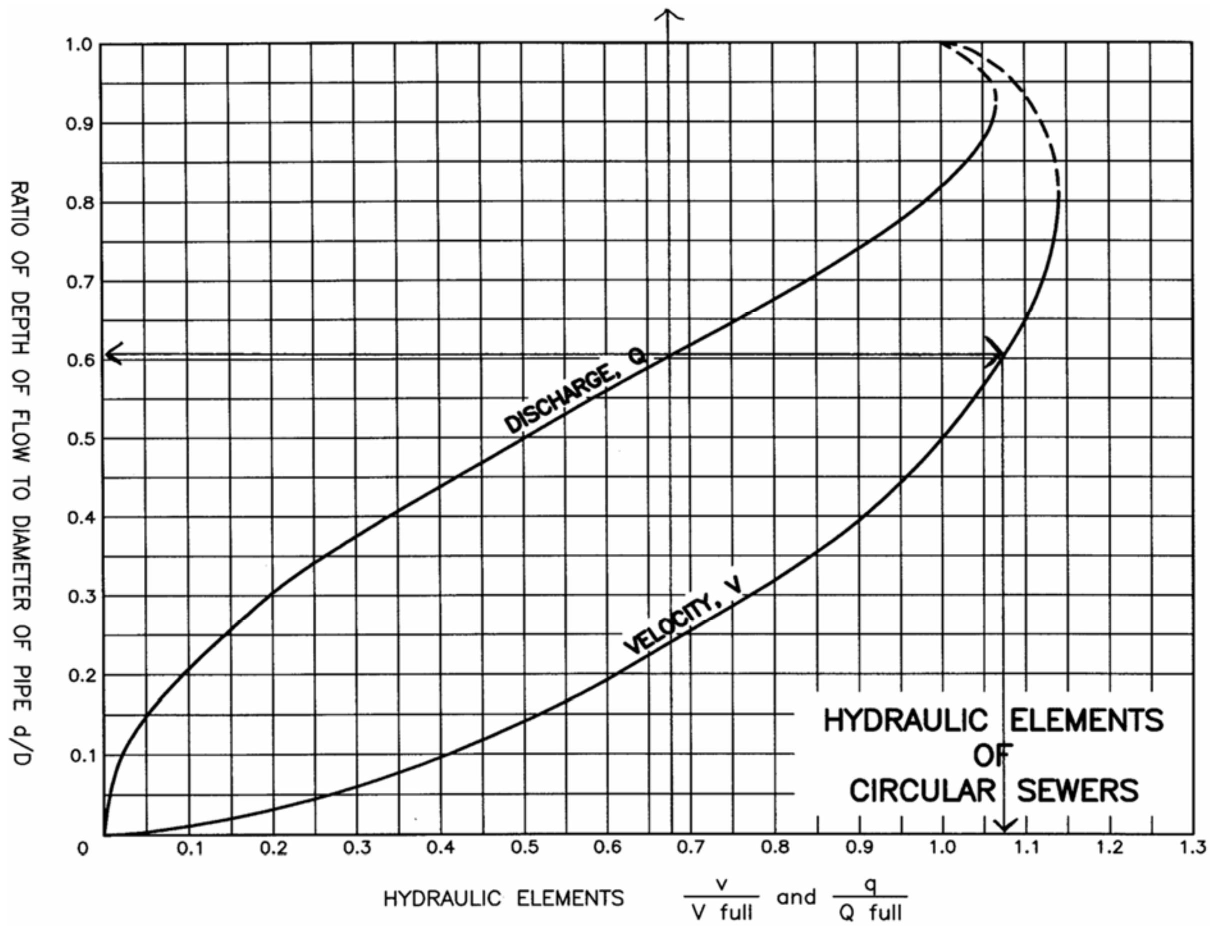
Hydraulic Elements for Partially Full Circular Storm Drains



Source: Louisville & Jefferson County M.S.D. Design Manual, January 2001

Chart 5-31

Hydraulic Elements: Example



Given: D_{full} = 18 in, Q = 7 cfs (the design flow - i.e., what the actual flow will be under design conditions), V_{full} = 5.96 fps (what the velocity would be at full flow), Q_{full} = 10.5 cfs (what Q will be when the pipe is flowing full)

Required: (a) depth, d, when Q = 7 cfs (i.e., when the sewer is partially full) and (b) velocity, v, when Q = 7 cfs (i.e., when the sewer is partially full)

Solution: $Q / Q_{full} = 7 / 10.5 = 0.67$

From figure related to hydraulic elements, $d / d_{full} = 0.61$

$d = (d_{full})\ (0.61) = 11$ in

From figure related to hydraulic elements, $V / V_{full} = 1.08$

$V = (V_{full})\ (1.08) = 6.4$ ft / sec

Source: Louisville & Jefferson County M.S.D. Design Manual, January 2001

5.5 REFERENCES

(1) Design of Bridge Deck Drainage, Hydraulic Engineering Circular-21 (HEC-21), Federal Highway Administration, May 1993

(2) Highway Drainage Guidelines, American Association of State Highway and Transportation Officials, 1999

(3) Model Drainage Manual, American Association of State Highway Transportation Officials (AASHTO), 2005

(4) Urban Drainage Design Manual, Hydraulic Engineering Circular-22 (HEC-22), Second Edition, Federal Highway Administration, August 2001

(5) Neenah Inlet Grate Capacities, Neenah Foundry company, 1987

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 6:
DITCHES

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF FORMS

LIST OF CHARTS

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CHAPTER 6: DITCHES

6.1 INTRODUCTION

Ditches are V-shaped or trapezoidal shaped channels lined with grass, concrete, rock or manufactured protective linings. They are generally designed to convey the design discharge and to resist erosion of the in-situ soil from a lower discharge. Higher design discharges may be necessary when the ditch intercepts offsite drainage. Ditches may also drain subsurface water from the base of the roadway that is conveyed through pipe underdrains.

Ditch design is accomplished by the selection of a cross-section geometry, horizontal alignment, grade, and protective linings.

6.2 DESIGN POLICY

Roadway ditches shall be designed to collect storm water runoff from the roadway right of way (including the pavement, median, cut slopes, and embankment fill slopes) and convey it in a manner that minimizes the potential for adverse effects to the roadway and adjacent properties.

The designer shall use the following general policies as a guide to plan, select and design ditches placed along roadways:

- Ditches shall be hydraulically designed.
- Ditches shall be located by: adhering to the required cross-section geometry of the WVDOH design directives, taking advantage of local terrain, and considering clear zone requirements for locations away from the roadside.
- Ditch locations should consider possible maintenance access.
- Ditch linings shall be designed to be structurally stable.
- Permanent erosion control matting shall not be used in a ditch where the frequent removal of sediment is anticipated.
- Ditches and ditch outlets shall be designed to consider construction and maintenance costs, risk of lining failure, risk of property damage, traffic safety and environmental considerations.

6.3 DESIGN CRITERIA

Ditch design criteria are controlled by geometric and safety standards applicable to the roadway design project. The following criteria shall apply to the hydraulic design of ditches:

6.3.1 FREQUENCY

Median, roadside, and secondary ditches shall be designed to carry onsite runoff from the roadway right of way for a 10-year rainfall return period or recurrence interval.

6.3.2 SIZE AND SHAPE

Roadside and median ditches shall be designed to be triangular or trapezoidal in shape with side-slopes specified by the typical roadway section (refer to Design Directive 601). Consideration shall be given to achieving the project clear zone when sizing a ditch. The bottom width of trapezoidal ditches shall vary depending on the capacity required.

The design flow capacity of trapezoidal ditches located at the toe of cut slopes shall be calculated assuming one-half of the constructed bottom width. This means that the required bottom width shall be doubled to account for filling due to erosion of the cut slope. For example: a roadside ditch design requires a two foot flat bottom to carry the 10 year discharge from the roadway and cut slope, this ditch shall be constructed with a four foot flat bottom to meet this criteria.

If significant flow from offsite is carried in a roadside ditch, then it shall be sized to carry the design discharge for the offsite drainage area. This design discharge may be defined by a nearby culvert design that will convey the offsite flow.

6.3.3 FLOW DEPTH

The maximum flow depth in a median ditch shall be 0.6 feet or 7.2 inches.

The maximum flow depth in a roadside ditch shall be 1 foot and shall not exceed a height of 1.5 feet or 18 inches below the edge of the shoulder. Roadside ditches along low volume roads may be exempted from this standard to enable a more cost effective design. The flow depth shall not be higher than the low elevation (bottom) of the roadway subgrade in all cases. If this cannot be achieved then use an underdrain (see Section 5.3.2.11) in conjunction with a ditch.

6.3.4 LONGITUDINAL SLOPE

The longitudinal slope of a median or roadside ditch shall not be less than 0.5 percent. Flatter slopes may be permitted with the use of paved gutters. In general, the longitudinal slope of the ditch shall be sufficient to satisfy the minimum velocity criteria.

6.3.5 MINIMUM FLOW VELOCITY

The flow velocity in a median or roadside ditch shall not be less than 0.5 foot per second when ditch is flowing at one-third of the design flow depth.

6.3.6 DITCH INLETS AND MAXIMUM SPACING

Inlets with the WVDOH standard Type G grates ranging in size from 32" X 38" to 60" X 66" are primarily used in median and roadside ditches. The Type 1 Grate has 1" bars, 2" on center. The type 2 grate has 1" bars, 4" on center.

The maximum pipe length between inlets shall not be greater than 400 feet to allow for maintenance access. Upon WVDOH approval, this distance may be extended up to 100 feet to enable practical drainage layouts. See design criteria for access structures in Chapter 5, Section 5.2.7.

6.3.7 SECONDARY DITCHES

Secondary ditches are to be considered at the top or toe of cut slopes and at the bottom of embankments in order to carry offsite drainage separate from the median or roadside ditch system. The need for secondary ditches shall be determined by factors such as:

- Ground cover
- Ground slope
- Type of soil
- Depth of cut
- Contributing drainage area
- Concentration of runoff
- Conveyance of runoff to a temporary sediment basin
- Maintenance of existing drainage patterns
- Need for protection of the median or roadside ditch system.

Pipe flumes and cascades may also be considered as options to convey offsite drainage from steep slopes. See the pipe culvert design criteria (Section 8.3) in Chapter 8.

6.3.8 ROUGHNESS COEFFICIENT SELECTION

The Manning's roughness coefficient selected for ditch protection should be based on the condition that represents the minimum resistance to flow. This design value shall be selected with consideration of a variation with flow depth and whether vegetative linings will be maintained (mowed or unmowed). Values for ditches with vegetative linings that cannot be mowed due to their location (such as ditches located at the bottom of steep embankments) shall be selected based on the condition that provides the maximum resistance to flow. This yields a higher flow depth and a higher shear stress.

Manning's roughness coefficient for vegetative linings shall be selected from the ranges shown in Table 6-1. Vegetative ditch linings should consist of seed mixture Type B with the use of Type C-1 in areas viewable and traversable by traffic. More information on the standard seed mixtures can be obtained from Section 652 of the WVDOH standard specifications. For flat longitudinal ditch slopes a higher value within the range should be selected.

Table 6-1

Manning's Roughness Coefficient for Vegetative Ditch Linings

WVDOH Grass Seed Mixture	**Minimum roughness coefficient. N**	**Standard roughness coefficient N**	**Maximum roughness coefficient n**
Type B (mowed)	0.036	0.042	0.050
Type C-1 (mowed)	0.030	0.036	0.040
Type C-2 (mowed)	0.022	0.027	0.033
Type B (unmowed)	0.050	0.090	0.140
Type C-1 (unmowed)	0.050	0.080	0.120
Type C-2 (unmowed)	0.025	0.030	0.040

Manning's roughness coefficient for non-vegetative linings shall be based on the material type and flow depth as shown in Table 6-2.

Table 6-2

Manning's Roughness Coefficient for Non-Vegetative Ditch Linings

Material Type	Manning's Roughness Coefficient, n		
	Depth Range (ft) 0 – 0.5	Depth Range (ft) 0.5 – 2.0	Depth Range (ft) > 2.0
Rigid			
Concrete	0.015	0.013	0.013
Grouted Rock	0.040	0.030	0.028
Unlined			
Bare Soil	0.023	0.020	0.020
Rock Cut	0.045	0.035	0.025
Rock (D_{50})			
4-inch	0.090	0.058	0.035
6-inch	0.104	0.069	0.035
12-inch	-	0.078	0.040

Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

6.3.9 DITCH PROTECTION

Ditches shall be protected with vegetative grass lining (with or without erosion control matting) or non-vegetative linings such as dumped and placed rock gutters, concrete, or grouted rock as required. Choice of the lining method shall depend on physical and economic considerations.

The selection of vegetative lining with or without temporary or permanent erosion control matting shall be based on permissible shear stress criteria. Proper installation of matting is critical to its performance. Each manufacturer provides guidelines on installation. The design drawings shall stress that matting must be installed in strict accordance with the manufacturer's specifications.

Rock linings shall also be designed based on permissible shear stress criteria. Standard drawings for rock linings and concrete gutters are shown in the WVDOH Standard Details. Rock or stone lining should generally not be placed within the highway clear zone. However, it may be permitted to protect existing ditches in the clear zone that have eroded to deep gullies. It also may be permitted where pavement resurfacings have created unsafe, deep ditches provided that the top 6 inches of rock does not have sizes exceeding 4 inches in diameter.

6.3.10 DESIGN METHODS

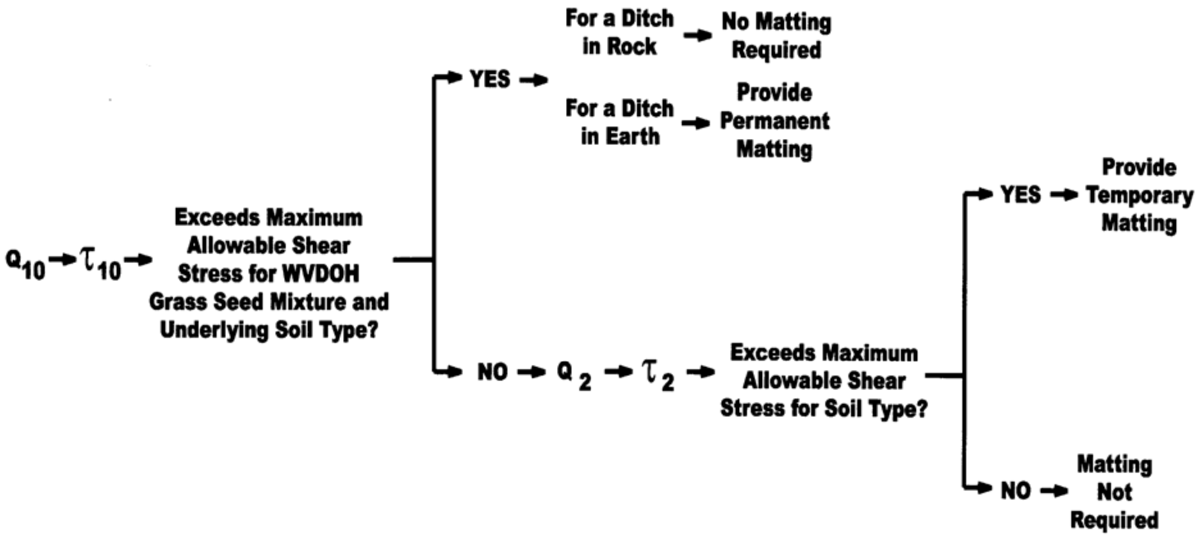
Ditches shall be analyzed for shear stress, depth and minimum velocity at appropriate intervals by solving Manning's equation using a trial and error process. Ditches should typically be analyzed at 50-foot intervals. Additional points of analysis may be required due to change in slope, inflow from other channels or pipes, a change in channel material or sudden increase in drainage area. Form 6-1 shall be used to record the calculations. Ditches may also be analyzed using computer based spreadsheet calculations. Publicly available ditch design charts or nomographs published by state or federal agencies may be used in lieu of spreadsheet calculations provided the sources are documented and the data required in Form DR-6 is provided.

Where design discharges exceed 50 cubic feet per second, the design of rock linings shall be based on FHWA's HEC-11.

The flow chart in Figure 6-1 outlines the ditch analysis method. The ditch shall be checked first to determine if vegetation is adequate as the ditch lining for the 10-year frequency. If vegetation is adequate, the earth-lined ditch as originally constructed shall be checked to see if temporary matting is required during the establishment of vegetation for the 2-year frequency event. It should be noted that even if the ditch is in rock, one side of the ditch consisting of fill will not be able to withstand the shear stresses comparable to rock. If vegetation is not adequate, permanent matting shall be required if the ditch cut is not in rock.

Figure 6-1

Ditch Analysis Flowchart



6.4 DESIGN CONCEPTS AND GUIDELINES

6.4.1 MANNING'S EQUATION

Uniform flow in a channel exists when there is no change of velocity along the channel. Under this condition, the convective acceleration is zero, and the streamlines are straight and parallel. Because the velocity does not change, the velocity head will be constant; therefore, the energy grade line and water surface will have the same slope as the channel bottom. For the flow to be uniform, the channel must be straight and without change in slope or cross section along the length of the channel. Such a channel is called a prismatic channel. When flow is uniform, the depth in the channel is called normal depth. Uniform or nearly uniform flow conditions can be assumed in ditches because they generally have prismatic shapes (triangular or trapezoidal). Therefore, Manning's equation can be used to compute the velocity and normal depth for steady, uniform flow:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

Where:

n = Manning's roughness coefficient

R = Hydraulic Radius = A/P, ft

A = Cross-sectional area of flow, ft^2

P = Wetted Perimeter of the cross-section, ft

S = Slope of the energy grade line in ft/ft

For a given discharge, normal depth is calculated by expressing A in terms of depth (d) and then solving for depth by trial and error. The slope of the energy grade line (S) can be approximated as the channel slope as long as steady, uniform flow conditions can be assumed. This method of calculating normal depth is referred to as the single cross-section method and it can be easily accomplished using a spreadsheet program.

6.4.2 DITCH PROTECTION

Ditch boundary protection is meant to produce stable ditches that are resistant to erosion. The two common protection approaches currently in use are the permissible velocity approach and the permissible shear stress (or tractive force) approach. The more physically based, permissible shear stress approach is preferred by the WVDOH.

6.4.2.1 PERMISSIBLE VELOCITY

The permissible velocity approach dates back to the 1920's and was widely used until the permissible shear stress approach became more popular a few decades ago.

Manning's equation is used to compute the average flow velocity in the ditch for the design storm. The flow velocity is then compared with the maximum permissible velocity for the soil and the proposed lining material in order to design the ditch protection. The ditch is assumed to be stable if the mean velocity in the ditch is lower than the maximum permissible velocity of the boundary material. Table 6-3 presents permissible velocity values for common ditch materials from USACE's Engineer Manual, Hydraulic Design of Flood Control Channels, EM 1110-2-1601.

Table 6-3

Permissible Velocities for Common Ditch Materials

Channel Material	Mean Channel Velocity, ft/s
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel	6.0
Earth	

Sandy Silt	2.0
Silt Clay	3.5
Clay	6.0
Grass Lined Earth (slopes less than 5%)	2.8
Bermuda Grass (resembles DOH seed mixture Type C-2) in	
Sandy Silt	6.0
Silt Clay	8.0
Kentucky Blue Grass	
…(major part of DOH seed mixtures Type B and C-1) in	
Sandy Silt	5.0
Silt Clay	7.0
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or metamorphic)	20.0

The permissible velocity approach is empirical in nature and does not consider the physical process of erosion and detachment of the ditch boundary material. It is recommended that the use of the permissible velocity method be limited to preliminary design only.

6.4.2.2 PERMISSIBLE SHEAR STRESS

The permissible shear stress or tractive force approach became recognized in the 1950's, based on research conducted by the USBR and later developed by the USDA SCS. This method is more physically based and focuses on stresses developed at the interface between flowing water and the materials forming the ditch boundary. Physical factors such as the vegetative stiffness, density, and height, ditch geometry, flow depth, and velocity of flow are taken into account.

Permissible shear stress is the force required to initiate movement of the ditch boundary material. The failure criteria for the lining material are represented by a single shear stress value that is applicable over a wide range of ditch slopes and shapes. The maximum shear stress shall be used for the design of ditch protection and it shall not exceed the permissible shear stress of the selected lining material.

The ditch slope is an important parameter in determining the shear stress and it is generally dictated by the roadway profile. Ditches with slopes steeper than 2% will generally flow in a supercritical state and have higher shear stresses.

Shear stress in a ditch is not uniformly distributed along the wetted perimeter. It is directly proportional to the depth of flow.

The maximum shear stress in a straight ditch occurs on the bed and is given by:

$$\tau_d = \gamma \, d \, S$$

Where: τ_d = Maximum shear stress, lbs/ft^2

γ = Unit weight of water = 62.4 lb/ft^3

d = depth of flow, ft

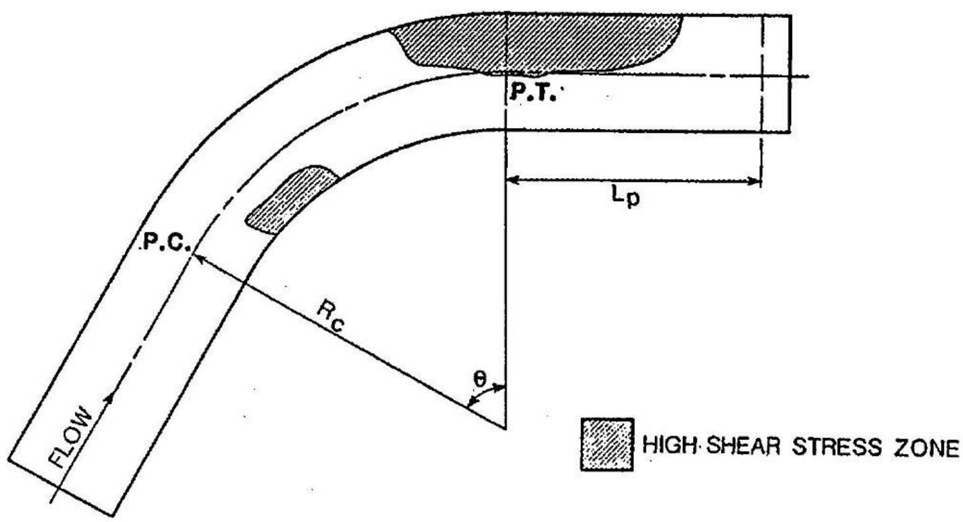
S = Energy slope assumed equal to the ditch average bed slope, ft/ft

Flow in a bend creates secondary currents, which impose higher shear stresses on the side slopes compared to a straight ditch.

As shown in Figure 6-2, at the beginning of the bend, the maximum shear stress is near the inside and then moves towards the outside as flow leaves the bend. The maximum shear stress in a bend is a function of the ratio of ditch curvature to bottom width (R_c/B). As the bend becomes sharper (i.e., as R_c/B decreases), the maximum shear stress in the bend increases as shown in Chart 6-1.

Figure 6-2

Shear Stress Distribution in a Bend



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

The maximum shear stress in a ditch bend is given by:

$$\tau_b = K_b \tau_d$$

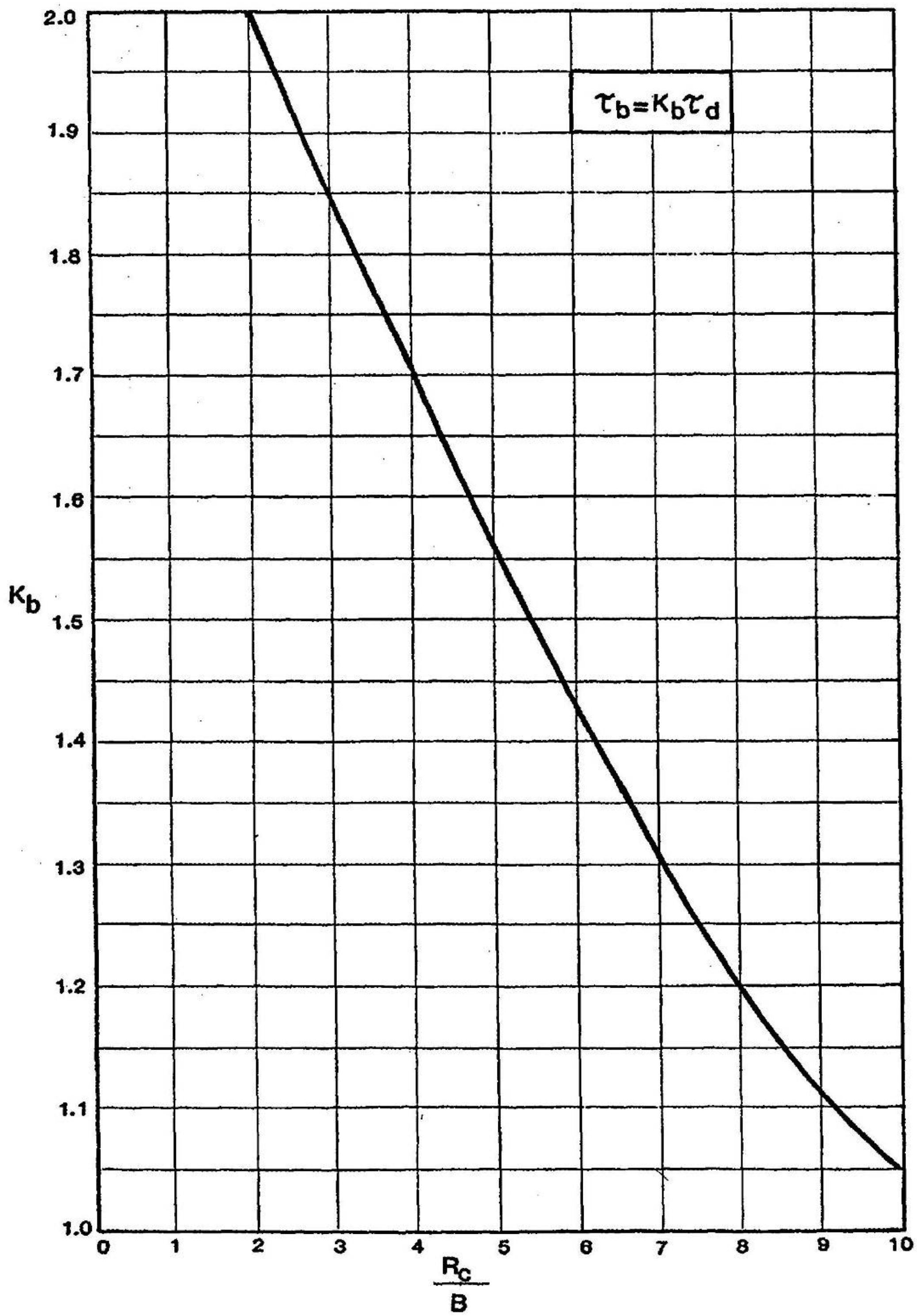
Where: τ_b = Maximum bend shear stress, lbs/ft^2

τ_d = Maximum shear stress in an equivalent straight section of ditch, lbs/ft^2

K_b = Dimensionless factor given by Chart 6-1.

Chart 6-1

K_b for Maximum Shear Stress in Ditch Bends



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

The increased shear stress caused by the bend persists a distance L_p, downstream of the bend.

The protection length downstream of the bend is given by the following equation:

$$\frac{L_p}{R} = 0.604\left(\frac{R^{\frac{1}{6}}}{n_b}\right)$$

Where: L_p = Protection length downstream of bend, ft

R = Hydraulic radius of the ditch, ft

n_b = Manning's roughness coefficient in the bend

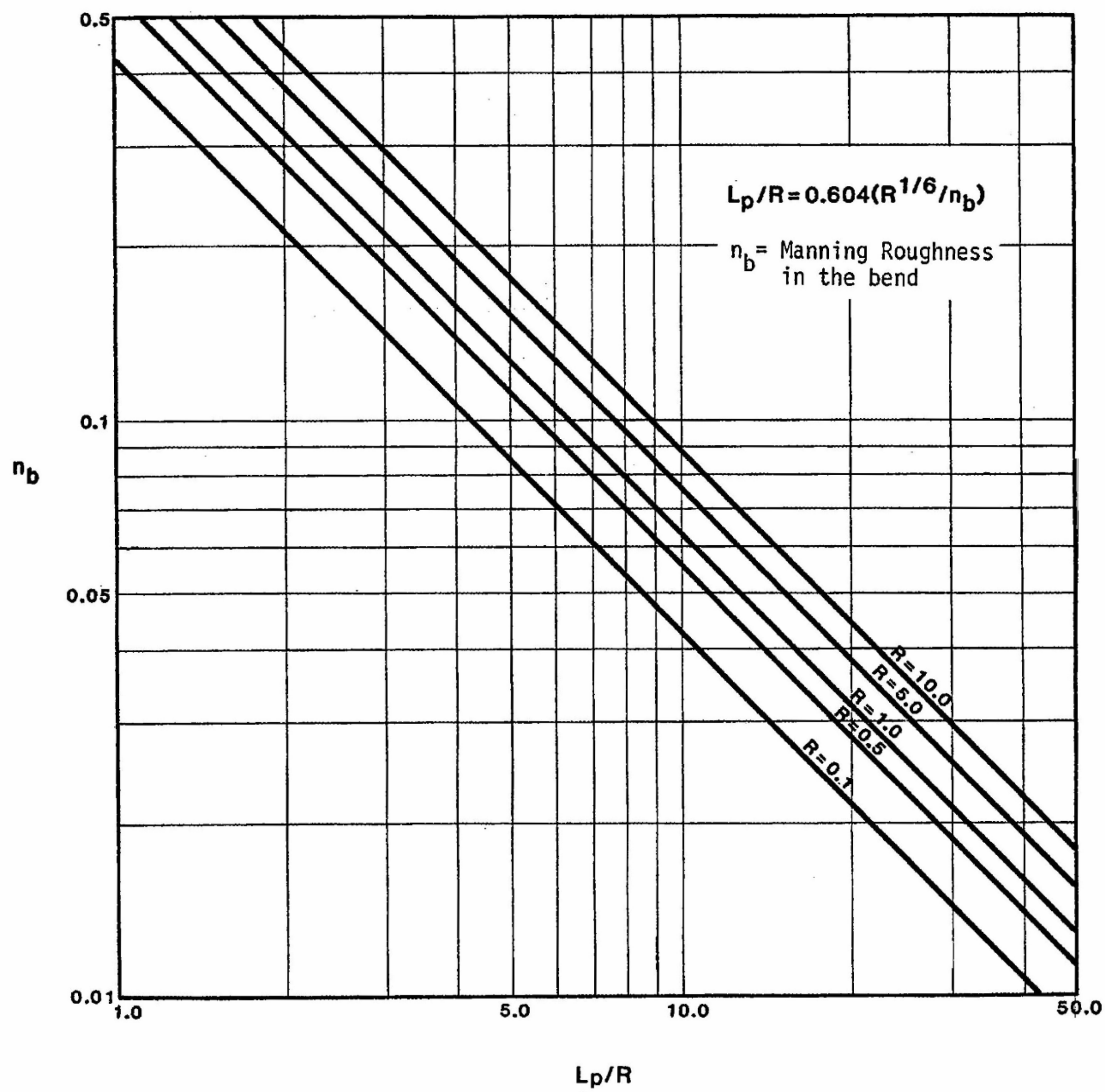
Chart 6-2 can also be used to determine the required protection length downstream of ditch bends.

6.4.2.3 PERMISSIBLE SHEAR STRESS FOR SOIL

A ditch lining moves shear stress away from the soil surface. The remaining shear at the soil surface is termed the effective shear stress. Erosion of the soil boundary occurs when the effective shear stress exceeds the permissible soil shear stress. Permissible soil shear stress is a function of particle size, cohesive strength, and soil density. The erodibility of coarse non-cohesive soils (defined as soils with a plasticity index of less than 10) is due mainly to particle size. Fine grained cohesive soils are controlled mainly by cohesive strength and soil density.

Chart 6-2

Protection Length L_P Downstream of Ditch Bend



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

Cohesive Soils

Cohesive soils are largely fine grained and their permissible shear stress depends on cohesive strength and soil density. Cohesive strength is associated with the plasticity index (PI), which is the difference between the liquid limit and plastic limit of the soil. The soil density is a function of the void ratio. A simplified approach for estimating permissible soil shear stress is illustrated in Figure 6-3.

Figure 6-3

Permissible Shear Stress for Cohesive Soil

			Stress Range lb/ft^2
Cohesive Soil	Fine Grained	10 < PI < 20	0.03 to 0.09
		20 < PI	0.08 to 0.09
	Clay	20 < PI	0.12
	Coarse Grained	10 < PI <20	0.10 to 0.15
		20 < PI	0.15

Plasticity Index

A clay soil can behave like a solid, semi-solid, plastic solid, or liquid, depending on the amount of water in the soil. The water contents corresponding to the transitions between these states are known as the Atterberg Limits. Each of the Atterberg limits varies with the clay content, type of clay mineral, and ions (cations) contained in the clay. The tests that determine the Atterberg plastic limit (PL) and the Atterberg liquid limit (LL) are used to classify a soil. The plastic limit is the amount of water in the soil that transitions it from the semi-solid state to the plastic state. The liquid limit is the amount of water in the soil that transitions it from the plastic state to the liquid state. The difference between the liquid and plastic limits is known as the plasticity index (PI = LL - PL). This index indicates the range in moisture content over which the soil is in a plastic state. In this state the soil can be deformed and still hold together without crumbling. A PI greater than 20 is considered high and indicates that a considerable amount of water can be added before the soil reaches the liquid state. The plastic index correlates with strength, deformation properties, and insensitivity.

The permissible soil shear stress is primarily needed to determine whether temporary matting will be used before vegetation is established. In most cases, the use of temporary matting is good practice; however, in certain cases the existing soil will stand up to the lower return period storm for the temporary situation. This soil stress value is intended to support the judgment of not using temporary matting.

6.4.2.4 PERMISSIBLE SHEAR STRESS FOR SOIL WITH VEGETATION

Grass linings are generally suitable for protecting ditches with gradients up to 10 percent and with side-slopes flatter than 3H:1V. The combined effects of the soil permissible shear stress and the effective shear stress transferred through the vegetative lining results in a permissible shear stress for the vegetative lining.

Table 6-4 provides the permissible shear stress values for the WVDOH Standard Specification seed mixtures. Vegetative ditch linings should consist of seed mixture Type B with the use of Type C-1 in areas viewable and traversable by traffic. Type C-2 may be used in areas next to the roadway in an urban setting.

The computed maximum shear stress value is compared to the permissible shear stress for the selected category of grass lining/seed mixture and underlying soil type. This is accomplished by determining the maximum depth in the ditch and computing the shear stress at the bed. If the permissible shear stress of the selected grass lining and underlying soil type is greater than the computed maximum shear stress at the bed, the grass lining is considered adequate. If the grass lining is determined unacceptable, another seed mixture with a higher permissible shear stress should be selected. The use of permanent erosion control matting should also be considered to supplement the grass lining if shear stress values are at the maximum permissible limit.

Table 6-4

Permissible Shear Stresses for Grass Linings

WVDOH Grass Seed Mixture	Permissible Shear Stress lbs/ft^2	Underlying Soil Type
Type B (mowed)	1.0	non cohesive and cohesive
Type C-1 (mowed)	1.0	non cohesive and cohesive
Type C-2 (mowed)	0.35	non cohesive and cohesive
Type B (unmowed)	3.2	non cohesive
	4.2	cohesive
Type C-1 (unmowed)	3.2	non cohesive
	4.2	cohesive
Type C-2 (unmowed)	0.53	non cohesive
	0.70	cohesive

6.4.2.5 PERMISSIBLE SHEAR STRESS FOR MATTING WITH VEGETATION

The Erosion Control Technology Council (http://www.ectc.org/index.html) refers to Erosion Control Mats as Rolled Erosion Control Products (RECP). RECPs are defined as temporary, degradable or long-term, non-degradable materials manufactured or fabricated into rolls designed to reduce soil erosion and assist in the growth, establishment, and protection of vegetation. Matting is classified as either temporary or permanent based on performance and durability.

Temporary Matting – Temporary, degradable matting is composed of materials that are designed to break down leaving a vegetative lining behind. They are typically constructed of straw, jute, and coconut fibers in one or two layers or nets. Pure straw matting is not recommended for use in ditch design.

There are two types of temporary matting that are acceptable for use in ditches. Each is classified by their effective lifespan and permissible shear stress. The first type usually consists of a 70% straw to 30% coconut fiber matrix that is stitched together with a biodegradable thread between two layers of biodegradable jute netting. This type typically has an effective life span of about 18 months. The second type usually consists of a 100% coconut fiber inner layer between two layers of biodegradable jute netting. This type typically has a slightly longer effective life span of about 24 months.

The permissible shear stress value for the permanent condition shall be the same as that for the vegetation and underlying soil after matting has biodegraded. The design shear stress for the temporary condition shall be that caused by a discharge for a two year recurrence interval storm (see Figure 6-1). The matting shall resist the shear stress for this temporary condition. Table 6-5 gives the permissible shear stress values for the two types of acceptable temporary matting.

Table 6-5

Permissible Shear Stresses for Temporary Matting

Matting Type	**Permissible Shear Stress for Temporary Matting lbs/ft^2**
Straw and Coconut matrix	2.0
Coconut only layer	2.2

Permanent Matting - Long term, non-degradable or permanent matting is composed of materials that are designed to enhance vegetative growth and extend the erosion control performance of grass linings. WVDOH permanent matting types are covered in the standard specifications in section 715.24. Each type is specified by ASTM test methods describing minimum mat thickness, tensile strength, elongation, and porosity along with resiliency and ultraviolet stability. Acceptance of permanent matting for use on a project shall be based upon meeting the state specification and provided certified test data justifying adherence to those specifications.

Permissible shear stress values for each permanent matting type in vegetated and non-vegetated states are given in Table 6-6. Permissible shear stress for the non-vegetated state is considerably less than the values for the vegetated state and will usually control the design. When the maximum shear stress exceeds the maximum permissible shear stress of the grass lining with permanent matting, stronger materials such as rock or concrete lining should be considered.

Table 6-6

Permissible Shear Stresses for Permanent Matting

Matting Type	Permissible Shear Stress for Permanent Matting lbs/ft^2	
	Vegetated (Evaluate for Q_{10})	**Non-vegetated (Evaluate for Q_2)**
Type A	4	1
Type B	6	1.5
Type C	8	2

While matting has been shown to be effective in protecting vegetated ground surfaces, it is not appropriate in all locations. Matting should not be used in:

- Ditches with continuous flow
- Unfertile soils
- Non-vegetated applications
- Severe slopes
- Shoreline applications with high wave action
- Rocky soils where anchorage is difficult
- Uneven surfaces where ground contact of the matting will not be continuous
- Ditches where maintenance is likely due to sediment build up. Pulling ditches with matting can result in loss of contact with the soil or loss of the matting completely.

It is important that the installation of temporary or permanent matting follow the manufacturers' guidelines. The permissible shear stress values depend upon proper installation.

6.4.2.6 PERMISSIBLE SHEAR STRESS FOR ROCK LINING

Ditches within rock cut sections shall not require a determination of a permissible shear stress.

Rock protection for ditches with gradients less than 10 percent should be based on the permissible shear stress discussed in this section. The permissible shear stress values in Table 6-7 should be used to obtain the required mean diameter of the lining gradation (D_{50}) for rock linings in ditches with side-slopes equal to or flatter than 3H:1V. These permissible values are compared to the maximum shear stress along the bottom of the ditch (see Section 6.4.2.2).

Table 6-7

Permissible Shear Stresses for Rock Linings

Rock (D_{50}) inches	**Permissible Shear Stress lbs/ft^2**
4	1.6
6	2.5
12	5.0

The shear stress on the sides of the ditch is less than the maximum shear stress occurring on the ditch bottom. The stability of a side slope lining is a function of the side slope and the angle of repose of the lining material. This essentially results in a lower permissible shear stress on the side slope than on the bottom. These two counterbalancing effects lead to the following equation to obtain the required mean diameter of the lining gradation (D_{50}) for rock linings in ditches with side-slopes steeper than 3H:1V.

$$\left(D_{50}\right)_{SIDES} = \frac{K_1}{K_2}\left(D_{50}\right)_{BOTTOM}$$

Where: K_1 = side to bottom shear stress ratio

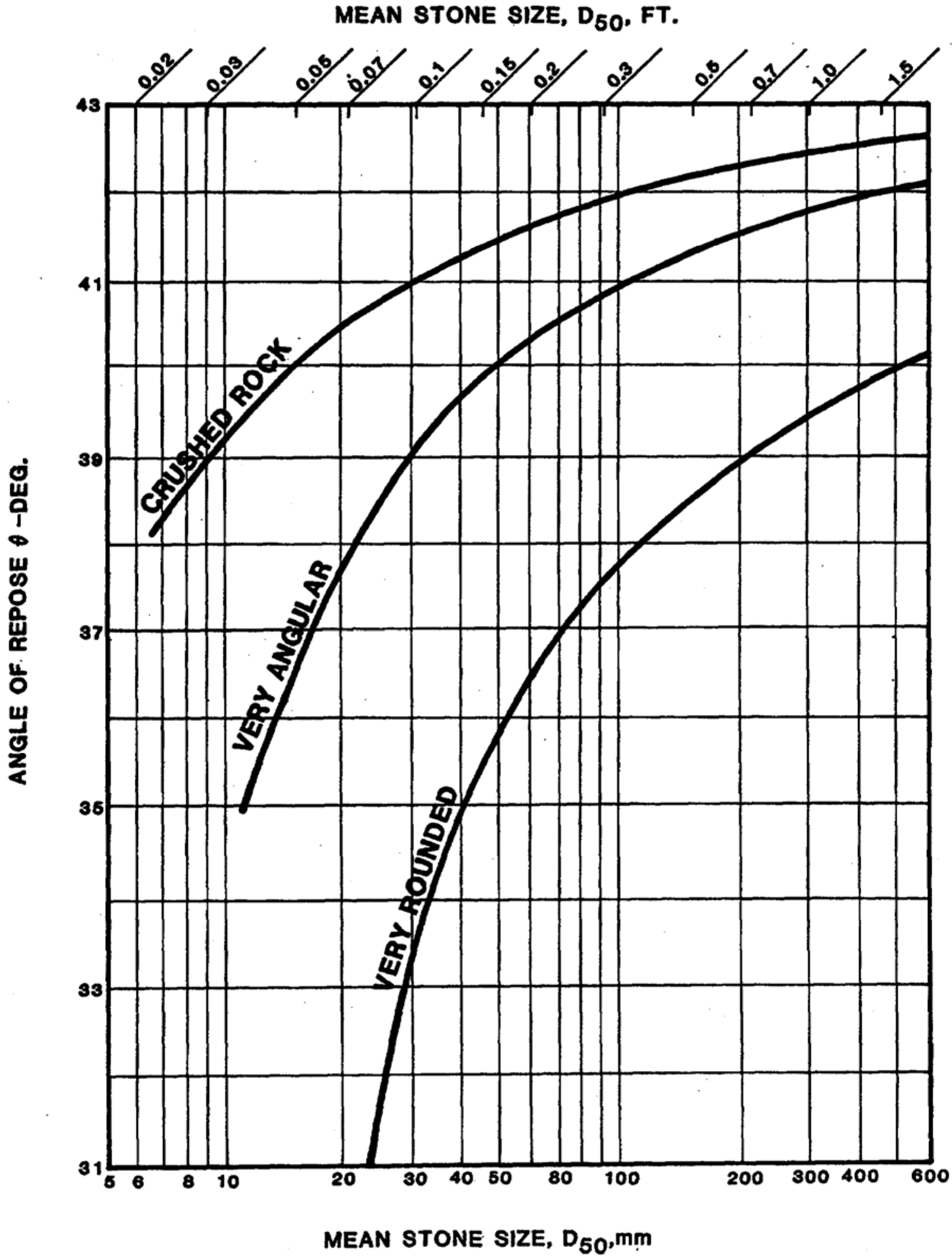
K_2 = tractive force ratio

The mean diameter of the lining for the bottom of the ditch is obtained from Table 6-6 by comparing to the maximum shear stress.

The angle of repose for different rock shapes and sizes is presented in Chart 6-3. Chart 6-4 is used to obtain the ratio of the shear stress on the side-slopes to the shear stress on the bottom of the ditch (K_1). Values needed to obtain this shear stress ratio are the ratio of the bottom width of ditch to flow depth and the ditch side-slope. Chart 6-5 is used to obtain the tractive force ratio (K_2) based on the angle of the side-slope and the angle of repose of the rock lining. Rock should be graded so that it follows a smooth size distribution curve and the spaces between the larger stones are filled in an interlocking fashion by the smaller stones. Most gradations that fall in the range of D_{100}/D_{50} = 3.0 and D_{50}/D_{20} = 1.5 are considered acceptable.

Chart 6-3

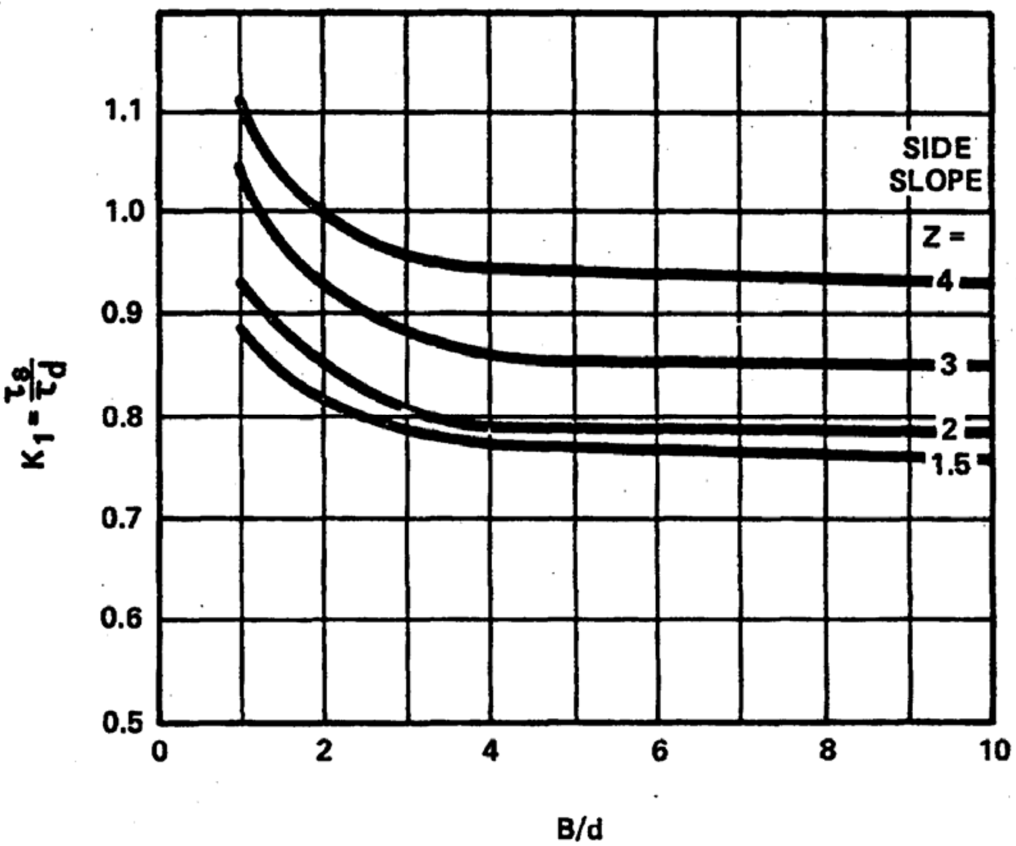
Rock Lining: Angle of Repose



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

Chart 6-4

Rock Lining: Side to Bottom Shear Stress Ratio K_1



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

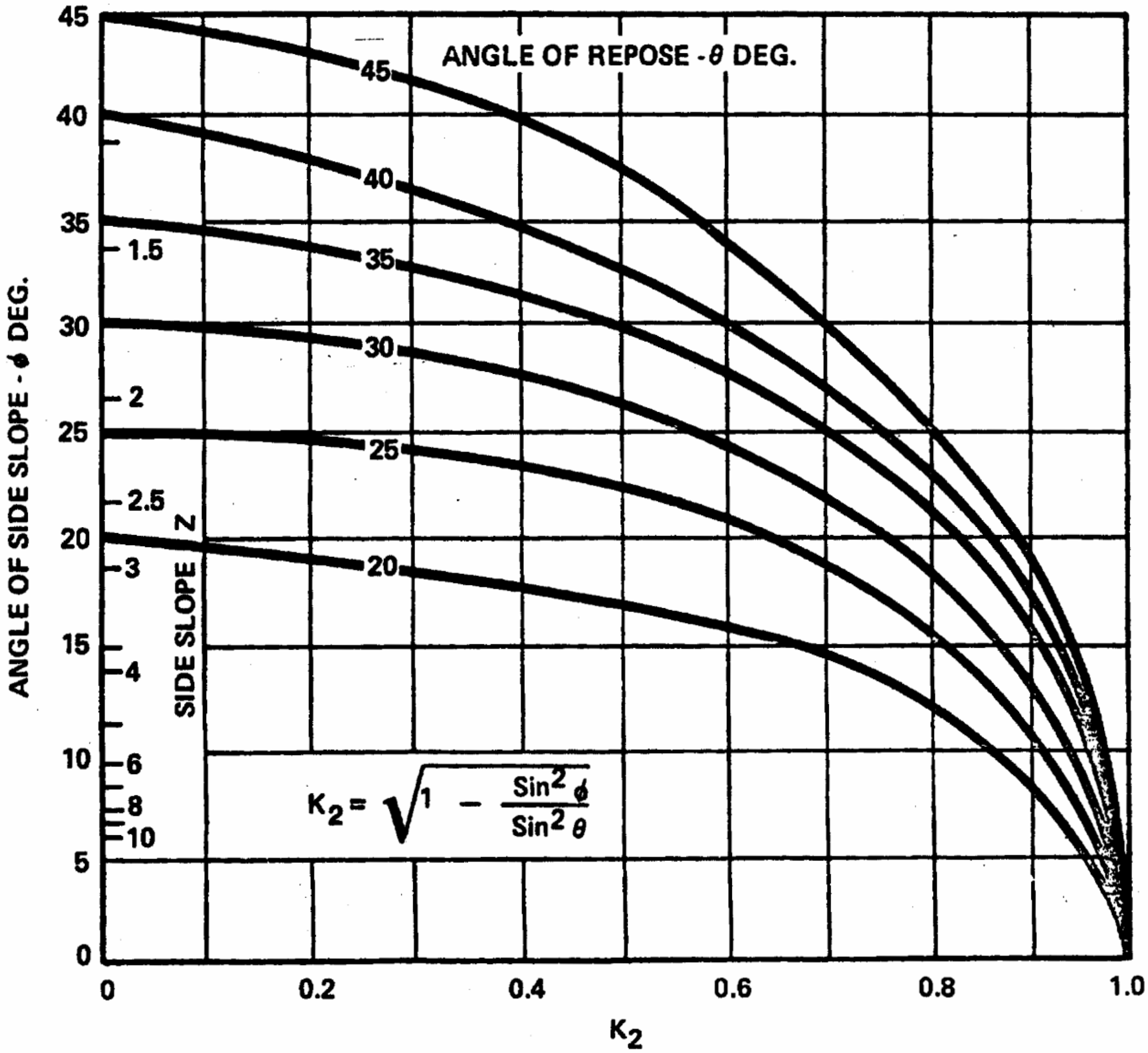
The rock should be angular rather than rounded, flat, or slab-like. An approximate guide to a stone shape is that neither the breadth nor the thickness should be less than one-third its length.

The thickness of the rock lining should equal the diameter of the largest rock size in the gradation. For most gradations, this will mean a rock lining thickness equal to1.5 to 3.0 times the mean rock diameter (D_{50}).

An engineering filter made of geotextile fabric should be placed between the underlying soil and the rock lining to prevent the migration of fine soil particles. This filter will also permit relief of hydrostatic pressures within the underlying soils. Engineering filter fabric is available though several commercial manufacturers and it should be selected and installed in strict accordance with the manufacturer's specifications.

Chart 6-5

Rock Lining: Tractive Force Ratio K_2



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

WVDOH standard engineering fabric for separation (spec 715.11) is required to meet the AASHTO Standard Material Specification for Subsurface Drainage Geotextile Fabrics (M288-00). This specification requires the following properties and performance characteristics:

(1) The fabric must be able to transmit water faster than the soil.

(2) The fabric should satisfy the following criteria for the Apparent Opening Size (AOS):

a. AOS<0.6 mm (0.024 inches) for soil with 50 percent of the particles by weight passing a U.S. 200 sieve.

b. AOS<0.297 mm (0.012 inches) for soil with more than 50 percent of the particles by weight passing a U.S. 200 sieve.

Granular rock filter may be used in lieu of an engineering fabric filter. The designer should refer to FHWA's latest version of HEC-15 for the design procedure for granular rock filters.

6.4.2.7 ROCK LINING IN DITCHES WITH STEEP GRADIENTS

Rock protection for ditches with gradients greater than or equal to 10 percent is subject to additional forces and requires special design consideration. The size of rock increases quickly as the channel gradient and discharge increase. For a given discharge, steep sloped ditches require larger stones to compensate for the larger forces in the flow direction and the higher shear stresses.

Bends should be avoided on steep gradient ditches. If the bend cannot be eliminated, a culvert should be used instead of the ditch in the bend section.

Extent of rock protection on a steep gradient ditch must be sufficient to protect transitions both above and below the steep gradient section. The transition from a mild gradient to a steep gradient ditch should be protected against local scour upstream of the transition for a distance of approximately 5 times the flow depth in the downstream channel. Transitions from a steep gradient ditch to a mild gradient ditch may require an energy dissipation structure.

Rock lining gradation, thickness, and filter requirements are the same as those for mild slopes.

The design procedure for steep slope rock lining design is as follows:

1. For a given discharge and channel slope, enter Chart 6-6, Chart 6-7, Chart 6-8, or Chart 6-9 for the correct ditch shape and determine the flow depth and mean stone size. For ditch widths not provided in the charts, interpolate between charts to find the correct value.
2. To determine flow depth and stone size for side slopes greater than 3H:1V, use the following steps:
   a. Find the steep ditch section flow depth ($d_{z:1}$) using the formula:

$$\frac{d_{z:1}}{d_{3:1}} = \frac{A_{3:1}}{A_{z:1}}$$

Where:

$d_{3:1}$ = flow depth using Chart 6-6 to Chart 6-9 for side slopes equal to or flatter than 3H:1V, ft

$A_{3:1}/A_{z:1}$ = ratio of the flow area for side slopes equal to or flatter than 3H:1V to the flow area for side slopes greater than 3H:1V.

The value of the $A_{3:1}/A_{z:1}$ ratio is found from Table 6-8.

The subscript z refers to the steeper side slope horizontal value.

In order to obtain $A_{3:1}/A_{z:1}$ you must use the ratio of $d_{3:1}/B$.

Where:

B = width of channel bottom, ft

b. Find the rock size using the formula:

$$D_{50} = \frac{d_{z:1}}{d_{3:1}} D_{50\ 3:1}$$

$D_{50\ 3:1}$ represents the mean diameter of rock using Chart 6-6 to Chart 6-9 for side slopes equal to or flatter than 3H:1V.

Table 6-8

A_3/A_z Values for Various Side Slopes and d/B Ratios

d/B	A3/AZ 2:1	3:1	4:1	5:1	6:1
0.10	1.083	1.000	0.928	0.866	0.812
0.20	1.142	1.000	0.888	0.800	0.727
0.30	1.187	1.000	0.853	0.760	0.678
0.40	1.222	1.000	0.846	0.733	0.647
0.50	1.250	1.000	0.833	0.714	0.625
0.60	1.272	1.000	0.823	0.700	0.608
0.70	1.291	1.000	0.815	0.688	0.596
0.80	1.307	1.000	0.809	0.680	0.586
0.90	1.321	1.000	0.804	0.672	0.578
1.00	1.333	1.000	0.800	0.666	0.571
1.10	1.343	1.000	0.796	0.661	0.565
1.20	1.352	1.000	0.793	0.657	0.561
1.30	1.361	1.000	0.790	0.653	0.556
1.40	1.368	1.000	0.787	0.650	0.553
1.50	1.378	1.000	0.785	0.647	0.550
1.60	1.381	1.000	0.783	0.644	0.547
1.70	1.386	1.000	0.782	0.642	0.544
1.80	1.391	1.000	0.780	0.640	0.542
1.90	1.395	1.000	0.779	0.638	0.540
2.00	1.400	1.000	0.777	0.636	0.538

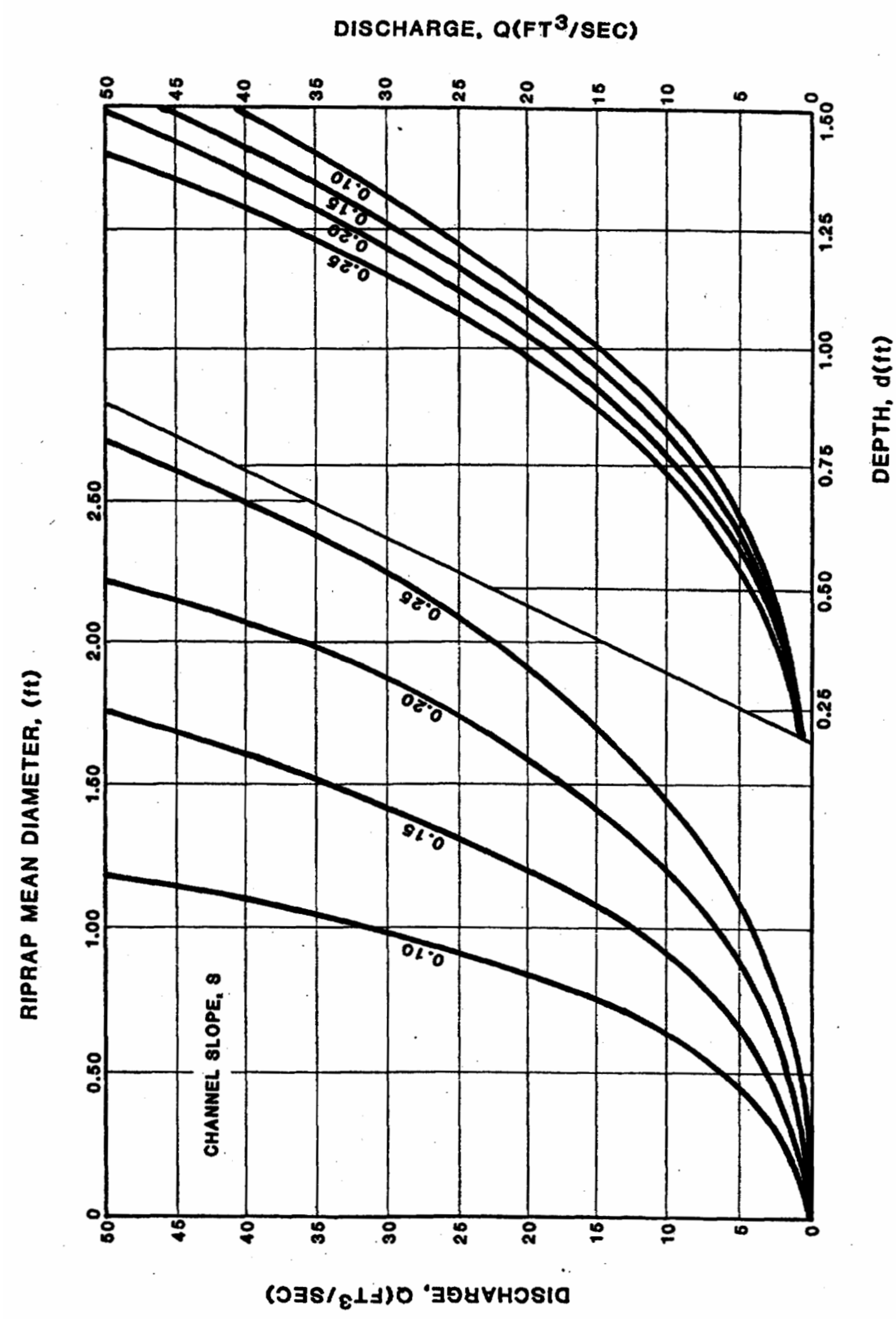
Based on the following equation:

$$A_3/A_Z = \frac{1 + 3(d/B)}{1 + Z(d/B)}$$

Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

Chart 6-6

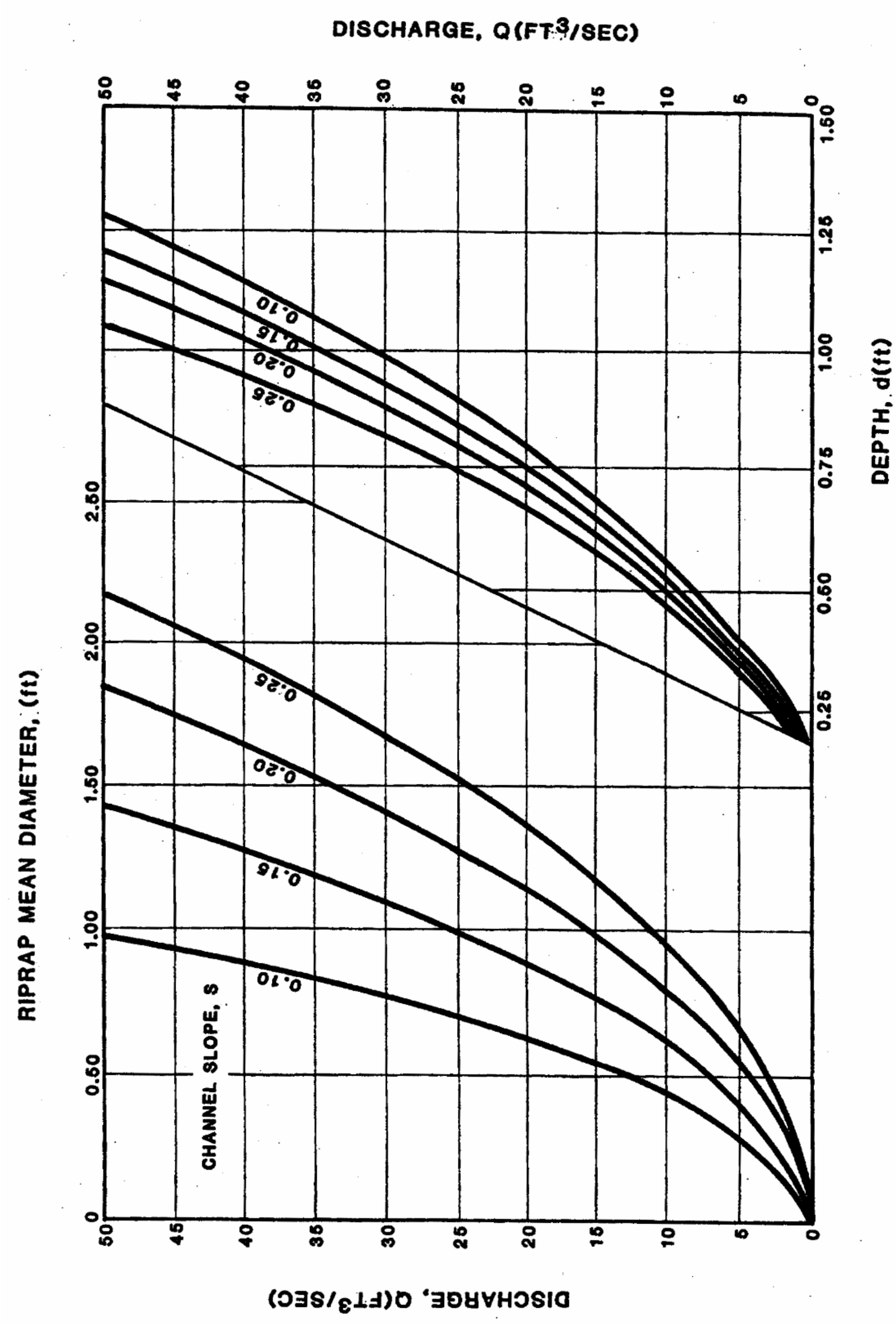
Steep Slope Rock Lining, Triangular Ditch, Z=3



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

Chart 6-7

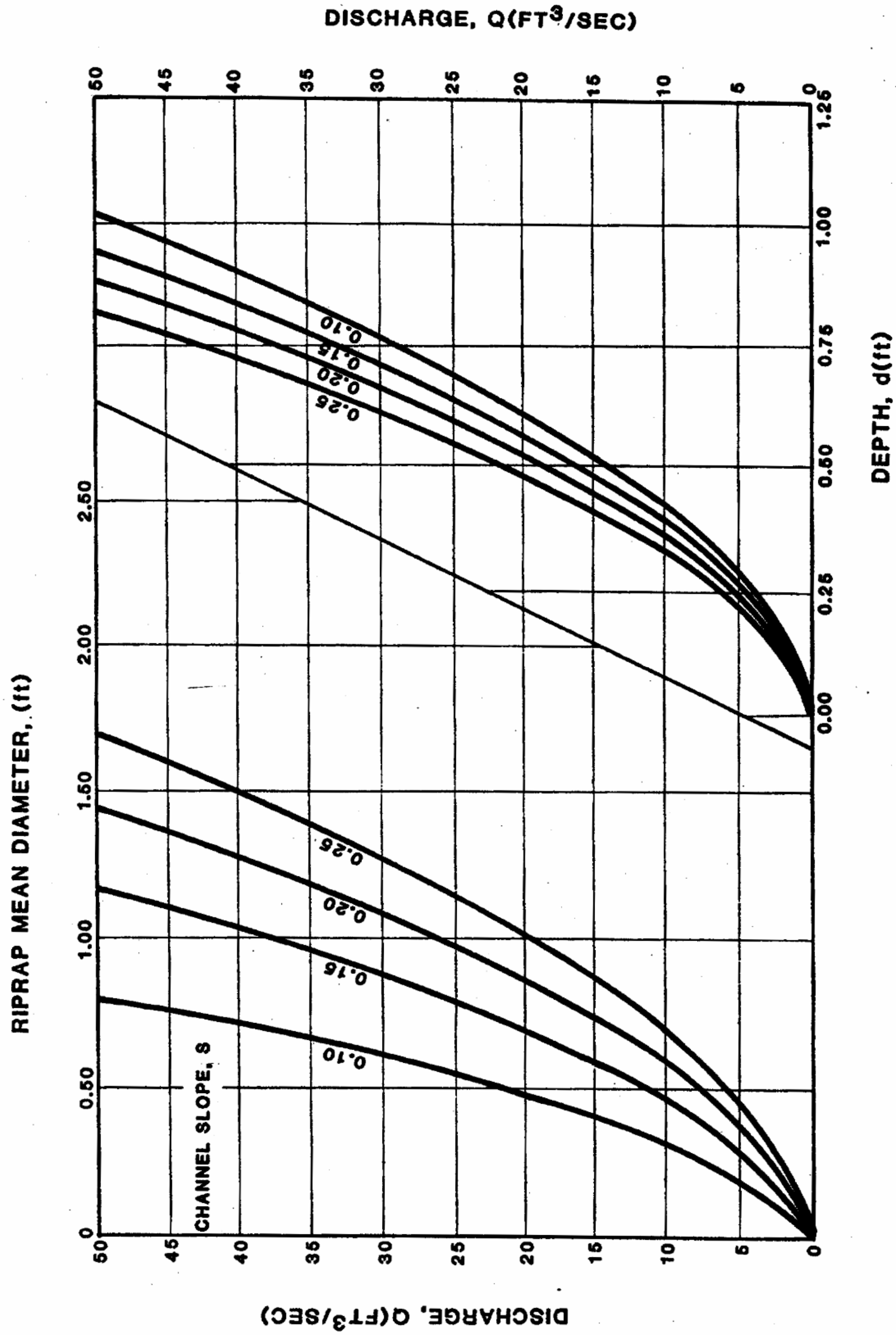
Steep Slope Rock Lining, B=2, Z=3



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

Chart 6-8

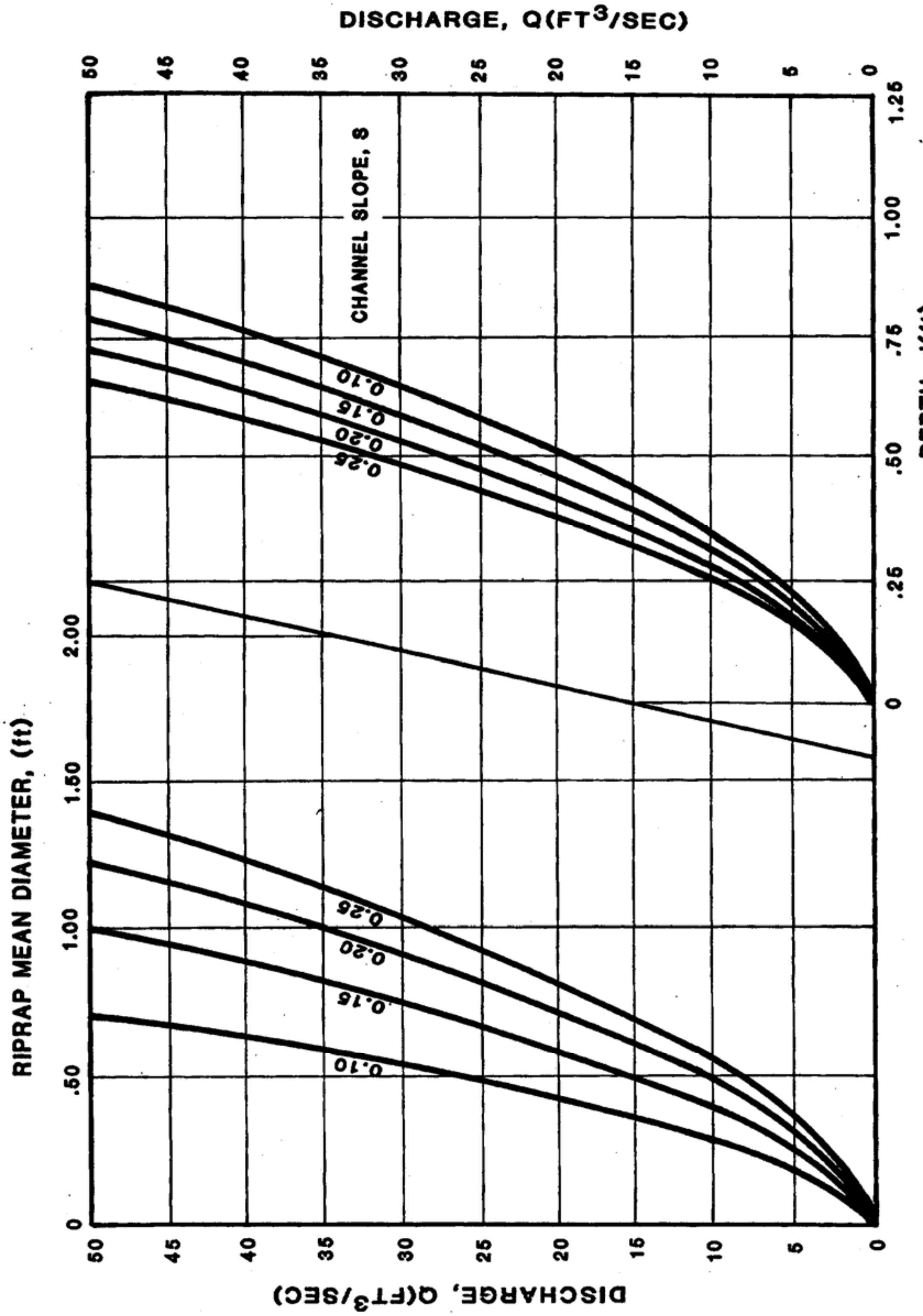
Steep Slope Rock Lining, B=4, Z=3



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

Chart 6-9

Steep Slope Rock Lining, B=6, Z=3



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

6.4.3 DITCH INLET CAPACITY

It is important to note that ditches are designed for a return period of 10 years; therefore, the grate inlet capacity should be based on the 10-year storm. As stated in Section 6.3.6, ditch inlets use Type G grates. These inlets are installed with a 1-foot high mound down grade of the grate at a distance of eight to ten feet (see WVDOH standard details and Table 6-9). This installation results in the determination of the inlet capacity as if it were in a sag vertical curve. See Section 5.3.4.8 in Chapter 5 for an explanation.

Chart 6-11 through Chart 6-24 were developed from equations in HEC-22. These charts provide the interception capacity for standard Type G grates ranging in size from 32” X 38” to 60” X 66” according to the average depth of flow over the grate. The average depth is used in case the choice is made to install the grate on the incline of the mound. This type of installation may be useful for a roadside ditch in an area where a large amount of natural litter is present.

The amount of available ponding depth should be checked for each inlet as this will depend on the grade of the ditch line it resides in (see Figure 6-4). In most cases, the maximum allowable depth within the ditch (see Section 6.3.3) or the maximum allowable inlet spacing will control the ponding at the inlet. However, the ponding depth should be checked to ensure that it does not adversely affect the roadway pavement subgrade. Table 6-9 shows the amount of available ponding depth for ditch slopes of 0.5% to 7.5%. A depressed inlet may be installed in order to provide additional ponding depth. For slopes greater than 7.5% a special detail for the inlet mound will be needed as the standard detail provides little to no ponding depth at such steep slopes.

Figure 6-4

Profile View of the Ponding Depth at a Type G Inlet

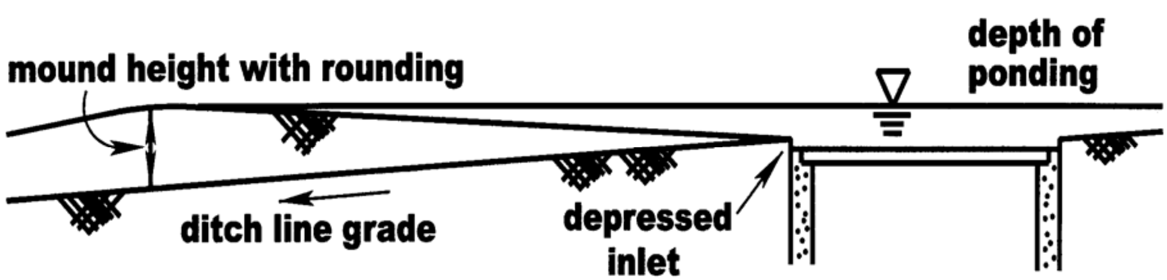


Table 6-9

Type G Inlet Available Ponding Depth

Ditch Grade (percent)	Horizontal Distance to Top of Mound (feet)	Available Ponding Depth (0.5” loss due to mound rounding) (inches)
0.5	10	10.9
1	10	10.3
2	10	9.1
3	10	7.9
4	9	7.2
5	9	6.1
6	8	5.7
7	8	4.8
7.5	8	4.3

Overtopping of the inlet mound shall not be allowed, but it may be calculated for a check storm. This overtopping discharge would be calculated using equations for flow over a broad crested weir according to the depth of flow over the mound. The capacity charts can be used to determine the ponding depth that causes the mound overflow.

In a case where a Type G inlet is used in a rural roadside ditch or an urban ditch without a mound down grade, it is treated as an inlet on grade (meaning bypass flow can occur). The ditch flow is primarily all frontal flow if the width of the grate is equal to the bottom width of the ditch. If the width of the ditch bottom is wider than the width of the grate there will be some side flow (see Figure 6-5).

As stated in Chapter 5 (see Section 5.3.4.4), the total intercepted flow capacity of an inlet grate is:

$$Q_i = E\,Q$$

With the total efficiency as:

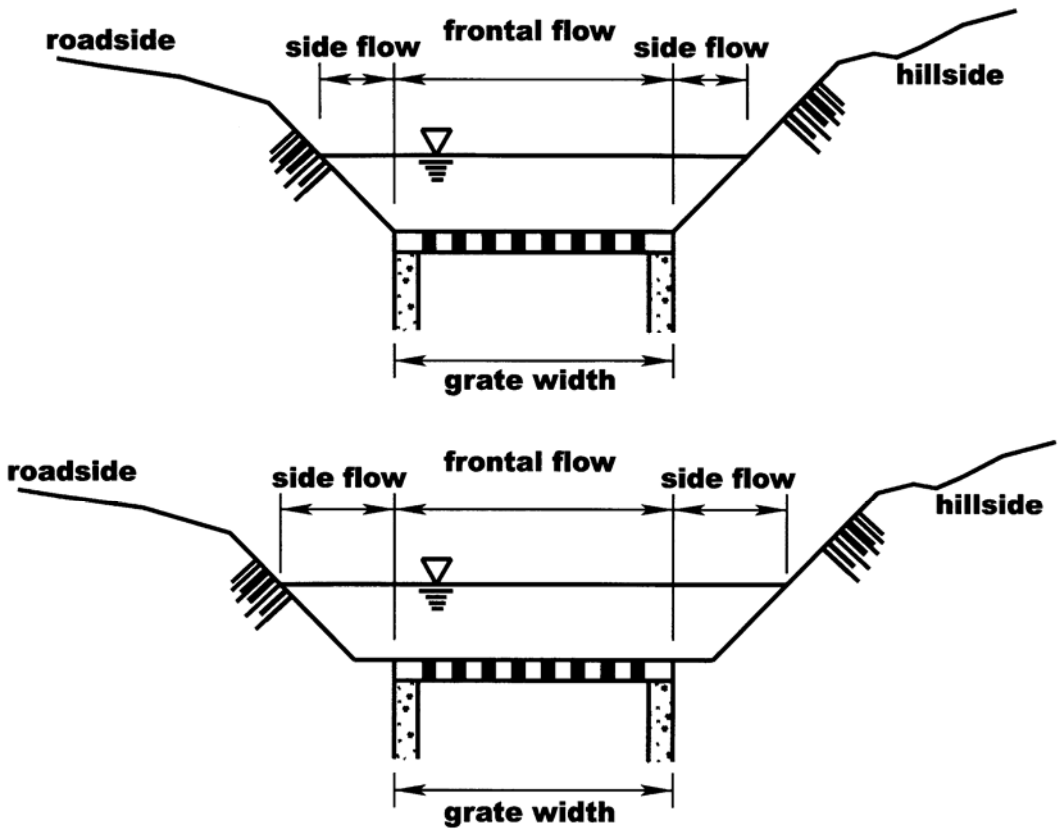
$$E = R_f\ E_o + R_S\left(1 - E_o\right)$$

Where: R_f = ratio of frontal flow intercepted to total frontal flow

R_s = ratio of side flow intercepted to total side flow

Figure 6-5

Frontal and Side Flow for Type G Inlets without the Mound



The interception efficiencies for frontal and side flow for Type G grates are obtained from Chapter 5, Chart 5-7 and Charts 5-8, respectively. The ratio of frontal flow to total gutter flow (E_o) for a trapezoidal channel is shown in Chart 6-10 and obtained from the following equation:

$$E_o = W / \left(B + d\ z\right)$$

Where: W = width of the grate, ft

B = bottom width of the channel, ft

d = depth of flow, ft

z = horizontal distance of side slope to a rise of 1 ft vertical, ft

For a trapezoidal channel, Manning's equation becomes:

$$Q = \frac{1.486}{n}\left(Bd + z\,d^2\right)\left(\frac{Bd + z\,d^2}{B + 2\,d\,\sqrt{z^2+1}}\right)^{0.67} S_L^{0.5}$$

Where: n = roughness coefficient

S_L = bed slope, ft / ft

This equation can be used to solve for depth of flow knowing the flow amount at any point in the ditch (usually obtained by the Rational Method). See Section 5.3.4.4 in Chapter 5 for more specific information on calculating inlet capacity on grade using frontal (R_f) and side (R_s) flow efficiencies.

The flow area will be needed to determine the average velocity within the ditch. It is calculated as the area of a trapezoid which is a four sided shape consisting of two parallel sides. The average velocity is required in order to determine the frontal flow and side flow efficiencies R_f and R_S, respectively.

$$A = \frac{1}{2}\,d\,\left(B + T\right)$$

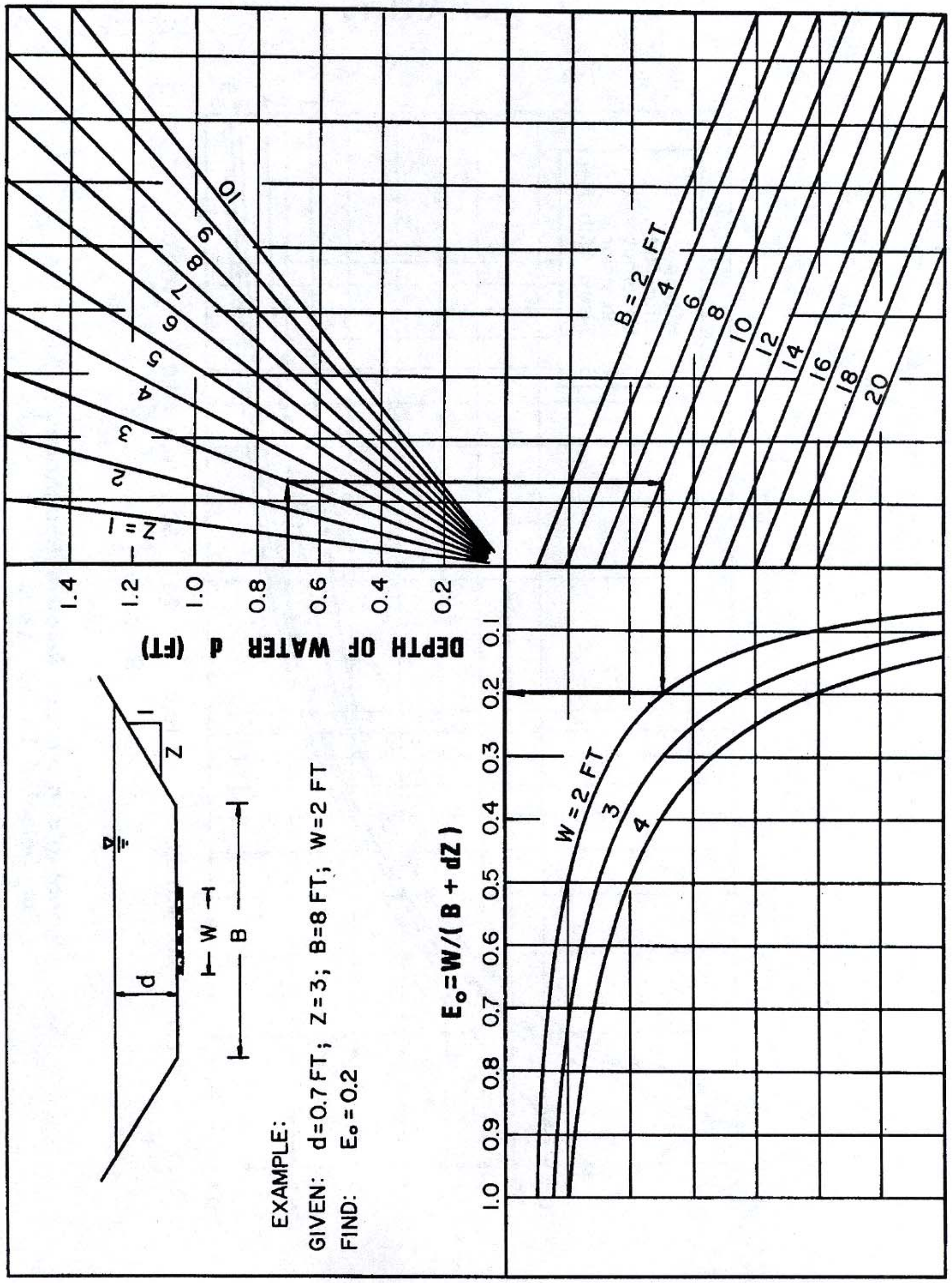
Where: T = top width of the flow, ft

Flanking inlets may be necessary if the roadside ditch alignment has a sag vertical curve and the mounding detail is not used. The designer shall examine the possibility of flooding the roadway subgrade if the grate should become completely blocked at the sag. The flanker inlets will most likely be the same as the sag inlet for grate installations without the mounding detail. See Section 5.3.4.10 in Chapter 5 for more information.

As stated in Section 5.3.4.7 a grate inlet in a sag location operates as a weir to depths that are dependant on the size of the grate and as an orifice at greater depths. Charts 6-12 through 6-24 give the value for interception capacity (Q_i) for an average depth over the grate (d_{AV})in weir flow based on the effective perimeter (P) in feet and in orifice flow based on the effective clear open area of the grate (A) in square feet.

Chart 6-10

Frontal Flow / Total Flow for a Trapezoidal Channel



Ratio of Frontal Flow to Totl Flow in a Trapezoidal Channel - English Units

Source: Urban Drainage Design Manual, HEC-22, FHWA, August 2001

Chart 6-11

32” X 38” Type G grate inlet with mounding detail (Type 1 grate)

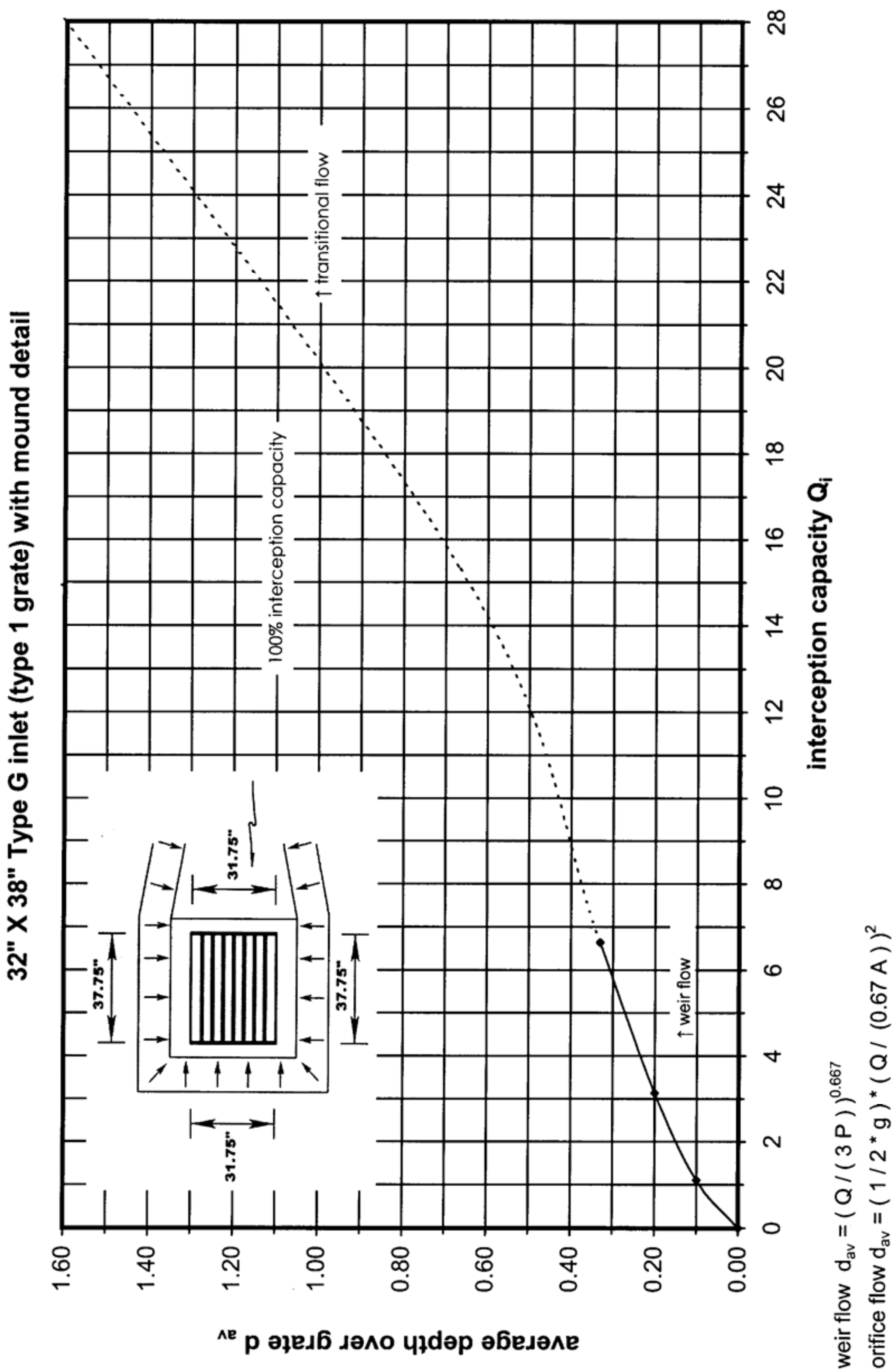


Chart 6-12

36” X 42” Type G grate inlet with mounding detail (Type 1 grate)

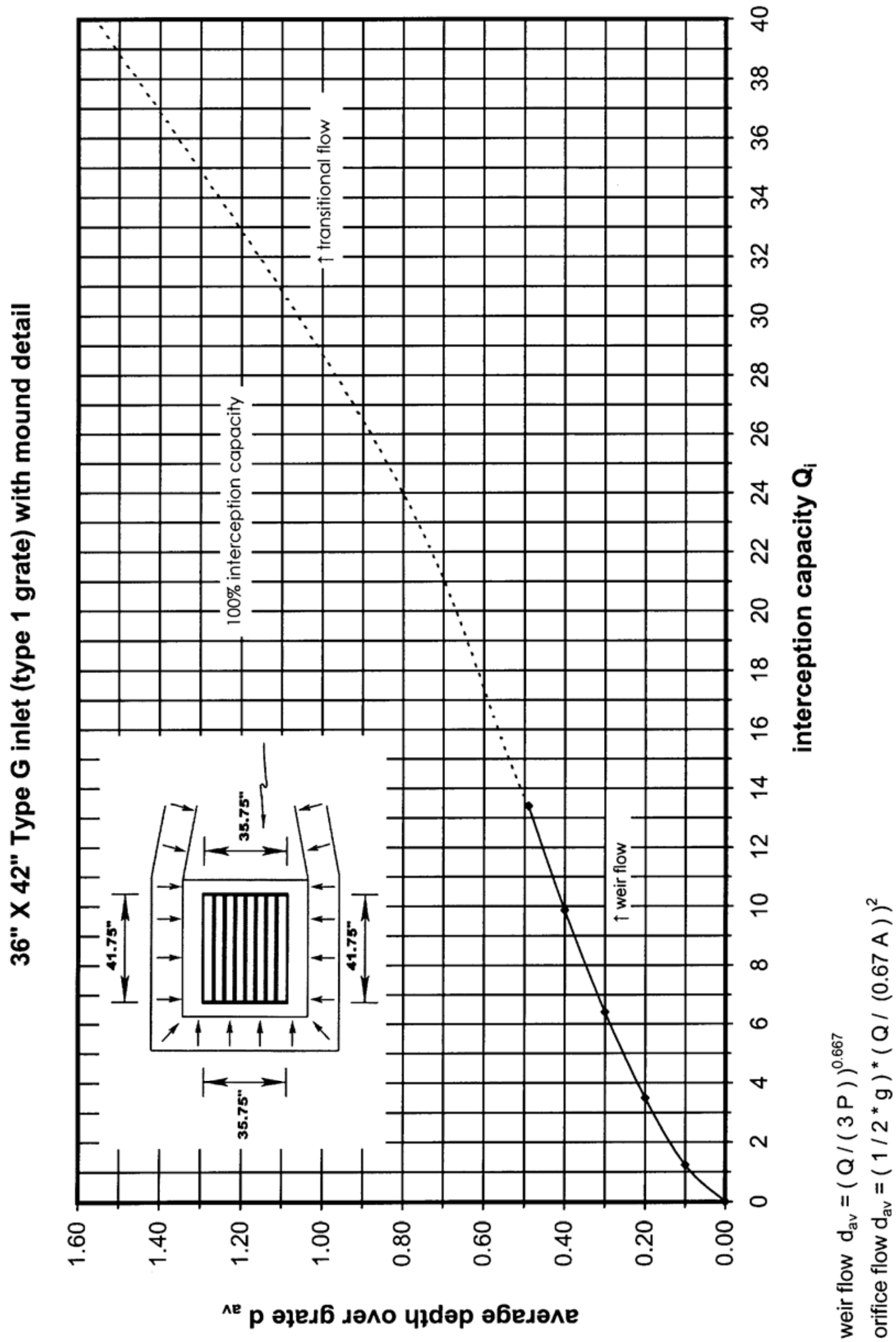


Chart 6-13

42” X 48” Type G grate inlet with mounding detail (Type 1 grate)

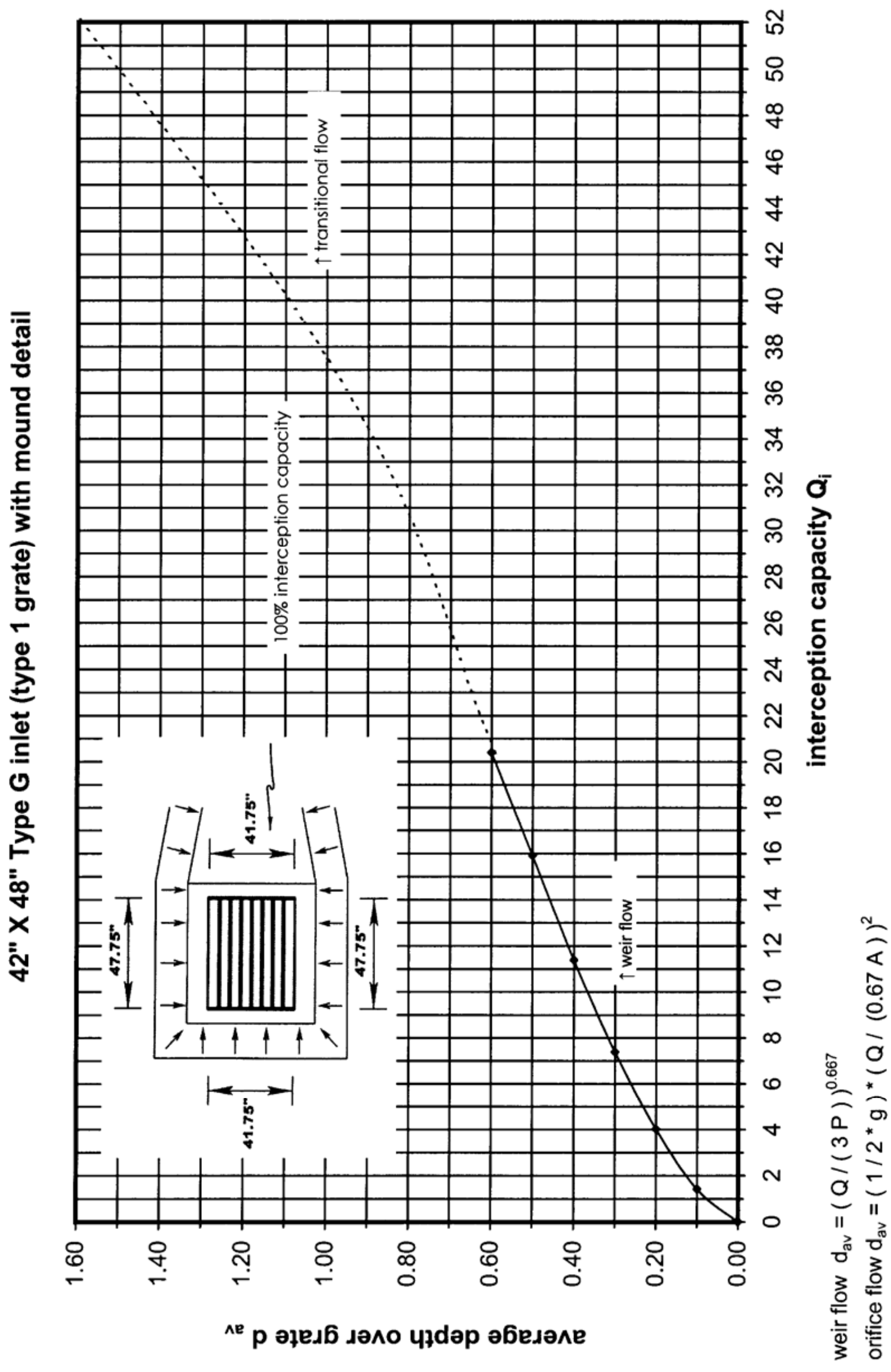


Chart 6-14

45” X 51” Type G grate inlet with mounding detail (Type 1 grate)

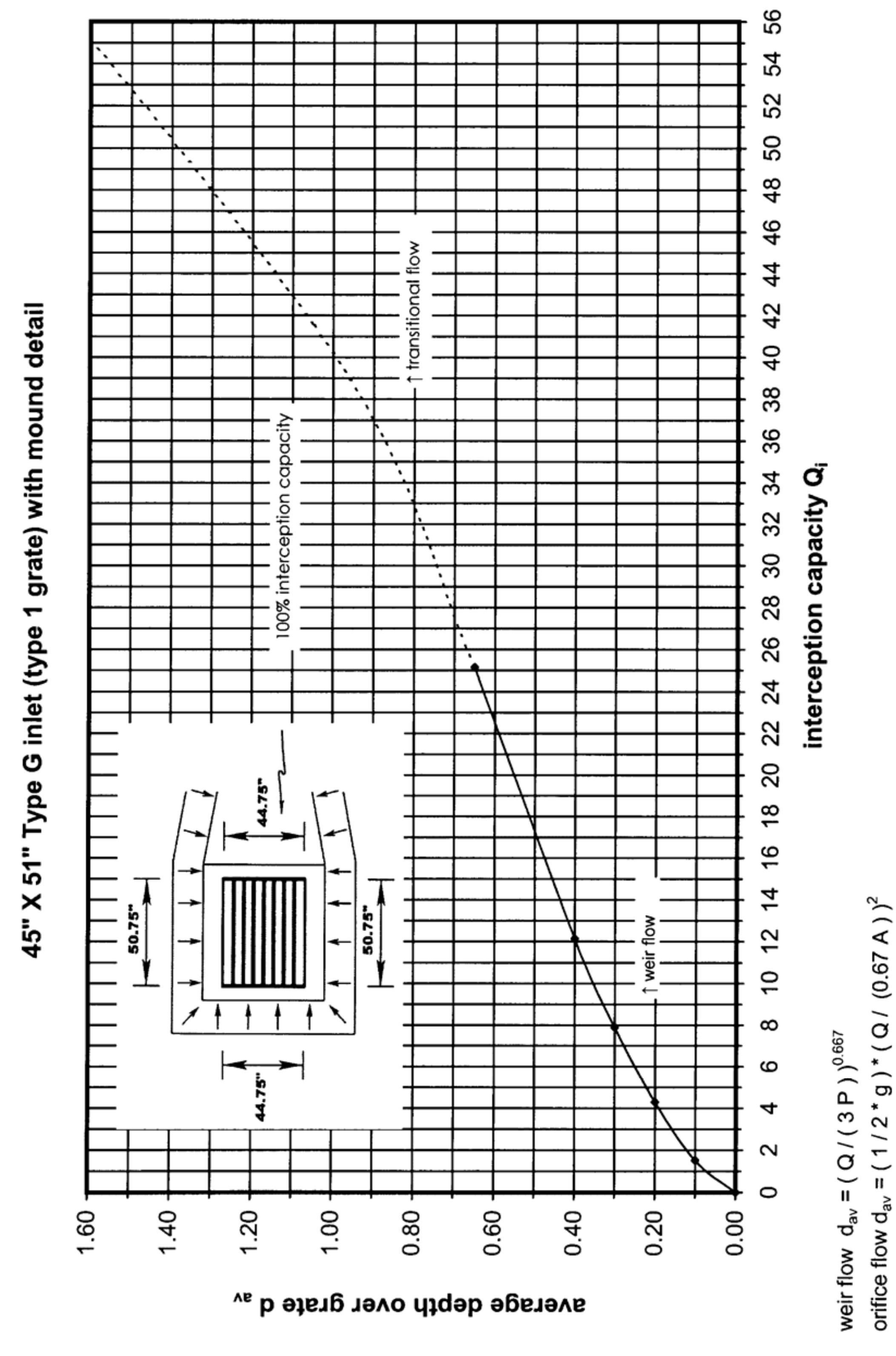


Chart 6-15

48” X 54” Type G grate inlet with mounding detail (Type 1 grate)

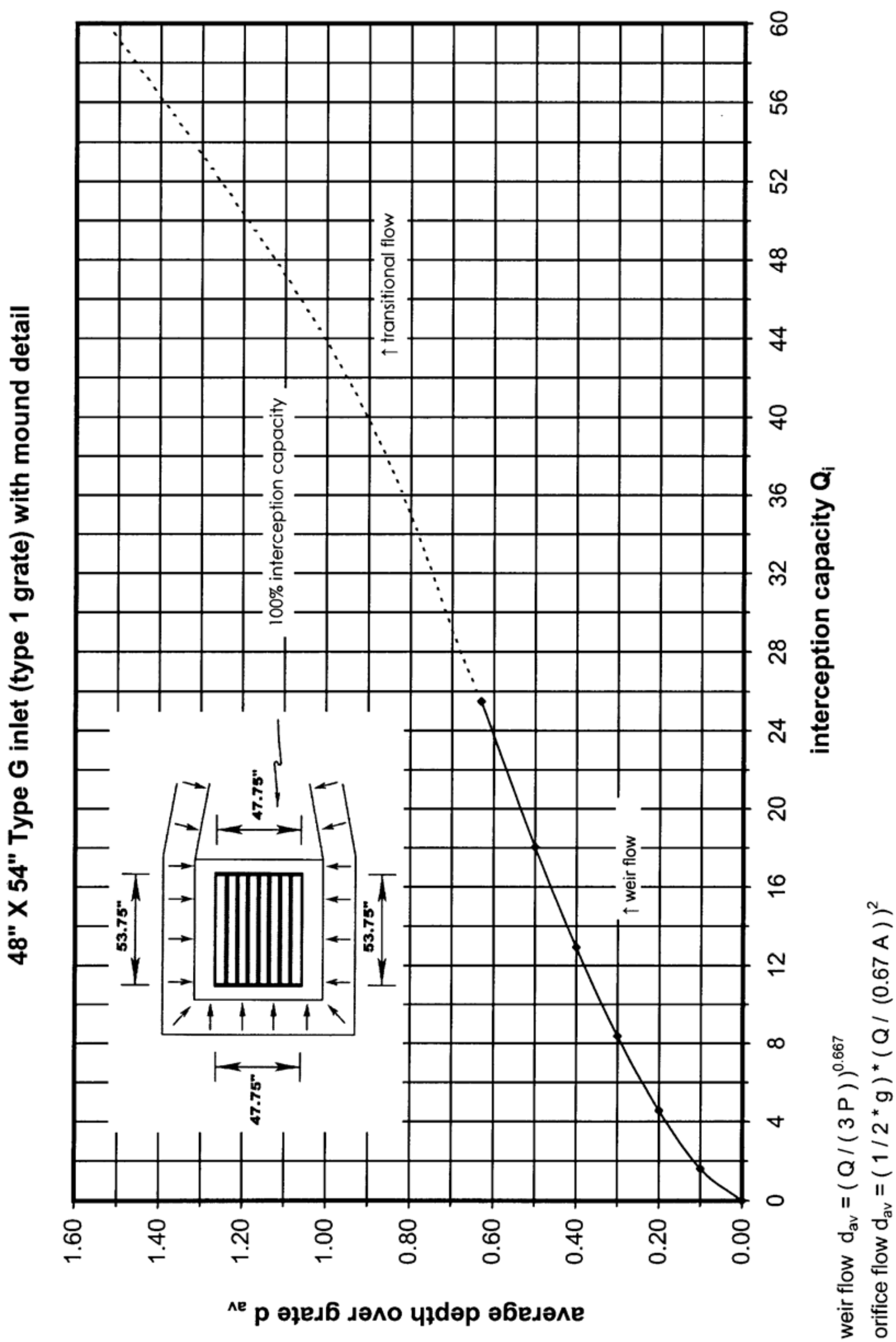


Chart 6-16

54" X 60" Type G grate inlet with mounding detail (Type 1 grate)

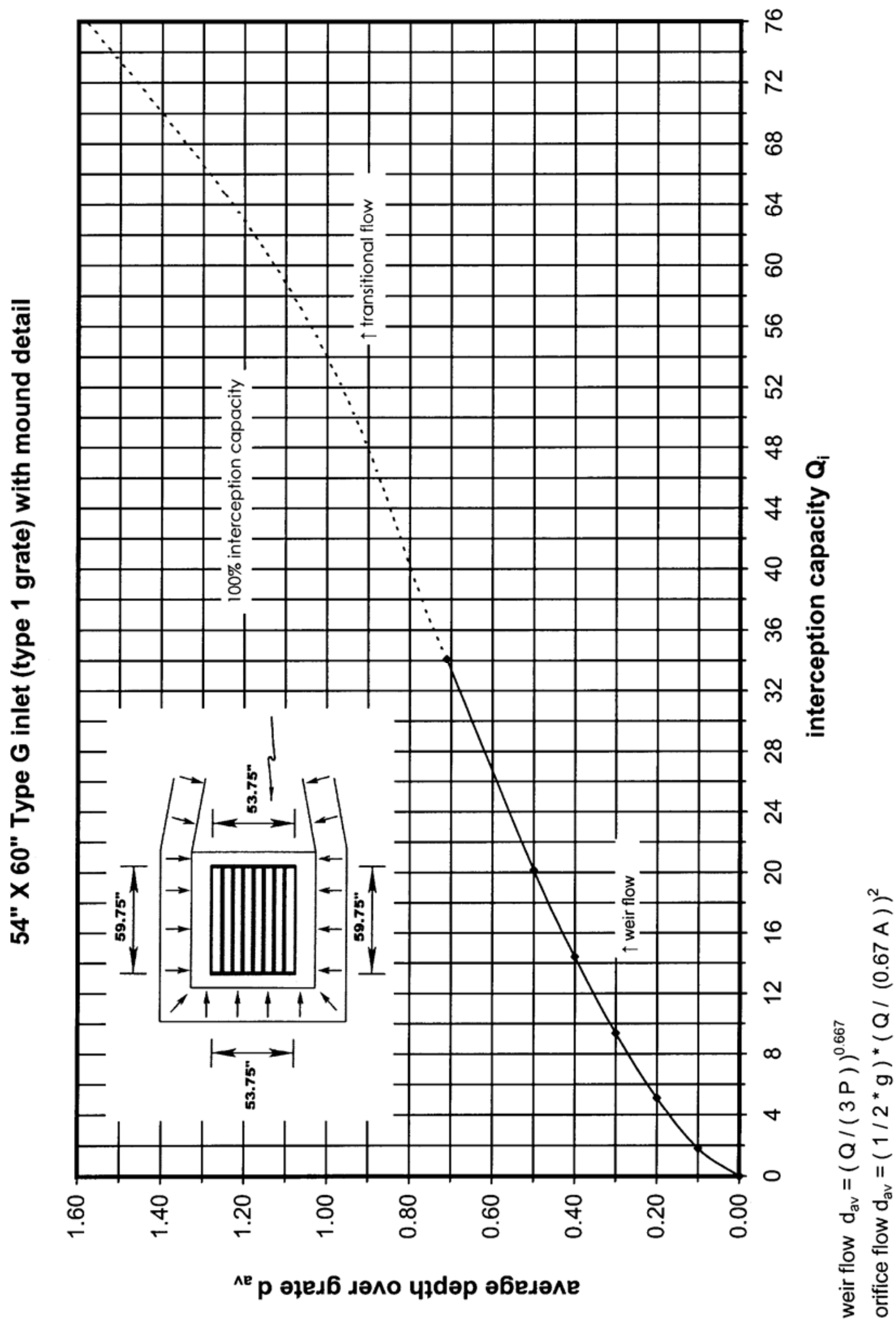


Chart 6-17

60” X 66” Type G grate inlet with mounding detail (Type 1 grate)

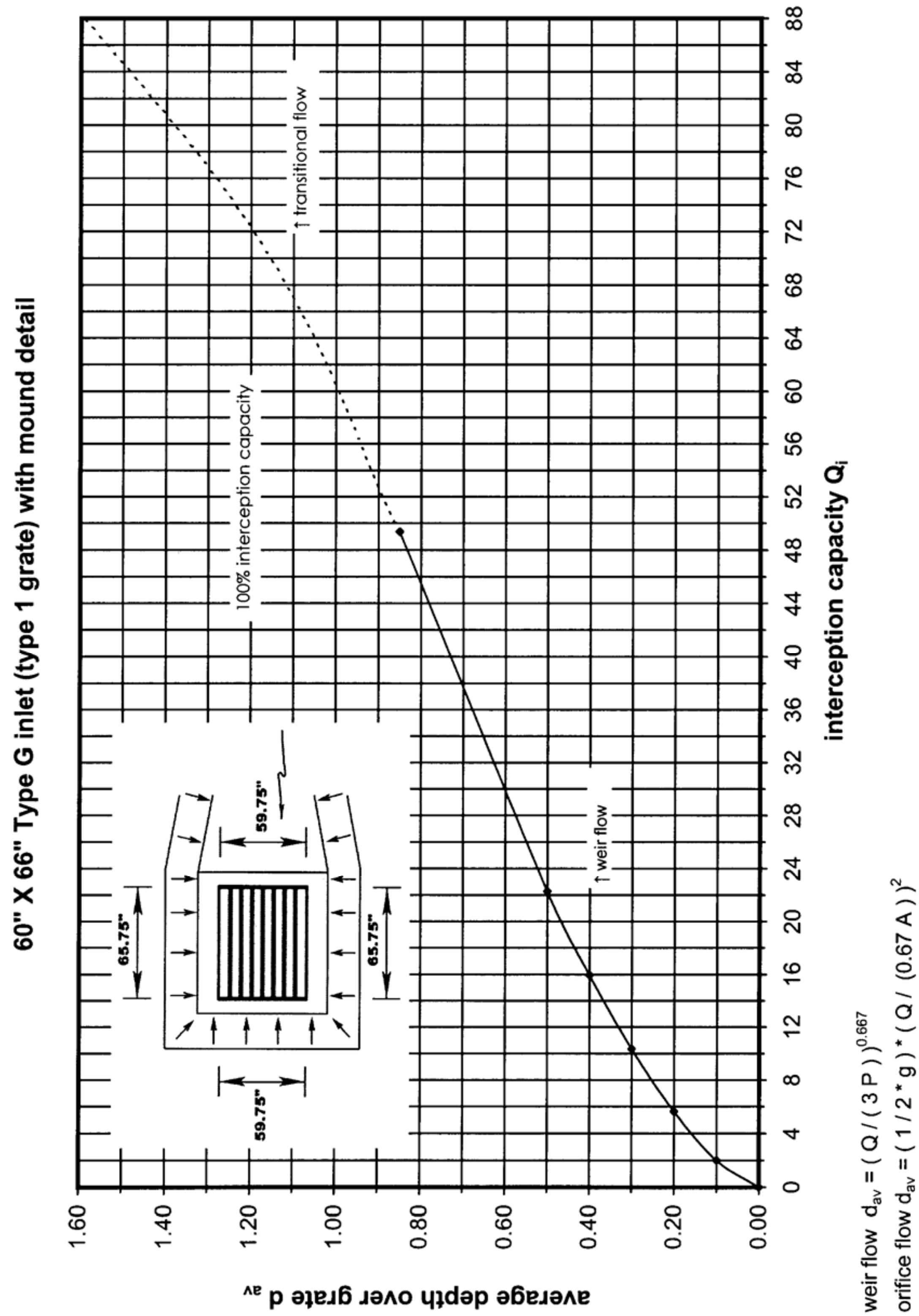


Chart 6-18

32” X 38” Type G grate inlet with mounding detail (Type 2 grate)

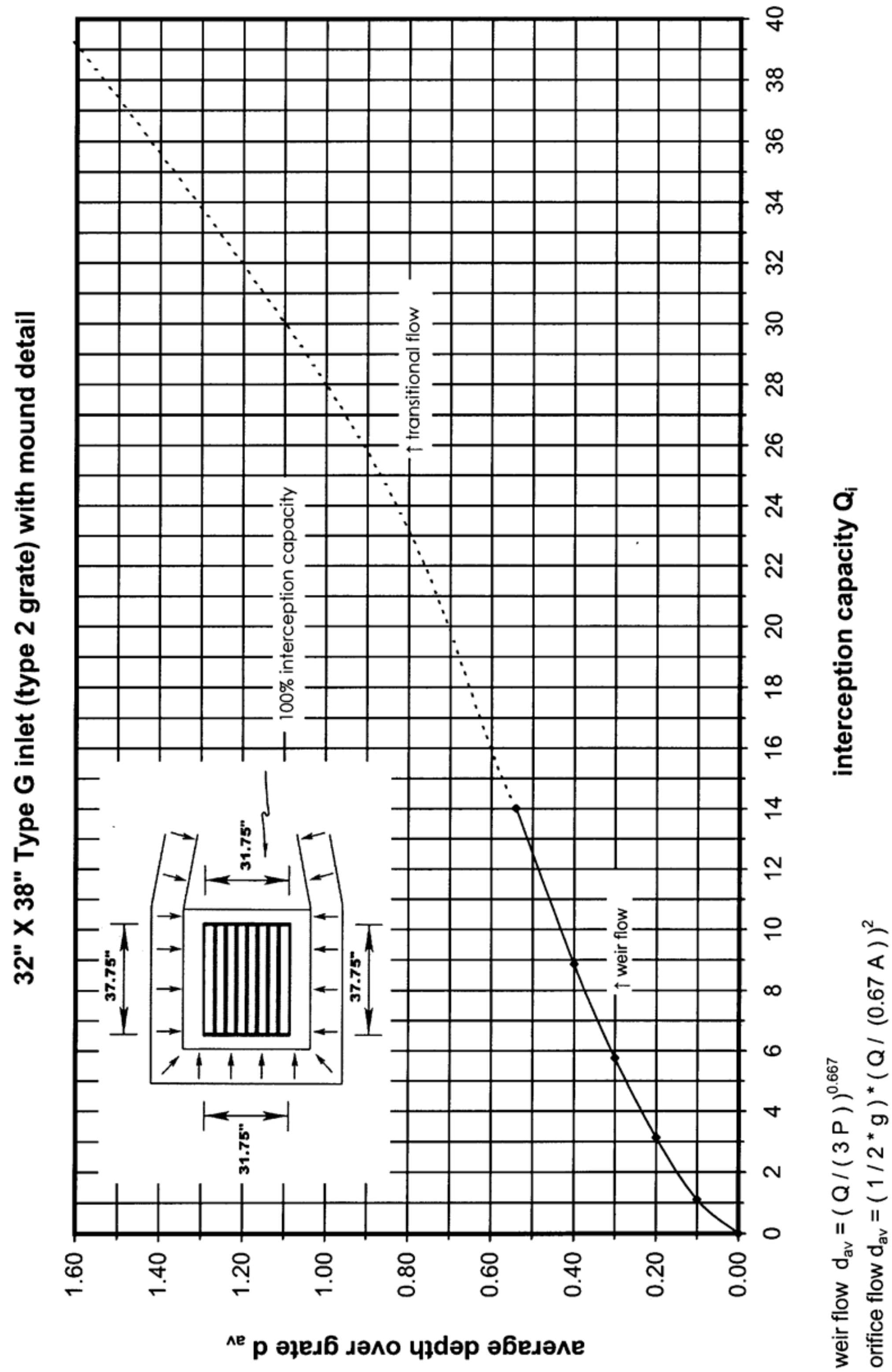


Chart 6-19

36” X 42” Type G grate inlet with mounding detail (Type 2 grate)

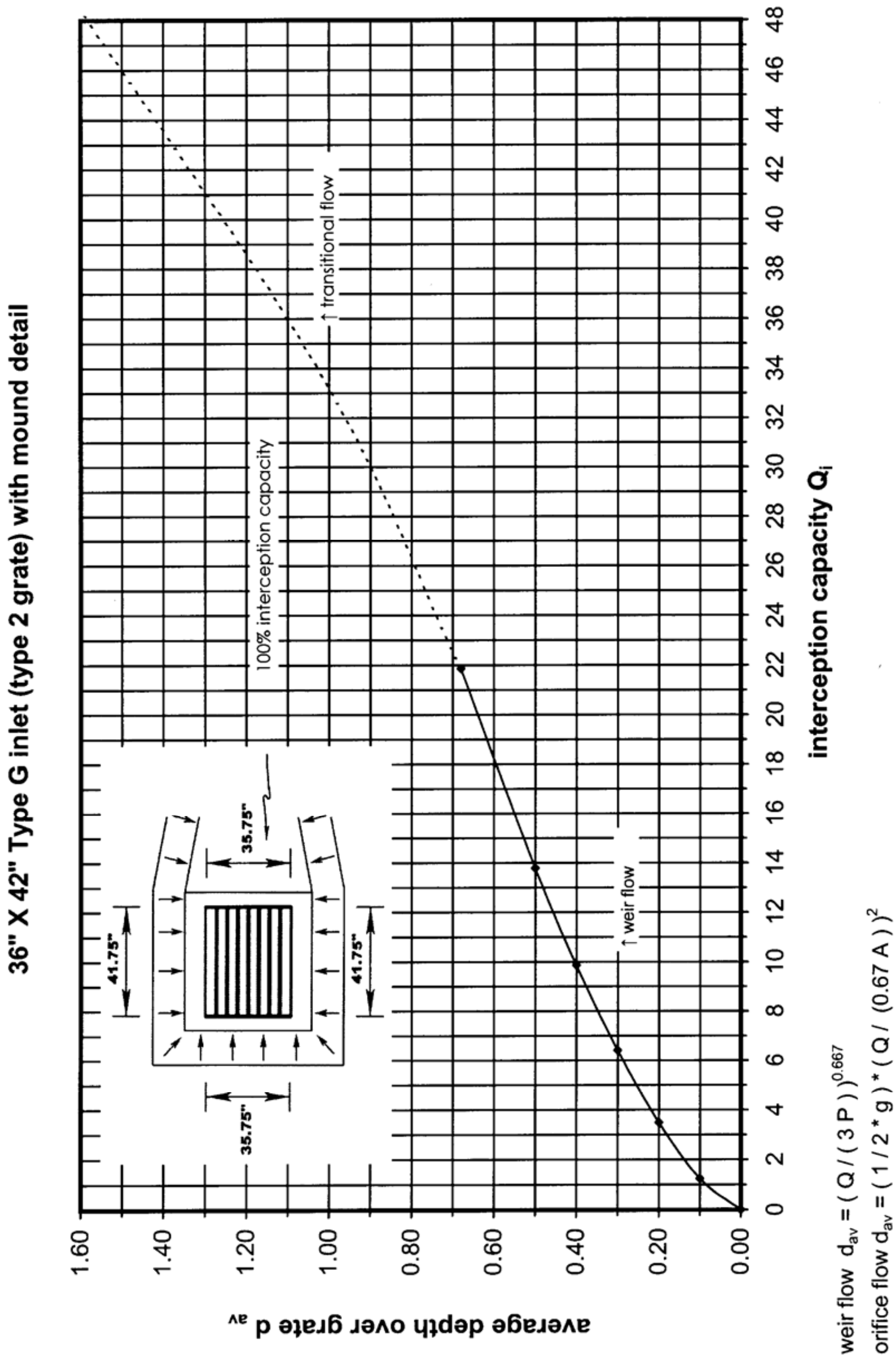


Chart 6-20

42” X 48” Type G grate inlet with mounding detail (Type 2 grate)

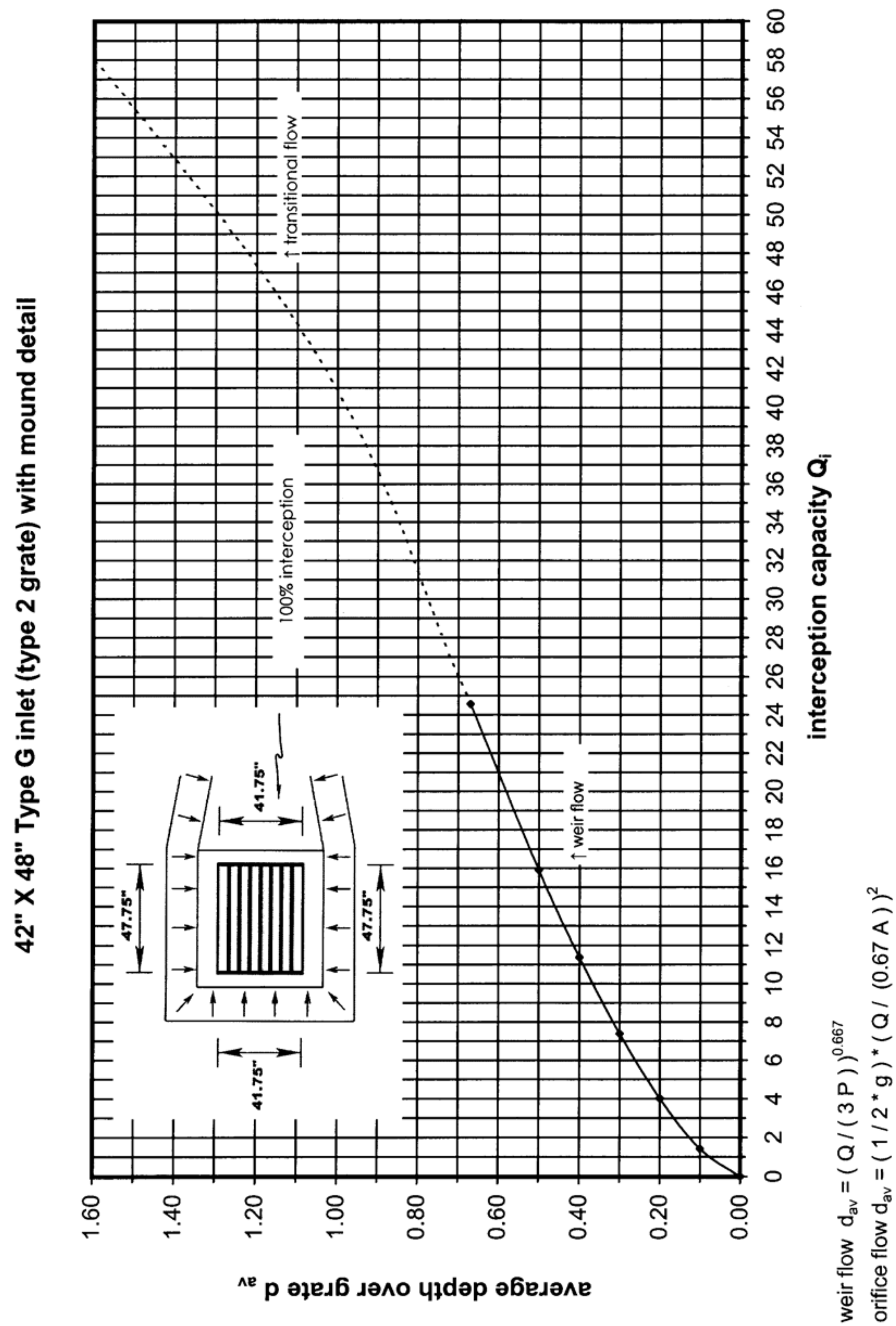


Chart 6-21

45” X 51” Type G grate inlet with mounding detail (Type 2 grate)

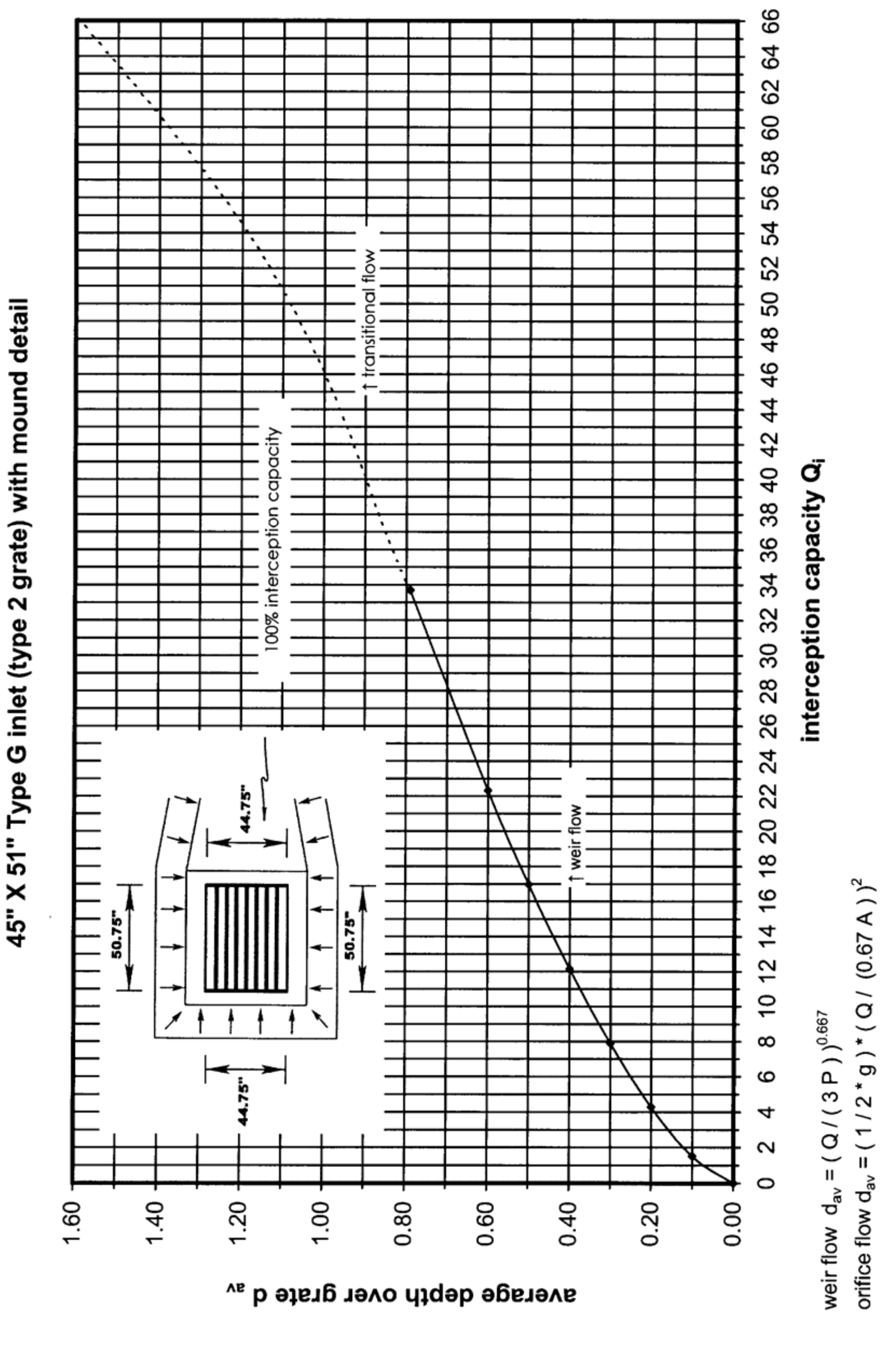


Chart 6-22

48" X 54" Type G grate inlet with mounding detail (Type 2 grate)

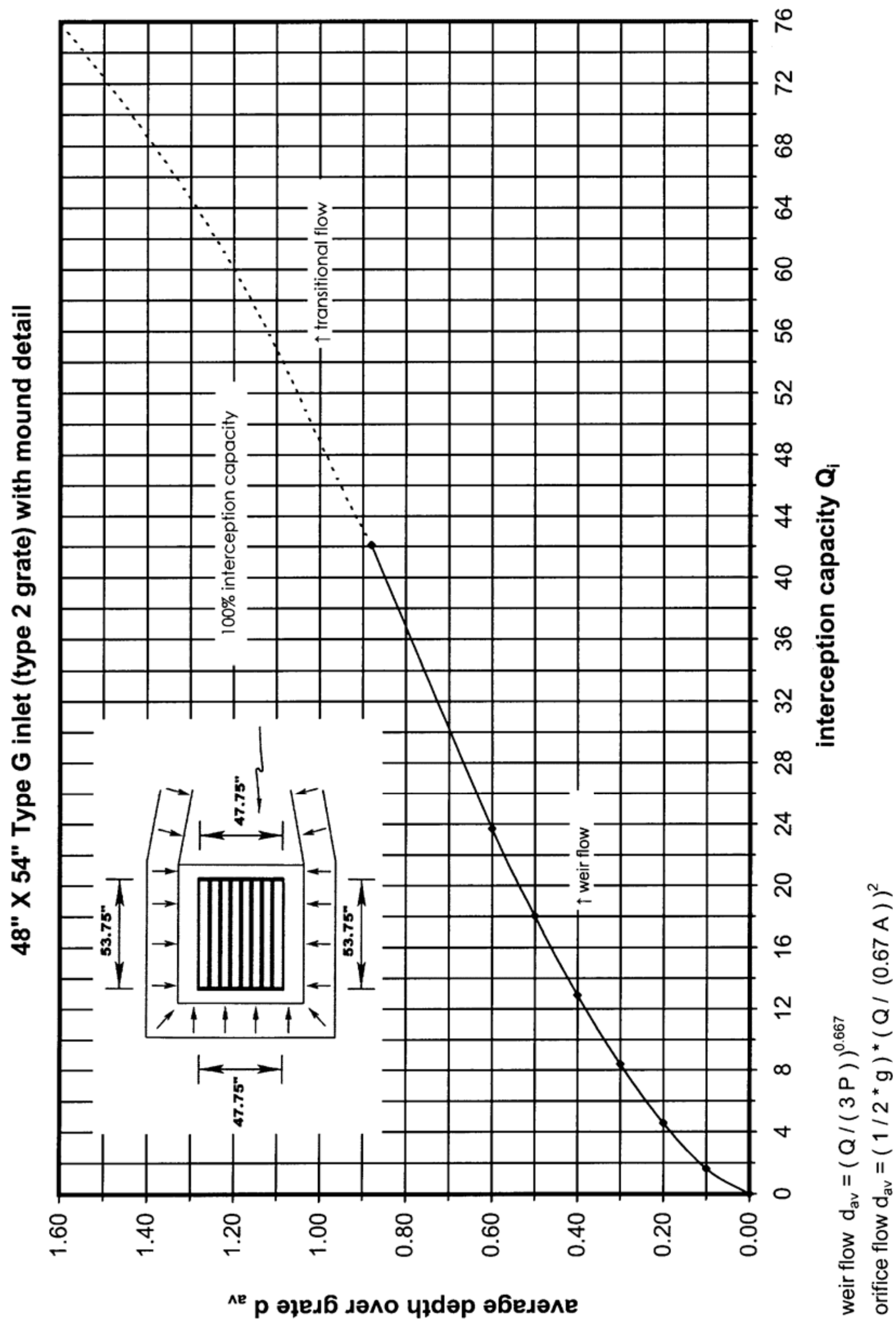


Chart 6-23

54" X 60" Type G grate inlet with mounding detail (Type 2 grate)

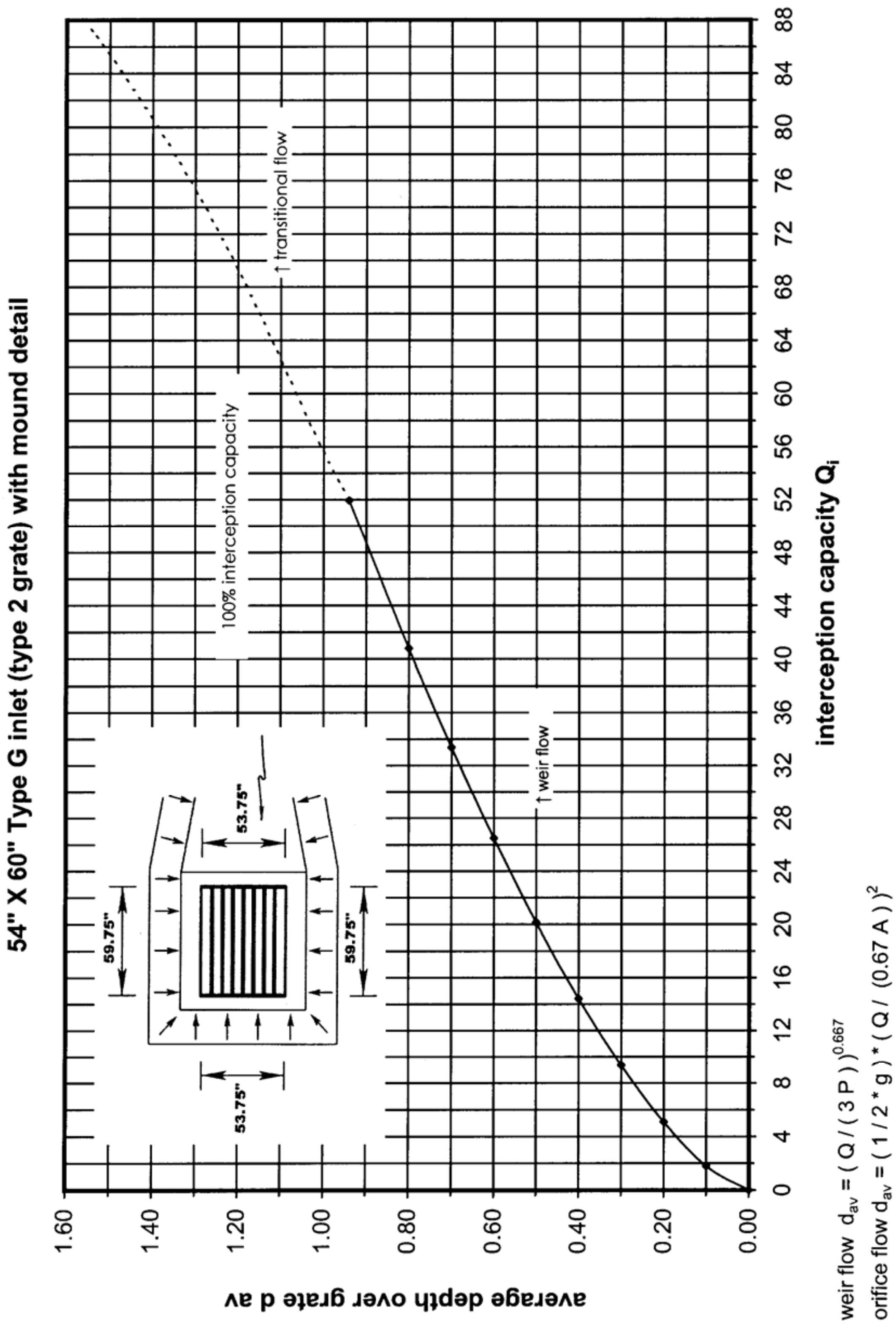
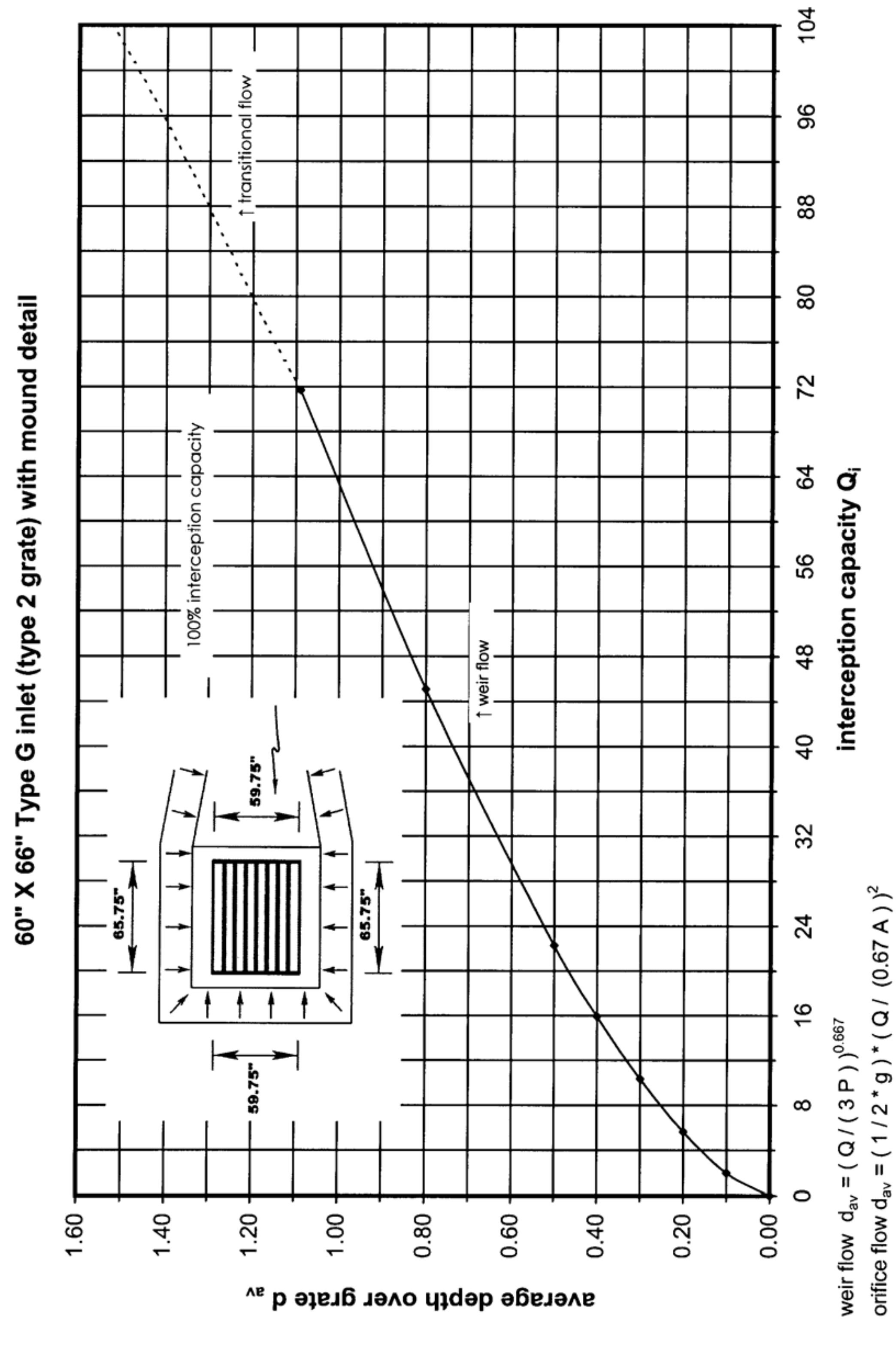


Chart 6-24

60” X 66” Type G grate inlet with mounding detail (Type 2 grate)



6.4.3.1 DITCH INLETS AND STORM SEWER CONSIDERATIONS

Storm sewers connecting to ditch inlets shall be subject to the same rules and guidance as stated in Section 5.3.6. Section 5.3.6.1 states that under ordinary conditions storm drains should flow at a depth of 0.8 times the pipe diameter or equivalent at the design discharge. It may not be economically feasible to provide this much capacity when ditch inlets are involved. Pipe size between the inlets may be limited due to available cover or earthwork requirements for installation (such as pipes in ditches along high cut slopes). Limitations such as these may require special circumstances that allow the pipes connecting the inlets to flow above 0.8D at the design storm.

It is important to be mindful when a number of ditch inlets are connected by conduit (thus creating a storm sewer) and a large offsite drainage area flows to one of the inlets in the drainage system. The hydraulic grade line of the storm sewer should be checked to ensure the influence of the offsite area does not cause a surcharge from one of the ditch inlets associated with the roadway. For example: say an offsite flow enters an inlet from an area oriented perpendicular to the roadway and outlets via a pipe underneath the roadway. The inlet that accepts the offsite flow is connected to the ditch storm sewer paralleling the roadway. A tailwater condition may be created in the ditch storm sewer at the inlet accepting the offsite flow, thus affecting the hydraulic grade line of the ditch storm sewer for the design flow.

The outlet pipe from a ditch storm sewer may require special considerations if this pipe crosses the roadway. This is because the pipe passing under a roadway may be considered a culvert rather than the outlet to a storm drain. Since the design storm is larger for culverts, the outlet pipe for the storm drain should be checked using the culvert design storm criteria. This is especially important if the roadway is classified as a multi-lane or interstate highway, and/or there is an offsite drainage area contributing to the system. The minimum pipe diameter may also be larger than what a storm drain design requires.

6.5 DESIGN PROCEDURE

Each project is unique, but the following six basic design steps are normally applicable:

Step 1: Establish the roadside plan

- Collect available site data including design discharge, channel slope, and underlying soil and rock materials.

- Obtain or prepare existing and proposed plan-profile layout.
- Determine the roadside and median ditch outlets.
- Draw the drainage divides for each outlet location.
- Layout the proposed ditches.

Step 2: Select initial ditch cross-section

- Select initial ditch depth, side slopes, and bottom width.
- Identify constraints that may restrict cross-section design such as clear zone requirements, right-of-way limits, utilities, existing drainage facilities, and environmentally sensitive areas.

Step 3: Select initial longitudinal gradient

- Plot initial grade on plan-profile layout.
- Consider influence of type of lining on grade.

Step 4: Check flow capacity and adjust ditch size as necessary

- Compute 10-year design discharge at the downstream end of the ditch segment.
- Select roughness coefficient and the maximum allowable depth criteria.
- Compute flow depth using Manning's equation and check ditch capacity.
- If ditch capacity is inadequate, possible adjustments include: increasing bottom width, using flatter side slopes, increasing longitudinal gradient, providing smoother lining, installing drop inlets and parallel storm drain pipe to supplement ditch capacity.
- Provide smooth transitions at changes in ditch cross-sections.

Step 5: Determine channel protection lining needed

- Select a lining and determine the permissible shear stress.
- Calculate the maximum shear stress in the ditch and check if the lining is adequate. If the maximum shear stress in the ditch is more than the permissible shear stress of the lining, consider the following options: use more resistant lining, such as concrete or rock, decrease longitudinal slope, use drop structures, increase channel width, or decrease channel side slopes.

Step 6: Analyze outlet points and downstream effects

- Identify adverse impacts such as increased erosion or flooding to downstream properties that may result from discharge at the ditch outlet. Possible adverse impacts could include increase or decrease in discharge, increase in velocity, concentration of sheet flow, change in outlet water quality, and diversion of flow from the watershed.
- Mitigate adverse impacts as required. Mitigation activities could include the following possibilities: enlarging the receiving channel to accommodate increased flows, installing storm water detention structures, installing energy dissipaters to control high velocities, providing erosion protection for the downstream receiving channel, and installing sedimentation or infiltration basins.

The use of Form 6-1 is recommended for recording the design calculations. Form 6-1 should be used for mild gradient ditches (<10%) with side slopes flatter than 3H:1V. The design procedure for linings made of grass and supplemented with matting is described in Section 6.4.2.3 and Section 6.4.2.4. The design procedure described in Section 6.4.2.5 should be used to size rock protection when the side slopes are steeper than 3H:1V.

Form 6-1 does not cover ditch protection calculations for bends and steep longitudinal gradients. Section 6.4.2.2 describes the design procedure for protecting ditches in bends. Section 6.4.2.6 describes the design procedure for sizing rock protection in ditches with steep longitudinal gradients (>10%).

Form 6-1 Ditch Hydraulic Design

LOCATION		DITCH FLOW (AT THE END STATION)									DITCH PROPERTIES							DITCH FLOW PROPERTIES					MIN. FLOW VEL. (0.5 ft/s)	
				CA		TIME OF CONCENTRATION																		
BEGIN STATION	END STATION	DRAINAGE AREA	RUNNOFF COEFICIENT	INCREMENT	ACCUMULATED	INCREMENT	ACCUMULATED	RECURRENCE INTERVAL*	RAINFALL INTENSITY	RUNOFF = C I A	LONGITUDINAL DITCH SLOPE	BOTTOM WIDTH	FORESLOPE	BACKSLOPE	SOIL TYPE	DITCH LINING	ROUGHNESS COEFFICIENT MANNING'S n	DEPTH	MAXIMUM ALLOWABLE DEPTH	VELOCITY	SHEAR STRESS	MAXIMUM PERMISSIBLE SHEAR STRESS	1/3 DESIGN FLOW DEPTH	DITCH FLOW VELOCITY
		A	C	ΔCA	ΣCA	Δt_c	Σt_c	2-yr. or	i	Q	S						n	d		V	τ		1/3 d	$V_{1/3}$
		acres		acres	acres	min	min	10-yr.	in / hr	cfs	ft / ft	feet						feet	feet	ft / s	psf	psf	feet	ft / s
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25

*Ditches shall be designed for a 10-year (10%) storm. If permanent matting is used, then the 2-year (50%) storm shall also be analyzed for the non-vegetated state.

6.6 REFERENCES

(1) Design of Roadside Channels with Flexible Linings, Hydraulic Engineering Circular-15 (HEC-15), Federal Highway Administration, April 1988

(2) Design of Riprap Revetment, Hydraulic Engineering Circular-11 (HEC-11), Federal Highway Administration, March 1989

(3) Drainage Manual, West Virginia Department of Highways, Roadway Design Division, 1984

(4) Drainage Manual, West Virginia State Road Commission, Design Division, June 1963

(5) Erosion Control Technology Council, Internet Website: http://www.ectc.org/index.html

(6) Hydraulic Design of Flood Control Channels, EM 1110-2-1601, Engineer Manual, U.S. Army Corps of Engineers, July 1, 1991

(7) Model Drainage Manual, American Association of State Highway Transportation Officials (AASHTO), 1999 (Metric Edition), and 1991 (English Edition)

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 7:
CHANNELS

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF FORM

LIST OF CHARTS

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CHAPTER 7: CHANNELS

7.1 INTRODUCTION

Channels, or open-channels, are natural or constructed conveyances for water in which the water surface is exposed to the atmosphere and flow is driven by gravity.

A well-designed stable channel carries runoff without erosion or excessive sedimentation. It does not present a hazard to the highway facility, traffic or adjacent properties, and it provides the lowest overall construction and maintenance cost.

Factors involved in channel design include:

- Channel size
- Alignment
- Grade
- Protective linings
- Grade control structures
- Impact on the surrounding environment
- Aquatic habitat

7.1.1 PURPOSE

This chapter focuses on the hydraulic analysis and design procedures for major channels. Although the hydraulic design concepts in this chapter also apply to minor channels (i.e., roadside and median ditches), ditch design is treated separately in Chapter 6.

The purpose of this chapter is to:

- establish design policy,
- specify design criteria,
- review design concepts,
- outline design procedures and
- present example problems for major channel design.

7.1.2 CHANNEL CLASSIFICATION

On the basis of their origin, channels may be broadly classified as natural or constructed.

Natural channels are major alluvial channels formed through long-term geomorphologic activity including erosion and sedimentation. Natural channels have irregular cross-sections and are generally meandering in plan form. Several natural channel classification systems have been proposed over the years based on the relationship between form (morphology) and process (mechanics). Well known classification systems include those proposed by Brice and Blodgett (1978), Schumm (1981), Mollard and Janes (1984) and Rosgen (1994).

Constructed channels include: roadside and median ditches, drainage ditches, irrigation ditches or canals and lined channels. Constructed channels are usually characterized by regular geometric cross-sections and may be unlined (earth), or lined with natural (e.g., vegetation) or artificial (e.g., concrete, riprap) materials to protect against erosion.

Channels encountered in highway design are classified as minor or major channels based on their size and purpose.

Minor channels are small channels whose primary purpose is to collect sheet flow from the roadway right-of-way and discharge that flow into a major channel without causing any damage to the highway facility or adjacent property. Such channels collect flow from the pavement, median, cut slopes and fill slopes and are usually located parallel to the roadway and fall within the right-of-way. Minor channels are generally not considered jurisdictional streams by the U.S. Army Corps of Engineers. Contact the Corps for a jurisdictional determination if you are unsure. Minor channels as described here can usually be designed as ditches in accordance with Chapter 6 of this manual.

Major channels are natural channels, constructed replacements, or relocated sections of natural channels which collect and convey drainage from minor channels, storm drain pipes and off-site areas. The size and shape of major channels are determined either by natural forces or site-specific design requirements. Major channels may be located within or outside the highway right-of-way and they usually have more bends and variations in cross-section which can result in more turbulence and scour. The consequences of failure are usually more severe for major channels; therefore, they usually require more rigorous design procedures and water surface profile computations.

Major channels are usually jurisdictional streams and will require a permit from the U.S. Army Corps of Engineers. Contact the Corps for a jurisdictional determination if you are unsure.

This chapter pertains to major channel design.

7.1.3 MAJOR CHANNEL DESIGN CATEGORIES

There are three general categories of major channel design based on the site-specific situations encountered and the overall design objectives of the highway project:

Channel Stabilization: An unstable channel that has the potential to damage an existing highway requires stabilization in-place. Such projects could involve either a channel that crosses the highway alignment through a bridge or culvert, or one that flows alongside it. The extent or length of the stabilization will vary based on site-specific conditions and can extend beyond the existing highway right-of-way as deemed necessary by WVDOH. Channel stabilization may be accomplished with a variety of methods. Traditional methods involve lining the channel banks with “hard” armor (concrete, riprap revetments, articulated concrete block revetments, etc.). Other alternatives include “soft” armor (vegetative linings), in-stream grade control and bank stabilization structures.

Flood Control: In the interest of public safety, highway projects may be designed to reduce flooding of adjacent property. For example, channel conveyance may be increased by enlarging the existing channel or by creating an overflow channel or flood prone area to accommodate excess runoff or increases in water surface elevation caused by a highway project. Channel stability must be carefully considered.

Channel Relocations: Channel relocations typically involve severe impacts to the stream environment. The existing channel (stable or unstable) running alongside or crossing the highway, may need to be relocated to accommodate the highway alignment. Channel relocations typically require environmental commitments for stream mitigation and are usually accomplished using Natural Channel Design methods.

The selection of a major channel design method depends on local conditions, design objectives, risk of damage and cost. In all cases, channel design requires evaluating proposed changes to the hydraulic variables due to the highway project by comparing with the existing condition. Highway embankment fills and/or crossings can cause changes in water surface profiles, channel velocity, flow direction and cross-section flow distribution.

7.1.4 NATURAL CHANNEL DESIGN

This chapter does not cover the principles and procedures of Natural Channel Design. Guidance and procedures for Natural Channel Design are contained in the WVDOH Natural Stream Channel Design (NSCD) Manual being prepared by the WVDOH. Until the WVDOH NSCD manual is published, designers who lack adequate training and experience should consult the Hydraulic & Drainage Unit for assistance.

Natural Channel Design is the process by which the geomorphology (dimension, pattern, and profile) of an unstable stream channel is restored to the characteristics of a stable reference stream of the same type. A stable natural channel is defined as one that moves sediment and water from its watershed while maintaining a stable dimension, pattern and profile without aggrading or degrading.

While detailed design criteria and procedures for Natural Channel Design are not covered here, it is important that the designer recognize that the hydraulics and stable channel design principles discussed in this chapter form the basis for Natural Channel Design. The designer should be cognizant of situations that can trigger a Natural Channel Design requirement. For instance, channel relocations require individual US Army Corps of Engineers (USACE) 404 Permit and stream mitigation in which Natural Channel Design methods are used to fulfill the environmental mitigation commitments of a highway project.

The design principles discussed in this chapter are applicable to all highway projects that involve channel stabilization or flood control elements as well as channel projects requiring a water surface profile analysis. Some stabilization projects might also incorporate Natural Channel Design methods if deemed necessary by the WVDOH.

7.1.5 CHANNEL DESIGN PROCEDURE

Assistance with channel design is available from the Engineering Division, Hydraulic and Drainage Unit. The hydraulic analysis and channel protection design procedure should consist of the following general steps:

Step 1: Determine project scope and data requirements

Step 2: Coordinate with USACE and WVDNR

Step 3: Conduct field reconnaissance

Step 4: Assemble site data and project file

Step 5: Determine the storm frequency and design discharge based on highway classification as outlined in Chapter 4.

Step 6: Perform hydraulic analysis and determine hydraulic parameters

Step 7: Conduct stability evaluation

Step 8: Select type of protection needed using stable channel design concepts

Step 9: Design selected channel protection

Step 10: Prepare Design Report

Step 11: Show design details on plans

7.2 DESIGN POLICY

Hydraulic design of major channels shall involve the selection and evaluation of alternatives in accordance with WVDOH design criteria. This is to ensure that a highway facility meets its intended purpose without endangering the structural integrity of the facility and without causing adverse effects to the environment or the public welfare.

The following general policy statements shall apply to hydraulic analyses for design of major channels:

- Major channels shall be designed by selecting and evaluating a range of design frequency discharges based on the roadway classification, risk of flood hazard, economics and local site conditions.
- Major channels located in or outside the highway right-of-way shall be designed to be stable (i.e., transport water and sediment without causing flooding, excessive deposition or erosion).
- Public safety shall be a high priority in the design of major channels.
- Coordination with Federal, State and Local agencies concerned with water resources shall be a priority in channel design.
- The design of major channels shall consider the storm frequency, the type of maintenance required and make allowance for maintenance access.
- Placement of highway fill encroachments in designated regulatory floodways shall be avoided and encroachments in floodplains shall be minimized to the fullest extent practicable.
- Channel banks shall be protected from erosion with appropriate linings as necessary.

- Energy dissipation and/or grade control structures shall be considered to reduce scouring velocities as deemed necessary by WVDOH.
- The extent and degree of channel relocation and modification shall be minimized and shall be consistent with good design practice.
- Environmental impacts of channel modifications and/or relocations including disturbance to fish, macro-invertebrate and wetland habitat shall be addressed.
- Natural Channel Design and stream habitat enhancement features required by the environmental stream mitigation commitments of the highway project shall be incorporated as deemed necessary by the WVDOH.

7.3 DESIGN CRITERIA

The following criteria shall apply to hydraulic analyses for the design of major channels:

7.3.1 DESIGN FREQUENCY

Major channels shall be designed for the minimum storm frequency criteria based on highway classification, as outlined in Chapter 4.

7.3.2 FLOODPLAIN ENCROACHMENTS

The design water elevations shall be at least 1 foot lower than the roadway sub-grade for highway fill encroaching on floodplains. On major highway facilities, the hydraulic effects of highway fill encroachments in floodplains shall be evaluated over a full range of frequency-based peak discharges from the 2-year through the 500-year recurrence intervals, as deemed necessary by WVDOH.

7.3.3 BACKWATER EFFECTS

Channel projects shall be designed to avoid increases in backwater elevations for the design frequency discharge, where safety of public life and/or property is a priority.

7.3.4 NFIP COORDINATION

Where floodplains and/or floodways regulated by the National Flood Insurance Program (NFIP) are affected, increases in water surface elevations for the base flood (100-year discharge) shall be evaluated and coordinated with the affected local communities.

7.3.5 ROUGHNESS VALUE SELECTION

Manning's Roughness values selected to calculate design water surface elevations for channels shall be based on the future growth of vegetation during the summer months. This is when the vegetation provides the maximum resistance to flow and yields a higher flow depth.

Manning's Roughness values selected to design channel protection should be based on the vegetative condition that represents the minimum resistance to flow (such as in the winter months) and yields higher velocities and shear stresses (provided the energy gradient is steeper). The shear stress used for channel protection should be evaluated using a full seasonal range of Manning's Roughness values expected.

Manning's roughness coefficients shall be selected based on the minimum storm frequency criteria in Section 7.3.1.

7.3.6 CHANNEL STABILITY EVALUATION

The horizontal (plan form) and vertical (profile) stability of unstable channels that could affect a highway facility shall be investigated as deemed necessary by WVDOH. An unstable channel is one with a rate or magnitude of change that is large enough to be a significant factor in the design and maintenance of the highway facility.

The horizontal stability evaluation shall consider the rate of meander migration and the possibility of channel avulsion. The vertical stability evaluation shall consider methods such as incipient motion, equilibrium slope and sediment continuity. Refer to Section 7.4.2.2 for a brief explanation. Vertical stability evaluation shall be based on the methods described in Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures, Third Edition, March 2001 or latest revision, published by the Federal Highway Administration (FHWA).

7.3.7 STABLE CHANNEL DESIGN

Channels shall be protected with appropriate structural or vegetative linings using Stable Channel Design principles and permissible shear stress criteria. The calculated maximum shear stress in the channel shall be less than the permissible shear stress of the channel lining material. The design discharge shall be based on the minimum storm frequency criteria in Section 7.3.1.

For relocations and major channel alterations, the channel dimensions (width and depth) and profile (longitudinal slope) shall be checked for sediment continuity such that there is a balance between the available sediment supply and sediment transport capacity of the channel. Sediment transport evaluations shall be based on the methods described in Hydraulic Design Series No. 6, River Engineering for Highway Encroachments, Highways in the River Environment, December 2001 or the latest revision, published by the FHWA.

7.3.8 CHANNEL PROTECTION

Protective channel linings such as soft linings (vegetation), hard linings (rock riprap revetment, dumped rock gutter, select stone embankment, articulated concrete block revetment, grouted riprap, gabions) or other equivalent protection, shall be placed in areas where erosion is likely to occur, particularly on channel bends and near new structures (culverts and bridges).

The selection of soft linings such as vegetation with temporary or permanent matting shall be based on permissible shear stress criteria.

The design drawings shall stress that the matting must be installed in strict accordance with the manufacturer’s specifications.

Vegetated linings with matting shall not be used to protect the outlet channel for culverts. Hard linings should be used for culvert outlet protection.

Hard linings such as rock riprap, dumped rock gutter or select embankment stone shall extend below the estimated depth of total toe scour. Total toe scour depth shall consist of bend scour and long-term degradation. A range of flows up to the design discharge shall be used to check the stability of riprap revetments. Rock or stone should not be placed within the highway clear zone. If this can not be avoided, then it must be shielded with guardrail or other traffic barrier.

Where channel protection with linings cannot be considered adequate, appropriate energy dissipation and/or in-stream grade control structures shall be provided as deemed necessary by WVDOH. Refer to Section 7.3.11 for channel protection design methods.

7.3.9 CHANNEL RELOCATIONS

The design of relocated channels shall incorporate Stable Channel Design and/or Natural Stream Channel Design methods as deemed necessary by WVDOH.

Relocated channel sections shall closely approximate the dimensions, pattern, profile and roughness characteristics of the existing channel, provided it is in a stable condition or a stable reference stream of the same type.

7.3.10 ENVIRONMENTAL CONSIDERATIONS

Channel work should be kept to a minimum, and avoided whenever feasible in order to minimize unnecessary damage to aquatic habitat. The length of new channel should be equal to the length of channel being replaced to avoid increasing the stream slope. Detailed survey and data collection will be required prior to design and construction of stream projects. For projects where the new stream will be a different stream type than the existing channel, reference reach data collection will be required at another site. Assistance with reference reach data collection is available from the Engineering Division, Hydraulic and Drainage Unit.

7.3.11 DESIGN METHODS

The HEC-RAS computer program shall be used to perform water surface profile calculations and/or determine values of hydraulic variables such as water depth, velocity and shear stress. The use of a different computer program for water surface profile analysis shall be coordinated with the WVDOH.

Lateral and vertical channel stability analyses shall be based on the procedures outlined in FHWA's HEC-20. Stable channel design shall be based on the principles outlined in FHWA's HEC-15 publication. Refer to FHWA's HDS-6 publication for sediment continuity/transport analyses.

Vegetated linings shall be designed using FHWA's HEC-15. Rock linings shall be designed using FHWA's HEC-11. For rock linings in channels without bends, the total toe scour depth estimate shall be the sum of the scour depth for straight channels as determined by using HEC-11 plus long-term degradation. For straight channels and channels with minor bends, the equations presented in AASHTO's Model Drainage Manual (Equations 17.2 and 17.3) may be used to estimate toe scour depths. Long-term channel degradation may be estimated qualitatively using historic records and/or with the U.S. Bureau of Reclamation Method (1984), described in FHWA's HEC-20.

Hard linings other than rock, such as gabions and articulated concrete block systems, shall be designed according to the manufacturer's literature and specifications. If required, energy dissipation structures shall be designed in accordance with FHWA's HEC-14. The designer shall refer to a separate design manual for Natural Channel Design methods and procedures, currently under development by the WVDOH.

7.4 DESIGN CONCEPTS

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness and slope. The hydraulic analysis of the existing channel is the basis for channel design and determines the flood hazard at the project site. Values of hydraulic variables are also necessary for the design of channel protection. In addition, hydraulic analyses facilitate channel stability evaluations so that the severity of channel instability can be considered and channel features can be designed to promote channel stability.

The following sections discuss the concepts of hydraulics, channel stability evaluations, stable channel design and channel protection methods. For a detailed description of design and analysis procedures the designer should review open channel flow fundamentals found in standard hydraulics texts. The designer may also refer to the recommended publications in Section 7.4.7 of this manual.

7.4.1 TYPES OF OPEN CHANNEL FLOW

Open channel flow can be classified as steady or unsteady by using time as the criterion. Flow is considered steady if the depth and velocity at a given location do not change with time. Flow is considered unsteady, if the depth and velocity vary with time.

Open channel flow can be classified as uniform or non-uniform by using space as the criterion. Flow is considered uniform if the depth and velocity do not change with distance along the channel. Flow is considered non-uniform or varied, if the depth and velocity vary with distance along the channel. Flow is considered gradually varied if the depth changes gradually over a long distance. Flow is considered rapidly varied if the depth changes abruptly over a short distance (e.g., hydraulic jump).

Flow in prismatic channels (e.g., concrete channels and roadside ditches) can be assumed to be steady uniform flow, but flow in most natural alluvial channels is unsteady gradually varied flow. Most channel design problems can be solved assuming steady gradually varied flow.

7.4.2 HYDRAULIC THEORY

Open channel flow is analyzed using the basic laws of physics and the fundamentals of fluid mechanics. The laws of conservation of mass, energy and momentum are used to calculate the position of the free water surface, which is usually one of the unknown variables and depends on the proper quantification of the flow resistance or channel roughness.

7.4.2.1 CONTINUITY EQUATION

The continuity equation is a statement of the law of conservation of mass according to which mass can neither be created nor destroyed. For the case of one-dimensional steady flow, it is expressed in this form:

$$Q = A_1V_1 = A_2V_2$$

Where:

Q = Discharge, cfs

A = Cross-sectional area of flow, ft^2

V = Mean velocity perpendicular to the cross-section, fps

1,2 = Successive cross-sections along the flow path.

7.4.2.2 MANNING'S EQUATION

Manning's equation can be used to compute the mean velocity V, in an open channel of a given depth with steady, uniform flow:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

Where:

V = Mean velocity perpendicular to the cross-section, fps

n = Manning's roughness coefficient (Refer to Table 1-2 in Chapter 1)

R = Hydraulic Radius = A/P, ft

A = Cross-sectional area of flow, ft^2

P = Wetted Perimeter of the cross-section, ft

S = Slope of the energy grade line in ft/ft

7.4.2.3 NORMAL DEPTH

Normal depth is a unique value of depth for steady uniform flow that can be calculated for any given channel geometry, roughness, slope and discharge. Combining the continuity equation with Manning's equation results in the equation for steady, uniform flow:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

For a given discharge, normal depth is calculated by expressing A in terms of depth (d) and then solving for d by trial and error. The slope of the energy grade line, the hydraulic grade line and the channel are the same at normal depth. Therefore, S can be approximated as the channel slope for channels with irregular cross-sections so long as steady uniform flow conditions can be assumed. The selection of Manning's n should be based on field observations.

7.4.2.4 CONVEYANCE

Conveyance (K) in cubic feet per second represents the carrying capacity of a channel cross-section based upon its geometry and roughness characteristics. It is independent of the channel slope and it is expressed as:

$$K = \frac{1.486}{n} AR^{2/3}$$

The uniform flow equation can then be rewritten as:

$$Q = KS^{1/2}$$

The concept of channel conveyance is useful when computing the flow distribution in the channel cross-section. It is also used to determine the velocity distribution coefficient, α .

7.4.2.5 ENERGY EQUATION

The energy equation is a statement of the law of conservation of energy, according to which energy can neither be created nor destroyed. Energy can change from one form to another, but the total energy within a system remains constant.

The energy equation is applicable to gradually varied flow situations and can be stated as follows (see Figure 7-1): The total energy or head at the upstream section 1 should be equal to the total energy or head at the downstream section 2 plus the loss of energy or head loss (h_L) between the two sections. The energy equation is also known as Bernoulli's equation after the Swiss mathematician Daniel Bernoulli who pioneered the concept of "head" in the 1700's.

For the case of one-dimensional, gradually varied, steady flow in a channel of mild slope between sections 1 and 2, it is expressed as:

$$z_1 + y_1 + \alpha_1 \frac{V_1^{\,2}}{2g} = z_2 + y_2 + \alpha_2 \frac{V_2^{\,2}}{2g} + h_L$$

Where:

z_1 and z_2 = upstream and downstream channel bottom elevation respectively, ft

y_1 and y_2 = upstream and downstream water depths respectively, ft

h_1 and h_2 = upstream and downstream water surface elevations respectively, ft

α_1 and α_2 = Velocity distribution coefficients

V_1 and V_2 = Mean velocities, ft/s

g = Acceleration due to gravity, 32.2 ft/s^2

h_L = Head loss due to boundary resistance and cross-sectional changes, ft

The velocity distribution coefficient (α) accounts for non-uniform distribution of velocities in an irregular channel. It is usually greater than the average velocity head of the cross-section. α is assumed to be equal to 1 for prismatic channels, but is usually greater than 1 for irregular channels with non-uniform flow. The velocity distribution coefficient α is calculated by dividing the cross-section into sub-sections and is defined as follows:

$$\alpha = \sum_{i=1}^{n} \left(\frac{K_i^{3}}{A_i^{2}} \right) \Big/ \left(\frac{K_t^{3}}{A_t^{2}} \right)$$

Where:

K_i = conveyance in the sub-section

K_t = total conveyance in the cross-section

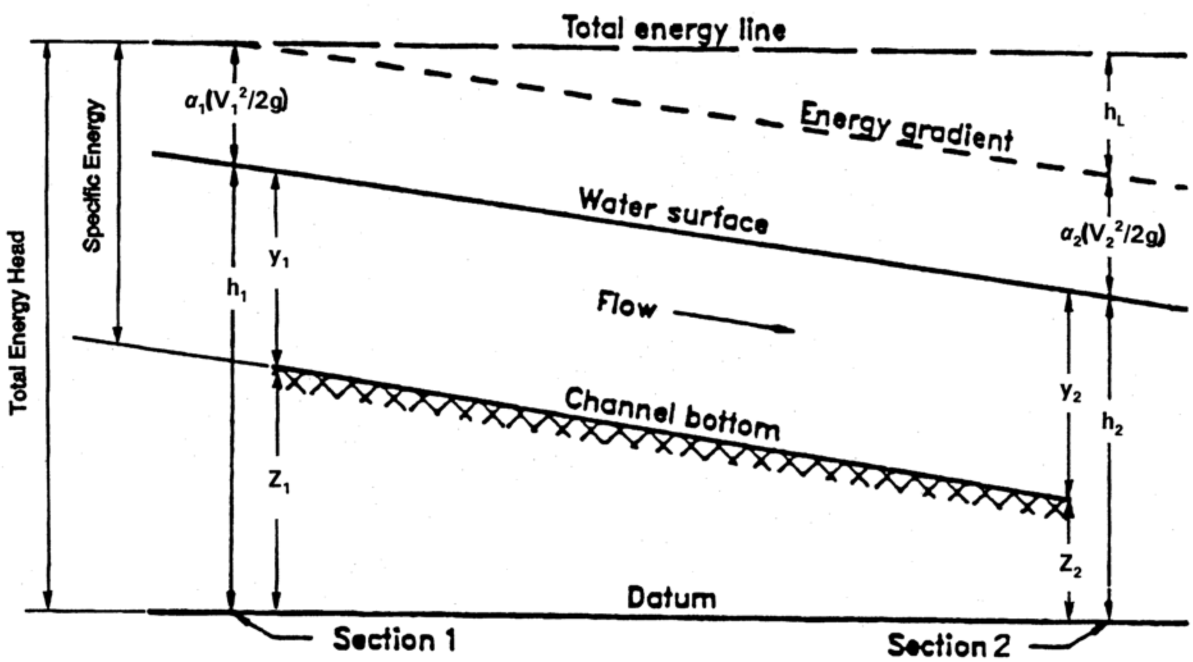
A_i = cross-sectional area of the sub-section

A_t = total cross-sectional area of the cross-section

n = number of sub-sections

Figure 7-1

Terms in the Energy Equation



The total energy head is equal to the sum of the potential energy head (elevation head Z and pressure head Y) and the kinetic energy head (velocity head $V^2/2g$). The head loss is a measure of the *internal* energy dissipated in the whole mass of water due to friction with the channel boundary and minor losses due to changes in shape, cross sectional area and direction of flow between cross-sections.

7.4.2.6 <u>FROUDE NUMBER</u>

The open channel flow regime can exist as sub-critical or supercritical based on the effect of gravity. The effect of gravity on the state of flow is represented by a dimensionless ratio of the inertial forces to gravity forces. This ratio is given by the Froude Number, which is defined as:

$$F = \frac{V}{\sqrt{g d_{md}}}$$

Where:

F = Froude Number

V = Mean velocity of flow in ft/s

g = Acceleration due to gravity, 32.2 ft/s^2

d_{md} = Hydraulic mean depth (feet) = A/T where,

A = Cross-sectional area of the channel in ft^2

T = Top width of the channel in ft

The denominator in the above equation is called the critical velocity and it represents the velocity of a shallow water gravity wave. If F is less than 1 gravity forces are dominant, the flow is tranquil and the flow regime is termed sub-critical. If F is more than 1 then inertial forces are dominant, the flow has high velocity and the flow regime is supercritical. When F is equal to 1 the flow is termed critical. In the case of sub-critical flow, the gravity wave can travel upstream because the critical velocity is greater than the velocity of flow, but this is not the case in supercritical flow.

7.4.2.7 SPECIFIC ENERGY

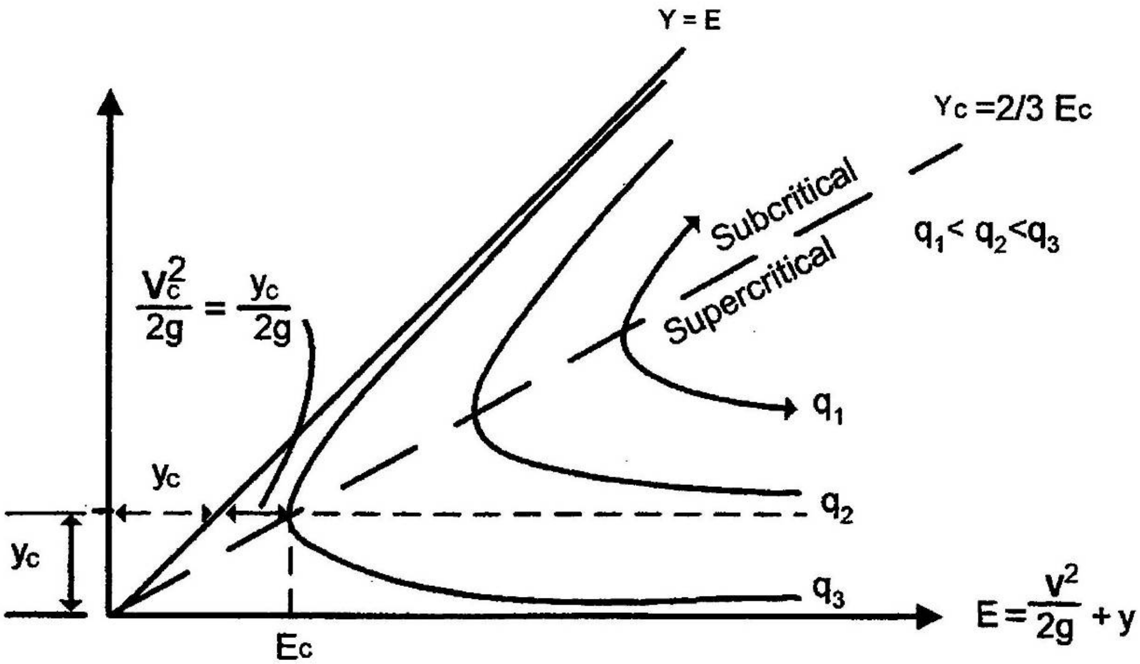
The specific energy (E) is defined as the energy head relative to the channel bottom. For a prismatic channel (i.e., uniform flow) with a mild slope, the specific energy is defined as the sum of the water depth (Y) and the velocity head.

$$E = Y + \alpha \frac{V^2}{2g}$$

The specific energy curves result when the specific energy (E), is plotted against the depth of flow (Y) for a given channel section using varying unit discharges q_1, q_2, and q_3. Each curve shows that for a given specific energy there are two possible depths known as "alternate depths". Critical depth (y_c) occurs at the minimum specific energy (E_c). Depths above critical depth are sub-critical (i.e., larger alternate depth with lower velocities and Froude Number less than 1) and depths below critical depth are supercritical (lower alternate depth with higher velocities and Froude Number greater than 1).

Figure 7-2

Specific Energy Diagram



The specific energy diagram illustrates that normal depth can occur for sub-critical, critical and supercritical flow conditions.

7.4.2.8 CRITICAL DEPTH

Flow at critical depth occurs when the specific energy is a minimum for a given discharge, and the Froude Number is equal to 1. The general expression for critical depth for uniform or gradually varied flow in a channel of small slope regardless of the shape is:

$$\frac{\alpha Q^2}{g} = \frac{A^3}{T}$$

Where:

α = Velocity distribution coefficient

Q = Discharge, ft^3/s

g = Acceleration due to gravity, 32.2 ft/s^2

A = Cross-sectional flow area of the channel in ft^2

T = Top width at the water surface in ft

If normal depth is greater than critical depth the flow is sub-critical and the channel slope is “mild”. If normal depth is less than critical depth the flow is supercritical and the channel slope is “steep”. If normal depth is equal to critical depth the flow is critical and the channel slope is a “critical” slope. There is no normal depth for horizontal (i.e., S=0) or adverse (i.e., S<0) slopes; however, critical depths can exist for horizontal or adverse slopes.

Flow at or near the critical state is unstable because a minor change in specific energy can cause a major change in depth. Such changes in specific energy can be caused by variations in channel roughness, flow area or slope.

7.4.2.9 HYDRAULIC JUMP

A hydraulic jump occurs at an abrupt transition from supercritical flow to sub-critical flow in the flow direction. There are significant changes in depth and velocity in a jump and energy is dissipated. For this reason, a hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures.

7.4.2.10 MOMENTUM EQUATION

The momentum equation is a statement of Newton’s second law of motion, which states that the rate of change of momentum or acceleration (a) of a fluid of mass (m) is directly proportional to the sum of the forces (ΣF_x) acting on it. It can be written as:

$$\sum F_x = ma$$

The change in momentum of a body of water enclosed by sections 1 and 2 over unit time can be written by applying Newton’s second law as:

$$P_2 - P_1 + W_x - F_f = Q\rho\Delta V_x$$

Where:

P = Hydrostatic pressure force at sections 1 and 2

W_X = Force due to the weight of the water in the flow direction, x

F_f = Force due to *external* frictional resistance between sections 1 and 2

Q = Discharge

ρ = Density of water

ΔV_x = Change in velocity between sections 1 and 2 in the x direction

The momentum equation is applicable to rapidly varying flow situations where the flow passes through critical depth and the head loss due to *external* resistance predominates. This occurs when the flow transitions from sub-critical to supercritical or vice versa. Situations where the momentum equation may be particularly applicable include drop structures or weirs, occurrence of hydraulic jumps due to large changes in channel slope, significant constrictions at bridges and at stream confluences.

7.4.3 HYDRAULIC ANALYSIS METHODS

Two methods commonly used in the hydraulic analysis of open channels are the single cross-section method and the water surface profile method.

The single cross-section method is a simple application of Manning's equation that can be used where uniform or nearly uniform flow conditions can be assumed. This method will generally yield less reliable results than the water surface profile method because it requires more judgment and assumptions. It is appropriate for roadside ditches, but usually not applicable for natural channels or major channels with irregular cross-sections and non-uniform flow conditions.

7.4.3.1 WATER SURFACE PROFILE ANALYSIS

Water surface profile analysis is used to compute steady or unsteady, gradually varied flow water surface profiles in major channels using computer programs such as HEC-RAS or WSPRO. HEC-RAS is also capable of computing supercritical flow and/or mixed flow (both sub-critical and supercritical) profiles. Sub-critical water surface profiles are computed from one cross-section to the next by solving the energy equation with an iterative procedure. This method is often referred to as the “step-backwater method” because the computations proceed in the upstream direction by stepping from section to section. However, the term “backwater” would not be appropriate for a channel reach with supercritical flow because the computations in that case would proceed in the downstream direction.

Water surface profile analysis requires a much greater level of detail compared to the single cross-section analysis. This level of detail is usually justified by the risk of damage or loss of life that can be caused by a failure of the facility. The designer should refer to the computer program user’s manuals for detailed guidance. Some of the more important hydraulic modeling aspects are discussed below.

7.4.3.2 BACKUP DATA

The first line of approach should be to check if FEMA flood insurance studies (FISs) and flood insurance rate maps (FIRMs) are available for the subject stream. FISs in PDF format and FIRMs (in TIFF format) organized by WVDOH District are available on the WVDOH Design Division server. Each WVDOH District Bridge Engineer also has a CD of the flood studies in the District. The guidance provided in Chapter 4 (Hydrology) should be followed to determine what frequency discharges are required for the project. In most cases, the discharges in the FIS can be used. The 100-year base flood elevations (BFEs) on the published FIRMs are usually based on detailed hydraulic models developed from topographic mapping and cross-sections. Therefore, it is cost-effective to obtain the FIS hydraulic model from FEMA and use it as the baseline hydraulic model.

FEMA backup data is in the public domain and can be obtained for a small fee from the FEMA Project Library by contacting the Library Manager at Michael Baker Jr., Inc., currently FEMA's National Service Provider, at (703) 960-8800.

If a FEMA study is not available, inquiries should be made to determine if hydrology and hydraulics data are available from other federal agencies such as the USACE, NRCS or the USGS. Telephone contact information for these federal agencies is provided below:

- USACE Huntington District, Phil Anderson, (304) 399 5603
- USACE Pittsburgh District, Nancy Pitrowski, (412) 395 7226
- USACE Pittsburgh District, Jim Koski, (412) 395-7346
- NRCS, West Virginia State Office, Tim Ridley, (304) 284 7573

Other State or local agencies, non-profit organizations and even private parties (e.g., land developers) may also have conducted hydraulic studies that might be useful.

7.4.3.3 FIELD RECONNAISSANCE

The designer should perform the field reconnaissance using a work map (e.g., FIRM, USGS Quad), preferably before the cross-sections are surveyed so that hydraulic controls in the channel can be identified and communicated to the survey party chief in advance of a survey. This preliminary field survey can save the cost of resurveys that may be required at a later time. The field survey should document conditions such as:

- channel and overbank roughness values
- control sections at bridges or culverts
- upstream and downstream reach controls
- ordinary high water
- location of nearby stream confluences
- high water marks from past floods, and
- presence of levees

Photographs should also be taken.

7.4.3.4 BOUNDARY CONDITIONS

If a FEMA hydraulic model is being used as the base model, the known water surface elevation at a cross-section downstream of the channel project location can be chosen as the downstream boundary condition for starting the water surface profile computations.

If a baseline model is unavailable, the downstream boundary condition has to be determined for each profile to be computed. A common practice is to use normal depth as the starting water surface elevation. Sometimes the boundary condition may be assumed to be equal to critical depth if the model begins at a known control section such as a weir. If the channel reach under consideration is steep and there is a possibility that a supercritical or mixed flow profile needs to be evaluated, then an upstream boundary condition will also need to be specified.

7.4.3.5 STUDY LIMITS

Determination of the upstream and downstream limits of the model requires experience and judgment. In general, the channels with steeper slopes will quickly converge to a consistent answer and channels with milder slopes will require greater lengths for convergence. It is recommended that the model limits be selected after giving consideration to the methods presented in this section.

The proper location of the downstream study limit (or boundary) of the model ensures that the results within the project reach are not affected by any user-defined boundary conditions. If the downstream boundary condition is known, the downstream boundary can be located close to the project. If the downstream boundary condition is unknown and normal depth is used as an approximation of the downstream boundary starting condition, it will introduce an error in the water surface profile at the boundary. This usually diminishes as the profile proceeds upstream. In order to prevent this error from propagating into the project reach, the downstream boundary cross-section should be placed far enough downstream so that the computed profile converges to a consistent answer by the time the computations reach the downstream limit of the project.

The location of the upstream limit of the model is important in order to evaluate any upstream impacts due to the project. The upstream limit should be far enough to allow any changes in the water surface elevations due to the project to converge with the baseline conditions' water surface elevations upstream. Similarly for a supercritical flow profile the upstream limit (or boundary) should be located far enough upstream so that water surface profile at the upstream end of the project is not affected.

The following approximate equation developed by WVDOH can be used to approximate the length of the reach needed to ensure convergence of the computed backwater curves upstream and downstream of the project location:

$$L = 0.1\frac{Y_n^{0.1}}{S_o^{1.5}}$$

Where:

L = Total reach length, ft

Y_n = Normal depth, ft

S_O = Bed slope, ft/ft

Bed slope and normal depth must be estimated.

The Hydrologic Engineering Center (HEC), USACE, conducted a study (Accuracy of Computed Water Surface Profiles, December 1986) for the FHWA, and developed the following regression equations for determining upstream and downstream reach limits for the normal depth starting condition:

$$L_{DN} = 8000\left(\frac{HD^{0.8}}{S}\right)$$

$$L_U = 10000 \left(HD^{0.6}\right)\left(\frac{HL^{0.5}}{S}\right)$$

Where:

L_{DN} = Downstream study length

L_U = Upstream study length

HD = Average hydraulic depth for the 100-year discharge

S = Average reach slope in feet per mile

HL = Maximum expected head loss between 0.5 and 5 feet at the structure for the 100-year discharge

The USGS recommends the following “rules of thumb” for determining the necessary length of the model reach:

- 10 times the width of the flood event being studied
- 75 times the mean depth
- Minimum of 0.5 foot of fall
- Fall greater than or equal to the velocity head

Conducting a sensitivity analysis will allow the designer to be sure that the selected boundary location is appropriate. Vary the boundary condition and compare the water surface elevation at the point of interest (usually the bridge) to ensure that it is not effected by any error in the boundary condition.

7.4.3.6 CROSS-SECTION LAYOUT AND SPACING

Cross-sections should be located on the preliminary work map with consideration given to:

- Changes in discharge
- Changes in slope
- Changes in shape
- Changes in roughness
- Where levees begin and end
- Upstream and downstream bounding sections of bridges and structures

Cross-sections should be drawn with bends or angles as needed so that the channel portion of the cross-section is perpendicular to the flow direction in the channel and the overbank portion of the cross-section is perpendicular to the flow direction in the floodplain.

Maximum cross-section spacing depends on the purpose of the study, the hydraulic characteristics of the stream and the accuracy desired. In general, large rivers with flat slopes and man-made prismatic channels require few cross-sections, but small rivers and streams require more closely spaced cross-sections. The accuracy of the computed water surface profile increases with closer cross-section spacing, but the cost of obtaining surveys can be a limiting factor. Therefore, as a rule of thumb, the cross-section spacing should be at least equal to the width of the main channel and up to a maximum of 500 feet for irregular sections.

7.4.3.7 SURVEYS

Determine if an existing hydraulic model is available from other sources before conducting surveys. Cross-sections should be surveyed as shown on the work map and tie into high ground wherever possible. If topographic mapping is unavailable, the cross-section surveys should extend beyond the 100-year floodplain whenever possible. If topographic mapping is available, but the stream channel is underwater, hydrographic surveys will be required to define the channel geometry and tie it in with the overbank geometry.

The following features should be included in each cross-section:

- Edge of water
- Thalweg (deepest part of the channel)
- Top of Bank
- Breaks in grade (islands, point bars, etc.)
- Ordinary High Water (OHW)
- Bankfull Indicators
- Expected limits of floodprone area (may be determined from USGS maps)

A profile of the thalweg should also be surveyed for the model reach. The survey should include shots at each break in the profile grade and at regular intervals.

7.4.3.8 MANNING'S ROUGHNESS VALUE SELECTION

Proper selection of Manning's roughness coefficients is very important to the accuracy of the computed water surface profiles. This value is highly variable and depends on a number of factors such as seasonal vegetation, size and shape of the channel, obstructions, stage, discharge, temperature, scour and deposition and suspended material and bedload. Considerable experience and judgment is required in its selection.

Manning's roughness values are first estimated during the site visit and should be calibrated if surveyed high water marks or gaged data are available. Figure 7-3 provides a range of Manning's coefficient values for various types of natural channels and floodplains. Separate roughness values are generally assigned to the left overbank, channel and right overbank of the cross-section.

Although it is quite common to use a single set of roughness values for modeling a series of discharges ranging from low to high, it may not always be appropriate. Manning's roughness values should be evaluated carefully to determine if the selected values are representative at all stages being modeled. Generally, flow resistance and hence roughness values would be higher at lower stages in natural, irregular channels. Depending on the desired accuracy of the water surface profiles at lower flows, a separate model with revised roughness values can be developed for the low flows.

The following publications can be useful for a better estimating roughness values in irregular channels:

- Roughness Characteristics of Natural Channels, Harry H. Barnes, Jr., USGS Water Supply Paper 1849, 1967
- Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains, Arcement, G.J., and Schneider, V.R., FHWA-TS-84-204, April 1984

7.4.3.9 Model Calibration and Sensitivity

If high water mark survey profiles and the corresponding discharge from gaged data are available, the model should be calibrated by adjusting the roughness values. If calibration data are unavailable, a sensitivity analysis may be required to determine the sensitivity of the input variables such as the estimated roughness values, cross-section spacing, boundary conditions and coefficients. Project managers and designers should work together to determine the extent of sensitivity analysis needed.

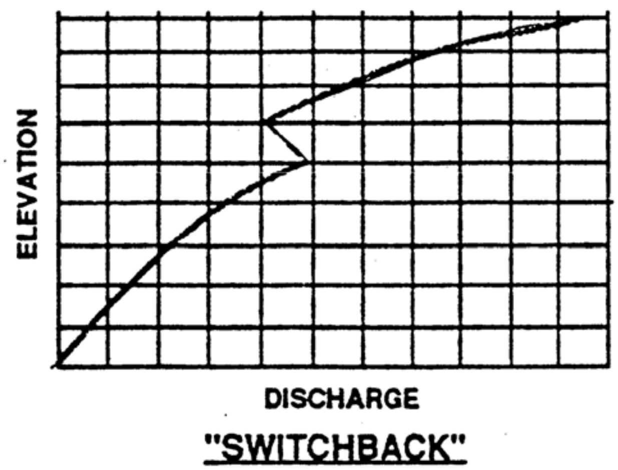
7.4.3.10 Switchback Phenomenon

If the cross-section is improperly subdivided, the mathematics of the Manning's Equation causes a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation.

This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area which causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. More subdivisions within such cross-sections should be used in order to avoid the switchback.

Figure 7-3

Switchback Phenomenon



This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step-backwater program. For this reason, the cross-section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value, itself, may be the same in adjacent subsections.

7.4.4 CHANNEL STABILITY EVALUATION

A channel stability assessment is useful in detecting existing or potential instability problems. The need for a stability evaluation usually becomes apparent during the preliminary field visit. Unstable streams readily exhibit characteristics such as failing banks, incised channel bed and headcuts. If the stream appears to be unstable, a stability analysis and corrective measures are strongly recommended. In most cases, qualitative predictions using geomorphic principles are sufficient to provide insight into the general trends of channel response to natural and/or manmade changes in the watershed and river system. But quantitative geomorphic and engineering techniques are required if remedial designs or countermeasures are needed to stabilize the channel. Trends are usually straight-line extrapolated to predict the expected change. A stability evaluation usually consists of an assessment of vertical and lateral stability of the channel. Detailed guidance can be found in FHWA's HEC-20 publication.

7.4.4.1 QUALITATIVE ASSESSMENT

Human activities are frequently the cause of channel instability. Information on urbanization, land clearing, channel modifications, meander cutoffs, streambed mining, dam construction, reservoir operations, navigation projects and other activities either existing or planned can provide insight on the past and future impact on stream stability. Changes in stream morphology rarely occur at a uniform rate and can be associated with an extreme event or a major disturbance in the channel system or watershed.

Data requirements for a preliminary qualitative assessment include time sequence topographic maps and aerial photographs, field inspection notes and photographs, historic channel profile data, historic channel bed elevations from as-built plans of nearby structures, and information on changes in the watershed and channel over time.

Vertical instability over the long term can be assessed by comparing the rate of change of the channel bed elevations upstream and downstream of the project using historic channel bed profile and/or bed elevation data, historic cross-section data if available and field inspection reports. Field inspection reports for highway structures can be obtained from the WVDOH Maintenance Division. Before and after comparisons can also be made using streambed elevations from as-built plans of nearby structures. Stage trends at stream gaging stations available from USGS can also provide information on changes in the stream profile.

Lateral stability, particularly rate of bend migration can be assessed by recording the position of the bend at two or more different times and comparing the difference using time sequence aerial photographs available from the USGS or the NRCS. Channel avulsion is the shifting of the of the flow path of the stream into a new channel in the vicinity of the highway project. Aerial photographs can also reveal a history of consistent overbank flooding, where evidence of the potential for stream capture exists in the form of a different stream channel. The potential for channel avulsions can be seen on topographic maps that show a perched channel above the surrounding alluvial surface with the inevitability of avulsion.

Geologic maps provide information on deposits, rock formations, and outcrops that control stream stability. Soils and land use maps provide information on soil types, vegetative cover and land use that affect the character and availability of sediment supply. Information from local officials such as the WVDOH, NRCS and USGS District staff can also prove very useful.

7.4.4.2 QUANTITATIVE ASSESSMENT

A detailed treatment of the quantitative methods is beyond the scope of this chapter. FHWA's HEC-20 publication should be consulted for a detailed discussion of lateral and vertical stability methods.

According to HEC-20, the maximum lateral erosion distance ranges between 2.5 to 3.5 times the channel width associated with the dominant discharge. The dominant discharge (also known as the bankfull or channel forming discharge) is the flow, which determines the principal dimensions and characteristics of the natural channel. It is normally assumed to be the discharge with a return period of 1.5 years in natural channels in the eastern United States.

The following methods are available in HEC-20 for evaluating vertical stability:

- Incipient Motion and Armoring Analysis
- Equilibrium Slope Analysis
- Sediment Continuity Analysis

Incipient motion can be used to calculate the sediment particle size that will move for a particular hydraulic condition or calculate the shear stress required to move a particular particle size. Channel armoring occurs when the hydraulic forces are unable to move the larger sizes of the bed material, which results in the formation of a coarse layer or armor layer on the bed. Armor layers usually form in gravel bed rivers as the flood recedes. A stable armor layer can prevent further degradation.

Equilibrium slope methods are based on Lane's relationship, which states that a reduction in sediment supply or an increase in discharge can cause channel degradation and a reduction in slope. Degradation will proceed until the formation of an armor layer stops. The slope at this condition is called the equilibrium slope. This state of equilibrium is also called dynamic equilibrium and can change over time. Dynamic equilibrium is a useful and practical concept, which can be considered to be stable over the engineering time-scale as long as changes do not exceed acceptable levels. A simple method developed by the US Bureau of Reclamation (USBR) to calculate equilibrium slope is presented in HEC-20.

The sediment continuity concept states that the sediment inflow (i.e., sediment supply from the watershed) minus the sediment outflow (i.e., sediment transport capacity) equals the time rate of change of sediment volume in a reach. The channel system is considered to be in dynamic equilibrium (i.e., stable) when the sediment supply equals the sediment transport capacity. If the sediment supply is less than transport capacity, erosion (degradation) will occur. If the sediment supply is greater than the transport capacity, deposition (aggradation) will occur. HEC-20 presents a procedure to calculate the change in bed elevation for a channel reach using the sediment continuity concept. The designer should refer to FHWA's HDS-6 (River Engineering for Highway Encroachments) for a detailed discussion of various sediment transport equations. The HEC-RAS computer program can also be used to compute the sediment transport capacity of a single cross-section using several equations.

Sediment transport modeling or sediment routing should be considered if stream stability problems are encountered which cannot be assessed with the simple techniques found in HEC-20. Sediment transport modeling is quite complex, and requires specialized expertise that is outside the scope of this manual as well as HEC-20. The BRI-STARS (Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation) computer program developed by the National Cooperative Highway Research Program and the FHWA, or the HEC-6 (Scour and Deposition in Alluvial Rivers) computer program developed by the USACE, are widely used for single event or long-term degradation estimates.

7.4.5 STABLE CHANNEL DESIGN CONCEPTS

Stable channel design concepts presented here are based on the procedures in FHWA's HEC-15 publication. Stable channel design concepts are used to evaluate and design channel boundary protection that will perform within acceptable limits of stability. Stability depends on whether the channel boundaries can be viewed as rigid or flexible. Rigid boundaries can resist the erosive forces and remain unchanged for almost all flow conditions (i.e., channel is in static equilibrium). If erosive forces can detach and transport the channel boundary materials, the channel boundary is said to be flexible and stability is based on the sediment continuity principle. That is, the sediment supply is equal to sediment transport and the system is in dynamic equilibrium.

For most highway projects involving channel design, bank instability and possible lateral migration cannot be tolerated. Consequently, development of static equilibrium conditions or utilization of rigid or flexible linings to achieve a stable condition is usually preferable to using dynamic equilibrium concepts.

For relocated channels however, the design channel dimensions (width and depth) and profile (longitudinal slope) of the channel reach should be checked for sediment continuity so that there is a balance between the available sediment supply and sediment transport capacity (i.e., dynamic equilibrium).

Two common design methods used to determine if a channel is stable (i.e., in static equilibrium) are the permissible velocity approach and the permissible shear stress (or Tractive Force) approach.

7.4.5.1 PERMISSIBLE SHEAR STRESS

The permissible shear stress or tractive force approach became recognized in the 1950's, based on research conducted by the US Bureau of Reclamation (USBR) and later developed by the USDA SCS. This method is more physically based than the permissible velocity approach covered in Chapter 6, and focuses on stresses developed at the interface between flowing water and the materials forming the channel boundary. Physical factors such as the bed material, channel geometry, depth and velocity of flow are taken into account. Permissible shear stress is the force required to initiate movement of the channel boundary material. Permissible shear stress values for non-cohesive and cohesive soils as a function of particle diameter and plasticity index, respectively, can be found in Chapter 6 (Ditches).

The tractive force approach is the basis for stable channel design of flexible linings. The failure criteria for the lining material is represented by a single shear stress value which is applicable over a wide range of channel slopes and shapes. The tractive force for the design discharge should not exceed the permissible or the critical shear stress of the lining material. Shear stress in channels is not uniformly distributed along the wetted perimeter.

The maximum shear stress in a straight channel occurs on the channel bed and is given by:

$$\tau_d = \gamma d S$$

Where:

τ_d = Maximum shear stress in a straight channel, lbs/ft^2

γ = Unit weight of water, lb/ft^3

d = Maximum depth of flow, ft

S = Average bed slope of the channel or energy slope, ft/ft

Flow in a channel bend creates secondary currents, which impose higher shear stresses on the channel sides compared to a straight channel reach, as shown in Figure 7-4. At the beginning of the bend, the maximum shear stress is near the inside and moves towards the outside as flow leaves the bend. The maximum shear stress in a bend is a function of the ratio of channel curvature to bottom width (R_c/B). As the bend becomes sharper (i.e., as Rc/B decreases), the maximum shear stress in the bend increases (See Chart 7-1).

The maximum shear stress in a channel bend is given by:

$$\tau_b = K_b \tau_d$$

Where:

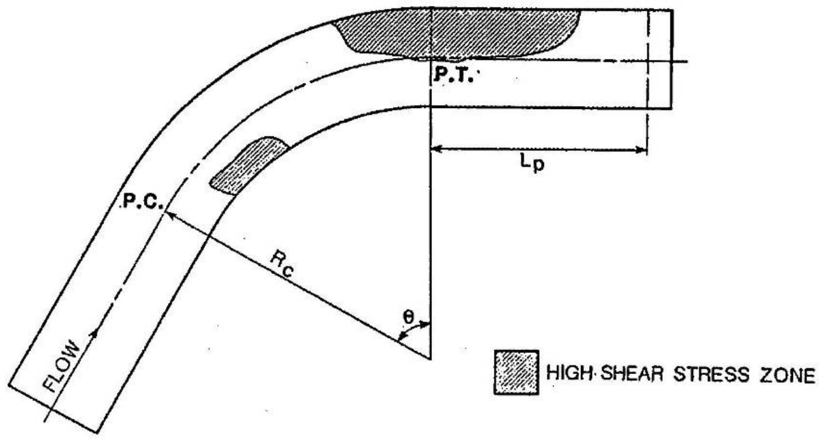
τ_b = Maximum bend shear stress, lbs/ft^2

τ_d = Maximum shear stress in an equivalent straight section of channel, lbs/ft^2

K_b = Dimensionless factor given by Chart 7-1

Figure 7-4

Shear Stress Distribution in a Channel Bend



The increased shear stress caused by the bend persists a distance L_p, downstream of the bend. See Figure 7-6 for additional length of protection required.

7.4.6 CHANNEL PROTECTION DESIGN

The need for channel protection is predicated on the fact that the native soil material may be displaced by design flows in the channel. The first step in the design process is to determine whether or not the native material will be displaced. The tractive force method is usually employed to make this determination. A geotechnical investigation and soil survey should be conducted through areas where a channel work is proposed. Borings along the proposed channel are to be taken at sufficient intervals to determine the type of material encountered along the banks and in the bottom of the channel.

Channels can be protected against erosion through the use of properly designed channel linings or revetments. A revetment is defined as a blanket of stone or concrete used to protect against the effects of erosion or scour.

Site conditions can vary significantly and there are a wide variety of materials available for channel protection. In general, “cookbook” solutions are impractical and lead to poor design. Good design practice requires judgment and experience. Therefore, the designer is cautioned that the methods presented should not be used as fixed solutions but rather to aid engineering judgment and experience.

The design of flexible linings with vegetation should be based on FHWA's HEC-15 and is covered in Chapter 6 (Ditches).

This section covers the design of rock protection. For other types of commercially available materials such as gabions or ACB (Articulated Concrete Block) systems, the designer should consult design publications and software available commercially through the manufacturers of these materials.

7.4.6.1 TYPES OF LININGS

Channel protection linings may be classified as "soft" (e.g., grass or other vegetation) or "hard" (e.g., rock riprap, stone masonry, concrete or articulated concrete blocks), based on their appearance.

Linings can also be classified as "flexible" or "rigid" based on their mode of operation. Flexible linings such as rock riprap, wire enclosed stone (gabions) and vegetation are "self-repairing" and can conform to the channel shape when subjected to minor displacements. Rigid linings such as concrete, grouted riprap or stone masonry tend to fail when a portion of the lining is damaged. Other examples of rigid linings are cast-in-place concrete, reinforced concrete, cast-in-place asphaltic concrete and soil cement. Damage can result from forces such as frost heave, bank slumping or toe scour. Flexible linings are generally less expensive, permit infiltration and exfiltration of water into the channel banks and are aesthetically more pleasing than rigid linings. Vegetative linings are suited to hydraulic conditions where shear stresses are moderate and the lining is not subjected to sustained flow conditions or long periods of submergence.

Flexible lining consisting of erosion resistant vegetation should be used whenever possible and may require the use of either a temporary protective matting or a permanent matting (see WVDOH Standard Specifications for Roads and Bridges, Section 715.24).

Rigid linings may be used on steep grades with high velocities, areas where channel width is restricted or areas where higher flow capacity (flood protection) is needed. Rigid linings may require channel protection or energy dissipation at the termination point to prevent scour due to the high outlet velocities. Rigid linings have a tendency to be damaged or destroyed by flow undercutting the lining at bends (toe scour) or by flow that overtops the lining, if designed improperly. Therefore, rigid lining design should include details needed to protect from toe scour and overtopping conditions. The design of rigid linings is not covered in this chapter because rigid linings such as concrete are seldom used due to environmental restrictions.

7.4.6.2 FLEXIBLE LINING DESIGN WITH ROCK

The design of flexible lining with rock for major channels should be based on FHWA's HEC-11 publication (Design of Riprap Revetment). Rock lining for minor channels (roadside ditches) should be based on FHWA's HEC-15 publication (Design of Roadside Channels with Flexible Linings).

7.4.6.3 TERMINOLOGY

WVDOH Standard Specifications (2000) require that the plan and sequence of construction operations make use of all rock obtained from excavation or borrow (Section 207.7.3.1). Therefore, it is important for the designer to understand the use of standard terms used to describe protection with rock in channels.

A rock-lined channel with a bottom width of 8 feet or more is designated as a "**Select Embankment Channel**" as shown on the WVDOH Standard Details. Select Embankment is rock from unclassified excavation that is typically used for constructing highway embankment lifts, but can also be used for lining drainage channels where hydraulic conditions permit. Select embankment rock can consist of no more than 15% of other suitable embankment material (by visual inspection), where the dominant size of the rock is at least 6 inches but not greater than 36 inches (Section 207.7.3.2.3). The maximum rock size for select embankment can equal the thickness of the blanket and the minimum rock size is usually one-half of the blanket thickness. Therefore, select embankment stone is not well graded and may not be suitable for all hydraulic situations. If hydraulic conditions warrant, the required stone gradation should be specified on the plans.

Rock lined channels with a bottom width less than 8 feet are designated as a "**Dumped Rock Gutter**", as shown on the WVDOH Standard Details. Dumped Rock Gutter also consists of rock obtained from unclassified excavation whose weight requirements based on the thickness of the gutter are specified in WVDOH Standard Specification 704.4. Dumped rock gutter is so called because hand placement is not required and it is dumped from trucks and bulldozed into place in a manner similar to placing rock fill (Section 633.6).

The locations of probable sources of rock for Dumped Rock Gutter and Select Embankment should be listed on the plans. "Rock borrow" is rock that has to be brought from off site because it is not available on site. Other terms in the

Standard Specifications Book such as grouted riprap, gabions and crushed rock slope protection are described in Section 218. Stone for masonry, riprap, gabions, special rock fill, aggregate, filter material are described in Section 704.

Riprap is defined as a flexible channel lining consisting of a well-graded mixture of rock, usually dumped or hand placed, to provide protection from erosion. It should be noted that riprap as defined in HEC-11 is different from the description provided in Section 218.3.2 of the WVDOH Specifications. Riprap as defined in HEC-11 is referred to as "rock lining" in this chapter.

7.4.6.4 ROCK SIZING (HEC-11)

The median size (D_{50}) of stable rock lining for uniform, gradually varying flow conditions in straight to mildly curving channels with a relatively uniform cross-section is given by:

$$D_{50} = CD_{50}'$$

Where: $C = C_{sg} S_{sf}$

$$D_{50}' = \frac{0.001\, V_a^3}{d_{avg}^{0.5} K_1^{1.5}} \qquad K_1 = \left[1 - \left(\frac{\sin^2\theta}{\sin^2\phi}\right)\right]^{0.5}$$

$$C_{sg} = \frac{2.12}{(S_s - 1)^{1.5}} \qquad S_{sf} = \left(\frac{SF}{1.2}\right)^{1.5}$$

D_{50} = Median corrected size of stable rock particle, ft

C = Correction Factor for rock size

D_{50}' = Median size of rock without correction, ft

C_{sg} = Correction Factor for rock with specific gravity other than 2.65

S_{sf} = Correction Factor for rock stability factor (SF)

V_a = Average velocity in the main channel, ft/s

d_{avg} = Average flow depth in the main channel, ft

K_1 = Bank angle correction factor

θ = Bank angle with the horizontal, degrees

Φ = Rock material angle of repose (Chart 7-5)

S_s = Specific gravity of stone=γ_s / γ_w

γ_s = Unit weight of stone, generally, 155 lbs/ft^3

γ_w = Unit weight of water, 62.4 lbs/ft^3

SF = Stability Factor

Chart 7-2, Chart 7-3, Chart 7-4, Chart 7-5 and Chart 7-6 can be used to determine the D_{50} of the stone.

The stability factor (SF) reflects the level of uncertainty in the hydraulic conditions at the site. SF is defined as the ratio of the average tractive force exerted by the flow field and the critical shear stress of the riprap material. The riprap is considered stable if SF is greater than 1. A stability factor of 1.2 was assumed in the development of the above equation, which assumes uniform or gradually varying flow conditions. These conditions are violated in many instances such as in bends, on steep slopes, locations with floating debris, and ice and wave impacts. In such cases, Table 7-1 should be used as a guide to select a stability factor that accounts for uncertain hydraulic conditions.

Table 7-1

Guide for Selecting Stability Factor (SF)

Condition	Stability Factor Range
Uniform flow; straight or mildly curving reach (curve radius/channel width>30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters	1.0 – 1.2
Gradually varying flow; Moderate bend curvature (30> curve radius/channel width> 10); Impact from waves or floating debris is moderate	1.3 – 1.6
Approaching rapidly varying flow; Sharp bend curvature (10>curve radius/channel width); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (1 to 2 feet); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 – 2.0

Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

7.4.6.5 OTHER ROCK SIZING METHODS

It is good design practice to check the stone size using other methods and adjust as necessary. The designer should refer to the following publications for sizing rock protection using other methods:

- USACE Method: Engineer Manual 1110-2-1601, June 30, 1994, “Hydraulic Design of Flood Control Channels.”
- ASCE Method: Vanoni, V.A. (ed.), 1977, “Sedimentation Engineering,” ASCE Manuals and Reports on Engineering Practice No. 54
- USBR Method: Peterka, A.J., 1958, “Hydraulic Design of Stilling Basins and Energy Dissipators.” USBR Engineering Monograph No. 25
- USGS Method: Blodgett, J.C., 1981, “Flood Flow Characteristics of the Sacramento River in the Vicinity of Gianella Bridge, Hamilton City, California.” USGS Open File Report 81-328.
- Isbash Method: Isbash, S.V., 1936, “Construction of Dams by Depositing Rock in Running Water.” Transactions of the Second Congress on Large Dams, Volume V, Communication No. 3, U.S. Government Printing Office, Washington D.C.

- Caltrans Method: California Department of Public Works, Division of Highways, 1970. “Bank and Shore Protection in California Highway Practice.”

7.4.6.6 LAYER THICKNESS

To provide maximum resistance against erosion, all stone should be contained reasonably well within the riprap layer thickness. The following criteria apply to layer thickness selection:

- Layer thickness should not be less than the spherical diameter of the D_{100} stone or less than 1.5 times the spherical diameter of the D_{50} stone, whichever results in the greater thickness.
- Layer thickness should not be less than 12 inches for practical placement.
- Layer thickness determined by either of the above criteria should be increased by 50 percent to account for uncertainties associated with riprap placement underwater.
- Layer thickness should be increased by 6 to 12 inches accompanied by an appropriate increase in stone sizes where the riprap revetment is subject to attack by floating debris, ice, waves from boat wakes, wind or bedforms.

7.4.6.7 ROCK GRADATION

The rock gradation can affect the performance of riprap. The stone should be reasonably well graded from the smallest to the maximum size specified, throughout the thickness of the riprap. Riprap gradation is controlled by visual inspection. Stones smaller than the specified 5 or 10 percent size should not be permitted in an amount exceeding 20 percent by weight of each load. The gradation should be specified if Select Embankment stone is to be used as riprap. Table 7-2 presents three gradation classes from HEC-11 based on AASHTO specifications.

Table 7-2
Rock Gradation Limits

Stone Size Range * (feet)	Stone Weight Range (lbs)	% of Rock Smaller Than
1.5 D_{50} to 1.7 D_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.0 D_{50} to 1.15 D_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 D_{50} to 0.6 D_{50}	0.1 W_{50} to 0.2 W_{50}	15

7.4.6.8 FILTER DESIGN

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the riprap layer. The filter prevents the migration of fine soil particles through the voids in the riprap layer, distributes the weight of the stone units to provide more uniform settlement and permits relief of hydrostatic pressures within the underlying soils.

The proper design of granular or fabric filters is critical to the stability of riprap installations on channel banks. If openings in the filter are too large, flow through the filter can cause soil particles to migrate through the filter and result in erosion and collapse of the bank material beneath the filter. On the other hand, if the openings in the filter are too small, the buildup of hydrostatic pressures behind the filter can cause a slip plane to form along the filter resulting in a massive slide failure of the bank. The designer should refer to HEC-11 if site conditions require the design of granular filters.

Fabric filters made of geotextiles are recommended for most applications because fabrics have definite advantages over granular filters in many applications. Fabric is less expensive and easier to obtain and install compared to granular filter material. However, fabric has some disadvantages in that it can be difficult to lay underwater and may induce slump failures in riprap installations steeper than 2H:1V. Filter fabric is available though several commercial manufacturers and should be selected and installed in strict accordance with the manufacturer's specifications.

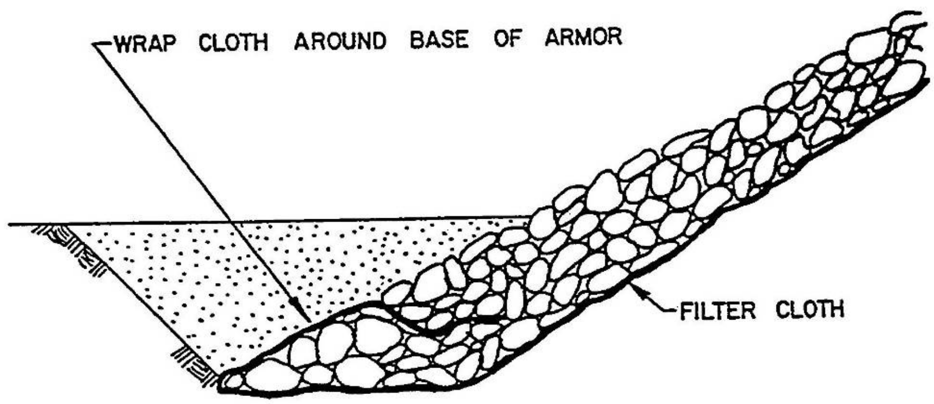
Filter fabric should be selected based on the following considerations:

- Fabric should be of adequate strength so that it does not stretch and rupture due to the tension caused by heavy riprap. To prevent rupture of the fabric, stones should not be dropped from heights greater than 2 feet.
- The fabric should be wrapped around the toe material as shown in Figure 7-5 (From FHWA's HEC-11, March 1989). The end of the fabric should be

embedded into the rock layer for a length equal to the thickness of the rock layer.

Figure 7-5

Filter Fabric Placement



- Adequate overlaps should be provided between the adjoining fabric sheets per the manufacturer's specifications. Overlapping fabric should be secured with adequately spaced pins as recommended by the manufacturer.
- Stone placement on the filter should begin at the toe and proceed up the slope.

7.4.6.9 EXTENT OF PROTECTION

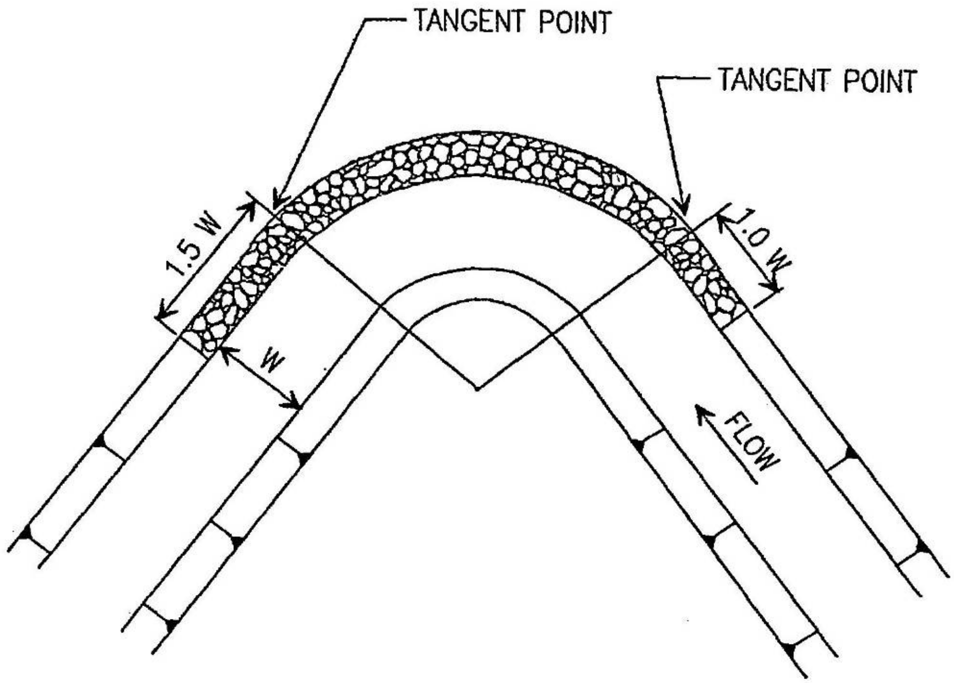
Extent of protection refers to the longitudinal and vertical extent of protection.

The longitudinal extent of protection is highly dependent on local site conditions. The methods presented here are not definitive and should only be used as a starting point or a guide. The actual length of protection should be based on engineering judgment and field conditions. In general, the length of protection should be a distance greater than the length that is impacted by channel flow forces severe enough to cause transport of bank material. A common misconception is to provide protection too far upstream and not far enough downstream.

The required length of protection can be determined using the procedure described in Section 7.4.5.1 or using the guidance in Figure 7-6 taken from FHWA's HEC-11.

Figure 7-6

Longitudinal Extent of Protection



As shown in Figure 7-6, the minimum upstream distance should be equal to the channel width and the minimum downstream distance should be at least 1.5 times the channel width from the corresponding tangent points. This guidance is based on studies of a symmetrical channel bend under ideal laboratory conditions. Real-world conditions are rarely as simplistic and the distances should be increased to reflect site conditions.

The vertical extent of protection for the rock riprap revetment includes design height and toe depth. The design height should be equal to the top of the bank elevation or the design flow depth plus freeboard whichever is greater. Hydraulic conditions such as wave action, superelevation in bends and hydraulic jumps should be considered in establishing the freeboard. The toe depth is determined by the total toe scour estimate. The total toe scour estimate is the sum of predicted long-term degradation and local bend scour depth (if any). Long-term channel degradation is determined using qualitative or quantitative procedures described in Section 7.4.4, based on FHWA's HEC-20. Local bend scour should be determined using the USACE's EM 1110-2-1601, Plate B-42 (June 30, 1994).

7.4.6.10 EDGE TREATMENT

The edges of riprap revetments (head, toe and flanks) require special treatment to prevent undermining.

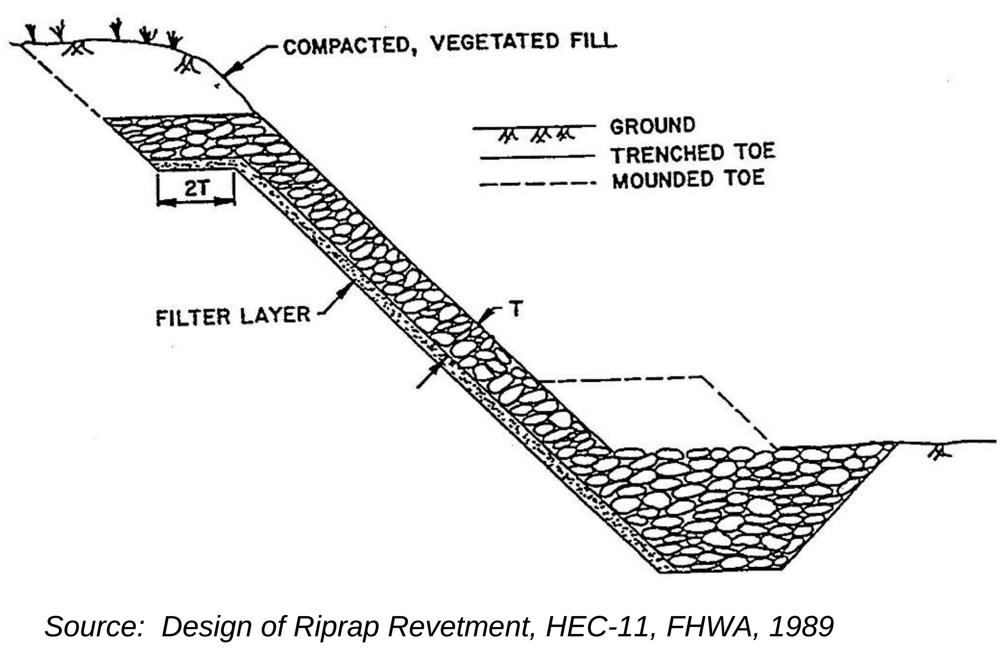
The upstream and downstream flanks should be detailed as shown in Chart 7-7. The compacted backfill shown in the upstream flank section can be replaced with riprap for added protection.

The riprap revetment should be embedded into the channel bank as shown in Figure 7-7 to provide protection at the head. The elevation of the head of the revetment should be extended to either the top of the channel bank or the design water surface elevation plus the required freeboard, whichever is greater.

The riprap revetment should preferably be extended down to the total toe scour depth, but this is not always feasible because it could involve considerable earthwork and may not be practical in underwater installations. Therefore, the riprap material is usually placed in a toe trench as shown in Figure 7-7. The size of the trench is dependent upon the total toe scour estimate.

Figure 7-7

Typical Riprap Toe Protection (End View)

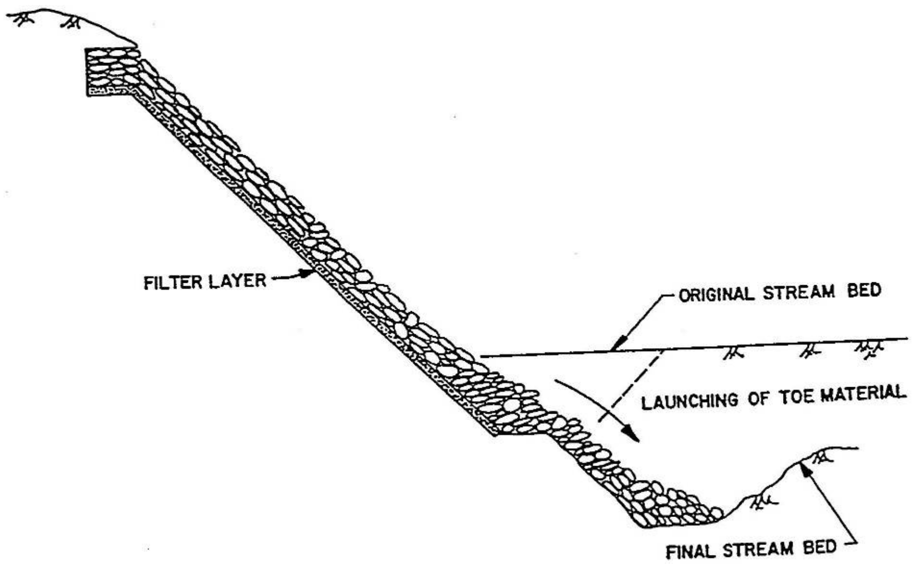


Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

As scour occurs, the toe material will launch into the eroded area as shown in Figure 7-8, at an approximate slope of 2H:1V.

Figure 7-8

Typical Launching of Riprap Toe Material



Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

The volume of rock required for the toe trench should be at least 1.5 times the volume of rock required to extend the riprap blanket (at its design thickness and at a slope of 2H:1V) to the estimated total toe scour depth.

7.4.6.11 ROCK QUALITY

Riprap stone should be hard, dense and durable. The stone should be resistant to weathering, free from overburden, spoil, shale and organic material. Rock that is laminated, fractured, porous or otherwise physically weak is unacceptable.

Stone should be preferably angular in shape. Flat slab like stones should be avoided since they are easily dislodged by the flow. An approximate guide to stone shape is that neither the breadth nor the thickness of the stone should be less than one-third its length.

7.4.6.12 CONSTRUCTION

The maximum recommended bank slope for stable riprap installations is 2H:1V. Before installation, the bank should be cleared of all trees and debris and graded to the desired slope. Common methods of placement include by hand or machine. Hand placement produces the best revetment but it is labor intensive and expensive. Machine placement is usually accomplished with a dragline or by dumping from trucks, followed by spreading with a bulldozer. Stones should not be dropped from an excessive height while dumping to avoid segregation and breakage of stone. Care should be exercised during spreading and smoothing operations to avoid segregation and breakage of stones.

7.4.6.13 SOIL BIOENGINEERING

Soil Bioengineering consists of using a combination of hard and soft armor to stabilize banks. Live plant material such as woody vegetation is integrated into traditionally designed protection such as rock riprap. The selection of plant material is based on establishing plant communities using ecological principles and horticultural methods. The traditionally engineered structures help protect the plant communities as they grow into maturity and function as they would in their natural settings. A discussion of soil bioengineering design methods and procedures is covered in detail in the Natural Channel Design Manual.

7.4.7 DESIGN PUBLICATIONS

For detailed engineering guidance on channel design, it is strongly recommended that the designer obtain and become familiar with the following publications by the Federal Highway Administration (FHWA), U.S. Army Corps of Engineers (USACE), U.S. Bureau of Reclamation (USBR), and the U.S. Geological Survey (USGS):

FHWA Publications

- River Engineering for Highway Encroachments, Highways in the River Environment, Hydraulic Design Series Number 6 (HDS-6), Publication No. FHWA-NHI-01-004, December 2001
- Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains, Report No. FHWA-TS-84-204, April 1984
- Design of Riprap Revetment, Hydraulic Engineering Circular 11 (HEC-11), Publication No. FHWA-IP-89-016, March 1989
- Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular 14 (HEC-14), English Version (September 1983), SI Version (2001)
- Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18), 4th Edition, Publication No. FHWA NHI 01-001, May 2001
- Stream Stability at Highway Structures, Third Edition, Hydraulic Engineering Circular No. 20 (HEC-20), Publication No. FHWA-NHI-01-002, March 2001 (Note: Although this publication was intended for

assessing stream stability at highway crossings, the concepts presented are also applicable to projects involving channel design)

- Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance, Hydraulic Engineering Circular No. 23 (HEC-23), Second Edition, Publication No. FHWA NHI 01-003, March 2001

FHWA publications can be downloaded free of charge at the following website: http://www.fhwa.dot.gov/bridge/hydpub.htm.

USACE Publications

- Hydraulic Design of Flood Control Channels, Engineer Manual, EM 1110-2-1601, June 30, 1994
- River Hydraulics, Engineer Manual, EM 1110-2-1416, October 15, 1993
- Channel Stability Assessment for Flood Control Projects, Engineer Manual, EM 1110-2-1418, October 31, 1994

USACE's Engineer Manuals can be downloaded free of charge at the following website: http://www.usace.army.mil/inet/usace-docs/eng-manuals/em.htm.

USBR Publications

- Computing Degradation and Local Scour, Technical Guideline for the Bureau of Reclamation, January 1984.

USGS Publications

- Roughness Characteristics of Natural Channels, Water Supply Paper 1849, Harry H. Barnes Jr., 1967. This publication can be downloaded at http://water.usgs.gov/pubs/wsp/wsp_1849/html/pdf.html.

7.4.8 DESIGN SOFTWARE

The following computer programs and manuals are widely used to perform water surface profile computations involving one-dimensional, steady, gradually varied flow for channel design:

- User's Manual, Applications Guide, and Hydraulic Reference Manual for HEC-RAS (USACE's Hydrologic Engineering Center-River Analysis System). The computer program and manuals can be downloaded at: http://www.hec.usace.army.mil/software/hec-ras/hecras-download.html.
- User's Manual for WSPRO-A Computer Model for Water Surface Profile Computations, Publication No. FHWA-SA-98-080, FHWA, June 1998

(computer program and manual can be downloaded at the following website: http://www.fhwa.dot.gov/bridge/hydsoft.htm)

- HEC-2 Water Surface Profiles computer program, Version 4.6, May 1991, and User's Manual, developed by the USACE, Hydrologic Engineering Center, February 1991. HEC-2 is no longer under development by the USACE and has been superceded by HEC-RAS, which is preferred over HEC-2. HEC-2 should be used only under special circumstances. The HEC-2 computer program and user's manual can be downloaded at: http://www.hec.usace.army.mil/software/legacysoftware/legacysoftware.html
- Computer program for Water Surface Profiles, WSP2, National Engineering Handbook, Part 60, Chapter 31, U.S. Department of Agriculture, Soil Conservation Service, October 1993. The computer program and user's manual can be downloaded at: http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-other-models.html.

Although most channel design problems encountered in highway projects can be evaluated using any of the above computer applications, the WVDOH recommends the HEC-RAS computer program because it is the most advanced and most widely used. HEC-RAS is also capable of one-dimensional unsteady flow computations as well as channel modifications, floodway encroachments, uniform flow, sediment transport, bridge scour and stable channel design. HEC-RAS can also be used to process geo-spatial data using Geographic Information Systems (GIS) software. Computer programs other than HEC-RAS should be used only under special circumstances and upon approval by the WVDOH.

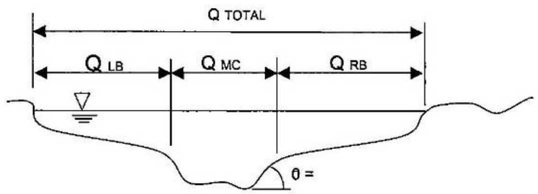
A discussion of computer programs for solving complex two-dimensional flow problems is outside the scope of this manual. If such situations are encountered, the designer should consult the WVDOH for guidance. Commonly used public domain 2-D flow models include, FESWMS-2DH, Flo2DH, and BRISTARS, developed by the FHWA (available at http://www.fhwa.dot.gov/bridge/hydsoft.htm), and TABS RMA2 developed by the USACE (available at http://chl.wes.army.mil/software/tabs/rma2.htp). Sediment routing calculations can be performed using FHWA's BRISTARS software or the USACE's HEC-6 computer program, "Scour and Deposition in Rivers and Reservoirs" (http://www.hec.usace.army.mil/software/legacysoftware/hec6/hec6-download.htm).

7.5 COMPUTATION FORM, TABLES AND CHARTS

Form 7-1
Riprap Lining Design Computation Form

Project: ______	Prepared By: ______ Date: ______	Upstream Channel Width: ______ft
Description: ______	Checked By: ______ Date: ______	Downstream Channel Width: ______ft
Station: ______ To Station: ______	Sheet ______ of ______	Channel Average Slope: ______ft/ft
		Natural Channel length: ______ft
		Description of bed material: ______

Definition Sketch:
(For major channels)



Soil Characteristics:

Q_{TOTAL} ______ D_{15} ______
Q_{MC} ______ D_{50} ______
Q_{LB} ______ D_{85} ______
Q_{RB} ______

MC = Main Channel
LB = Left Bank
RB = Right Bank

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Water Surface Elev. (feet)	Main Channel Flow Area (square feet)	Main Channel Average Velocity (feet/second)	Main Channel Average Depth (feet)	Bank angle θ (degrees)	Riprap angle of repose Ø Chart 7A-8	Bank angle Correction K_1 Chart 7A-7	Riprap size Chart 7A-5 (feet)	Stability Factor	Riprap Specific Gravity	Riprap size Correction Factor Chart 7A-6	Pier/Abutment Correction = 3.38 (If applicable)	Correction D_{50} =(8x11x12) (feet)	Notes

Size:
D_{50} ______ $2D_{50}$ ______ D_{100} ______ USE T = ______ feet

Table 7-3

Guide to Manning's Roughness Values (Uniform Flow)

Type Of Channel And Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1. Minor streams (top width at flood stage < 30 m)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones/weeds	0.030	0.035	0.040
3. Clean, winding, some pools/shoals	0.033	0.040	0.045
4. Same as above, but some weeds/stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large bolders	0.040	0.050	0.070
2. Flood Plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035

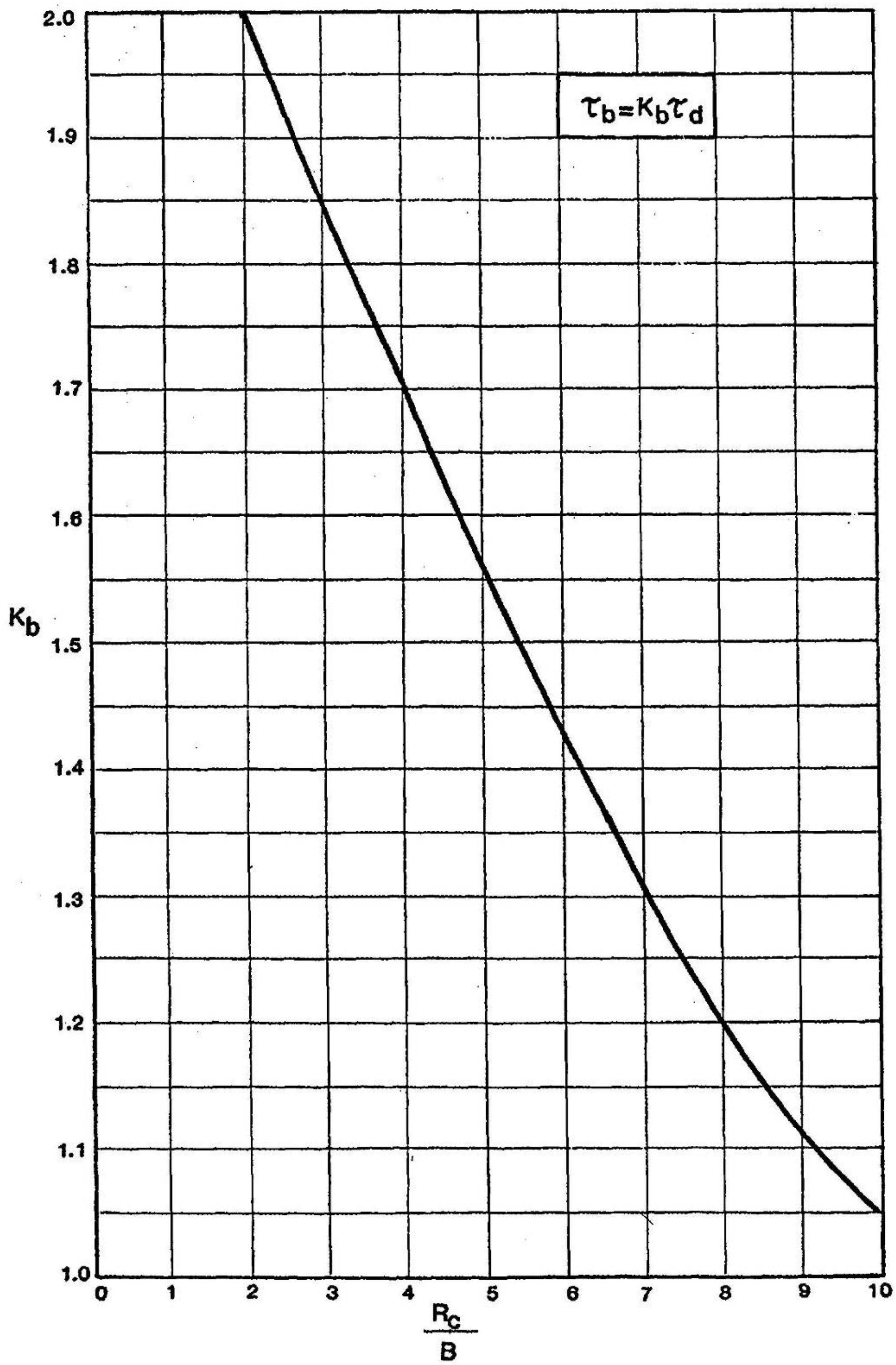
Table 7-3 (Continued)

Type Of Channel and Description	Minimum	Normal	Maximum
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense Willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of spouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Streams (top width at flood stage > 30 m). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	—	0.060
b. Irregular and rough section	0.035	—	0.100

Source: Chow, V.T.

Chart 7-1

K_b for Maximum Shear Stress in Channel Bends



Source: Design of Roadside Channels with Flexible Linings, HEC-15, FHWA, 1988

Chart 7-2

Riprap Size (D_{50}) Nomograph

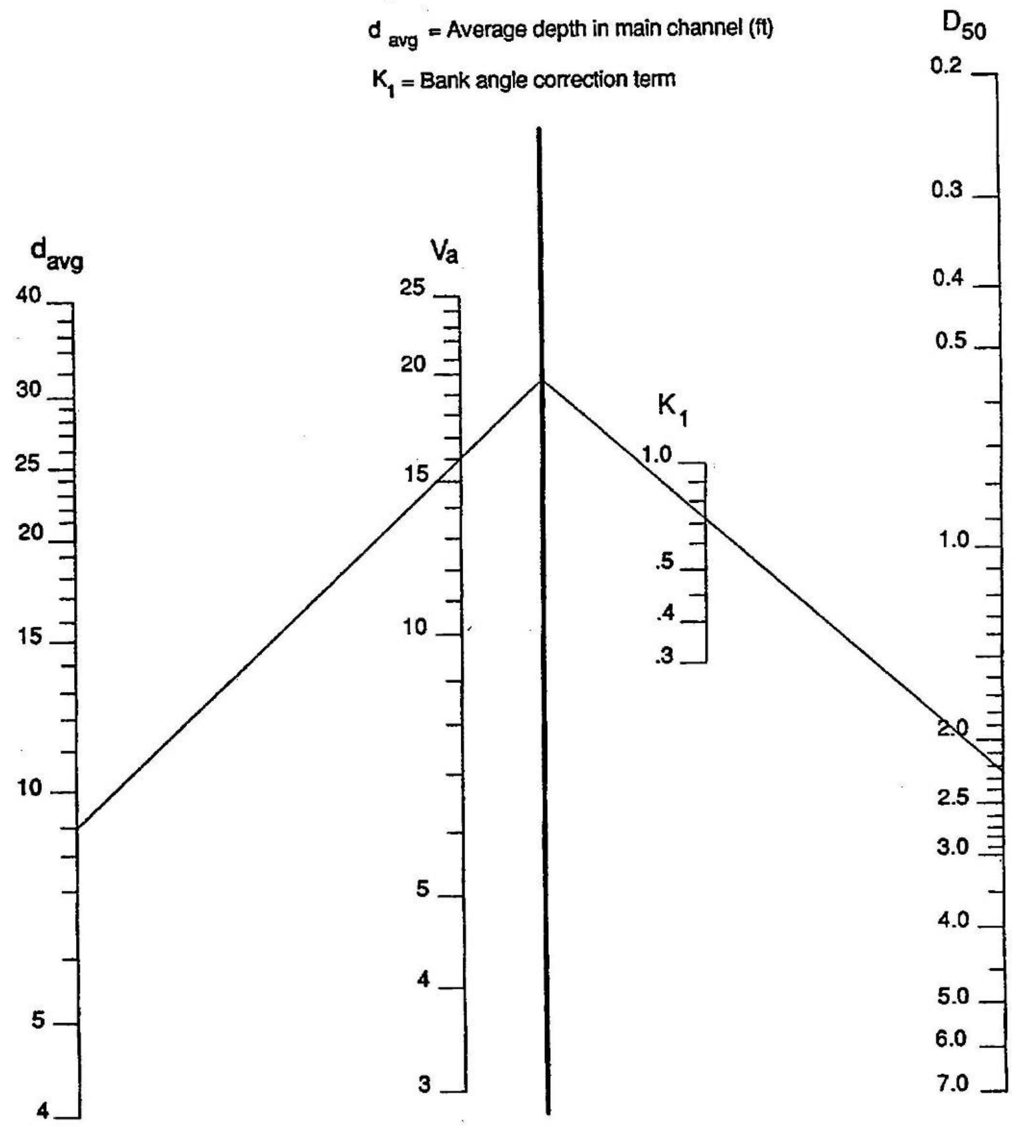
$$D_{50} = 0.001\, V_a^3 / (d_{avg}^{1/2} K_1^{3/2})$$

D_{50} = Median Riprap Size (ft.)

V_a = Average velocity in main channel (ft/sec)

d_{avg} = Average depth in main channel (ft)

K_1 = Bank angle correction term



Example

Given:
- V_a = 16 ft/sec
- d_{avg} = 9 ft
- K_1 = 0.72

Find:
- D_{50}

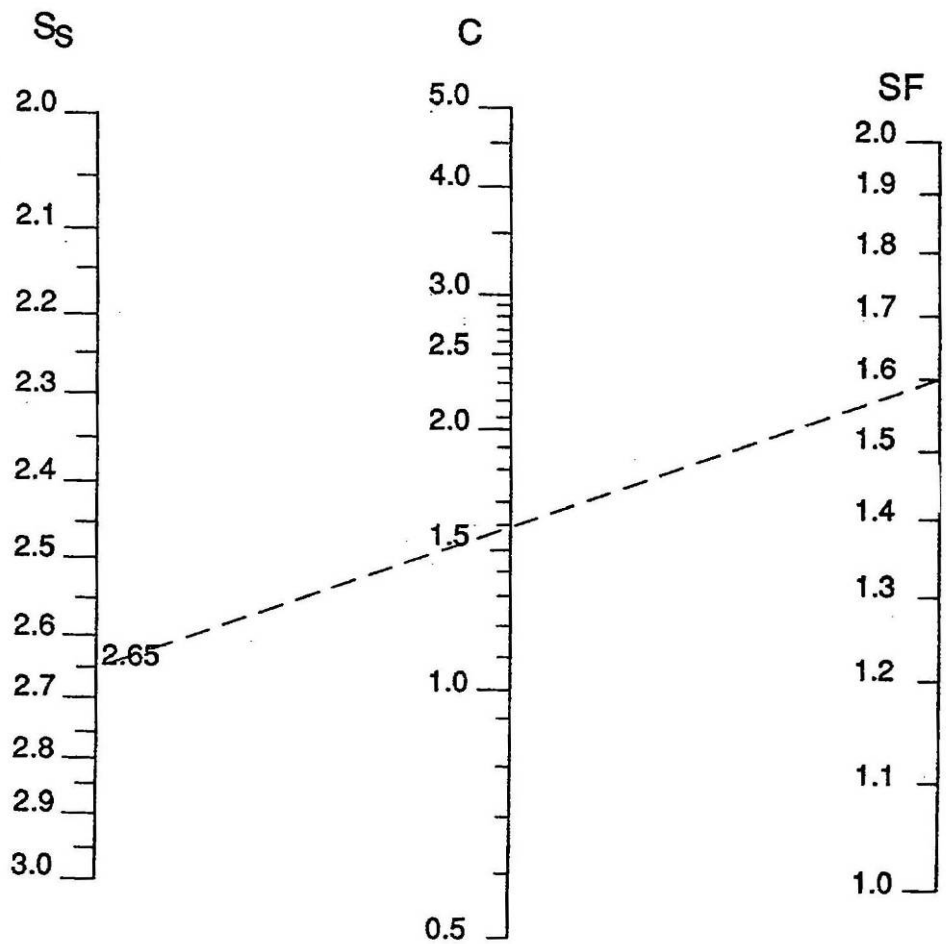
Solution:
- D_{50} = 2.25

Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

Chart 7-3
Nomograph for Riprap Size Correction Factor (C)

$$C=1.61SF^{1.5}/(S_S-1)^{1.5}$$

CORR= D_{50} CORRECTION FACTOR
SF = STABILITY FACTOR
S_S = SPECIFIC GRAVITY OF ROCK



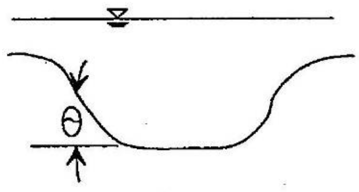
Example:

Given:	Find:	Solution:
S_S=2.75	C	C=1.59
SF= 1.60		

Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

Chart 7-4

Nomograph for Bank Angle Correction Factor (K_1)

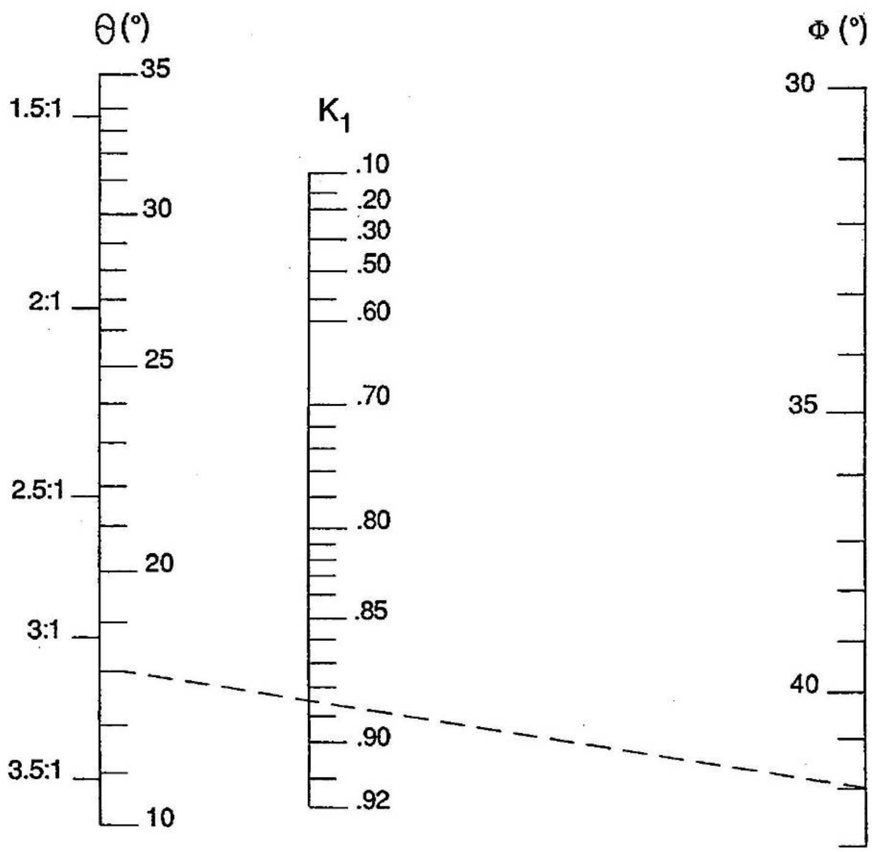


$$K_1 = \left[1 - \frac{\sin^2\theta}{\sin^2\Phi}\right]^{0.5}$$

θ = Bank angle with horizontal

Φ = Material angle of repose

See chart 7-5

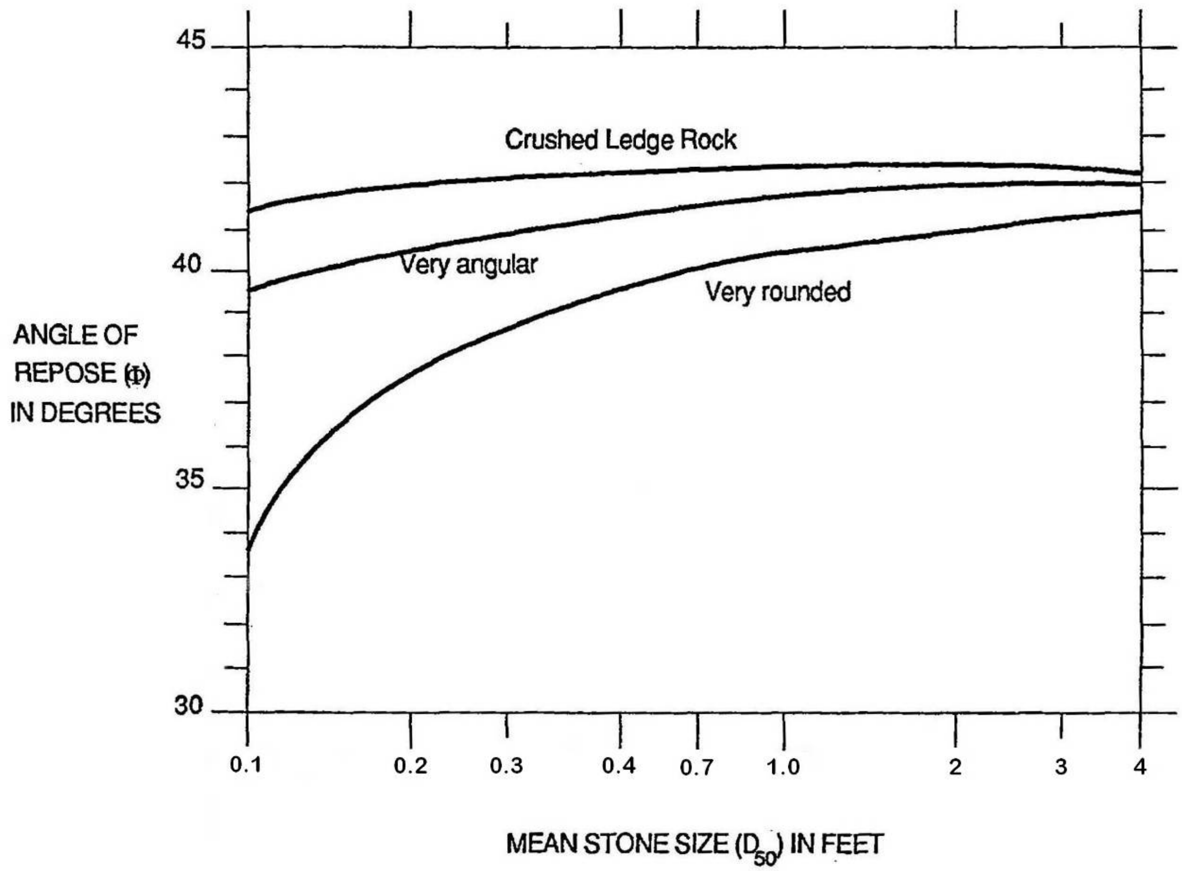


Example

Given:	Find:	Solution:
$\theta = 18°$	K_1	$\Phi = 42°$
Very Angular		$K_1 = 0.885$
$D_{50} = 1.5$ ft.		

Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

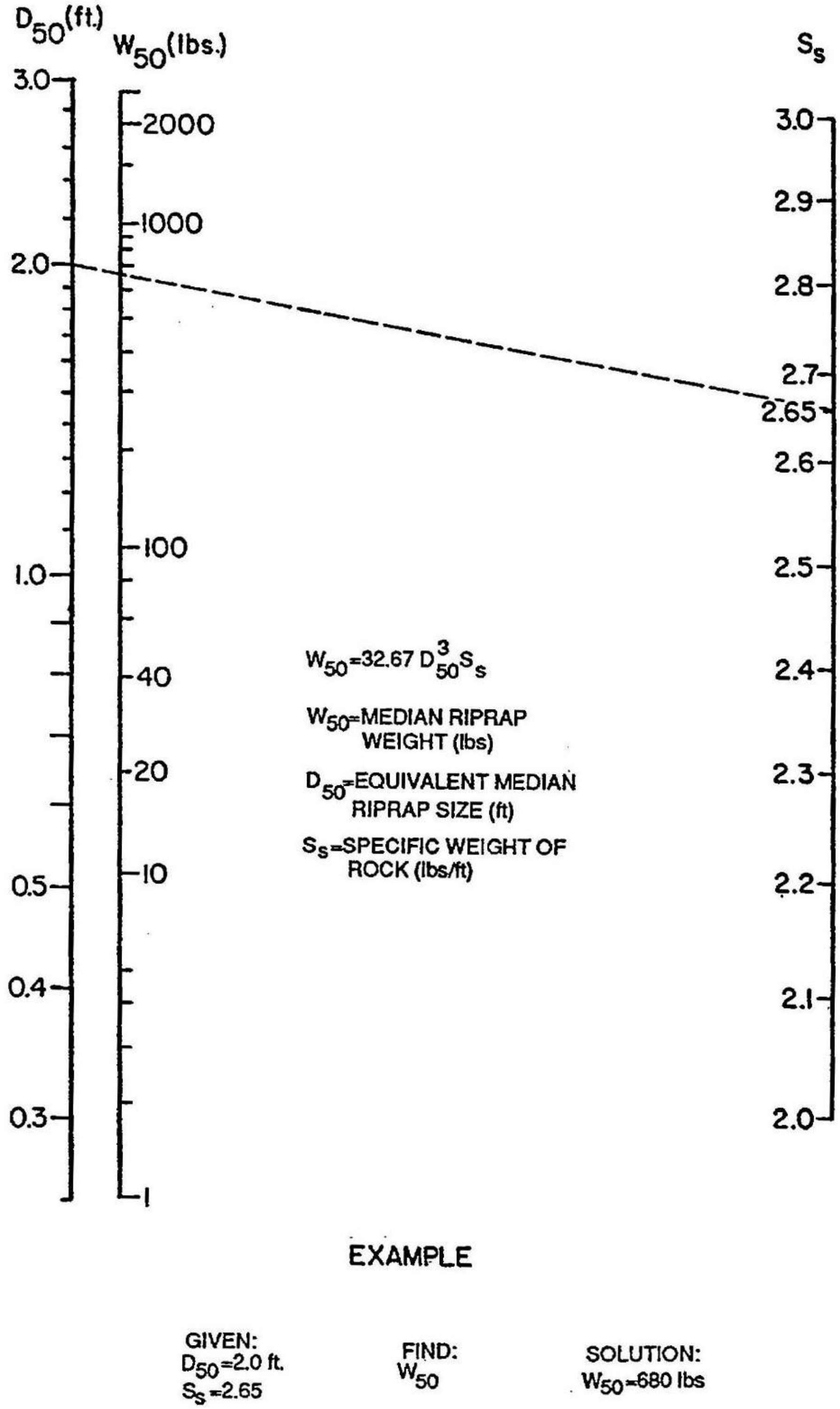
Chart 7-5
Riprap Angle of Repose (Φ)



Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

Chart 7-6

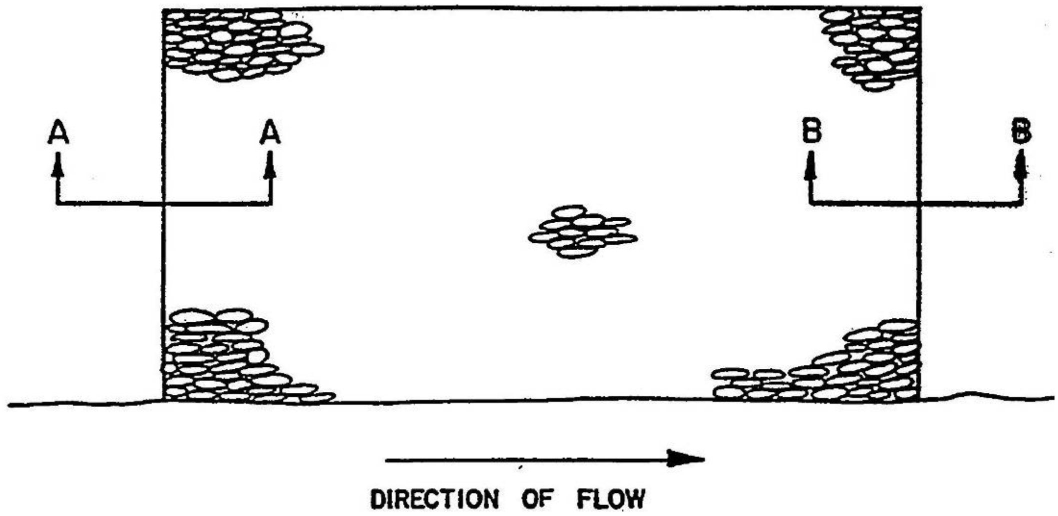
Nomograph to Convert Equivalent D_{50} (ft) to W_{50} (lbs)



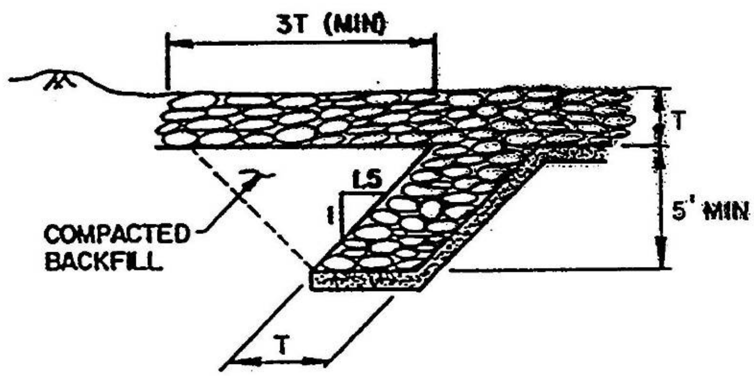
Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

Chart 7-7

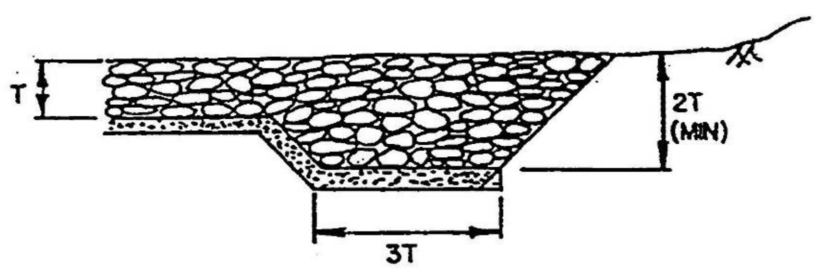
Typical Riprap Flank Details



PLAN VIEW



SECTION: A-A



SECTION: B-B

Source: Design of Riprap Revetment, HEC-11, FHWA, 1989

7.6 REFERENCES

FHWA Publications

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2. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains, Report No. FHWA-TS-84-204, April 1984
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5. Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18), 4th Edition, Publication No. FHWA NHI 01-001, May 2001
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7. Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance, Hydraulic Engineering Circular No. 23 (HEC-23), Second Edition, Publication No. FHWA NHI 01-003, March 2001

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8. Hydraulic Design of Flood Control Channels, Engineer Manual, EM 1110-2-1601, June 30, 1994
9. River Hydraulics, Engineer Manual, EM 1110-2-1416, October 15, 1993
10. Channel Stability Assessment for Flood Control Projects, Engineer Manual, EM 1110-2-1418, October 31, 1994

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11. Computing Degradation and Local Scour, Technical Guideline for the Bureau of Reclamation, January 1984.

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12. Roughness Characteristics of Natural Channels, Water Supply Paper 1849, Harry H. Barnes Jr., 1967. This publication can be downloaded at http://water.usgs.gov/pubs/wsp/wsp_1849/html/pdf.html.

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 8: CULVERTS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF FORMS

LIST OF CHARTS

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CHAPTER 8: CULVERTS

8.1 INTRODUCTION

Culverts are defined as hydraulic structures designed to convey surface water runoff under a highway embankment.

This chapter outlines the policies, criteria, and guidelines for the hydraulic design of highway culverts. The information presented in this chapter should enable the designer to select, plan and design conventional highway culverts.

This chapter does not cover all aspects of culvert design. The reference list at the end of this chapter includes publications by the FHWA that should be consulted for additional information regarding specialized aspects of culvert design. HDS-5 is the primary FHWA reference for culvert design.

8.2 GENERAL DESIGN POLICY

The designer shall use the following general policies as a guide to select, plan and design culverts placed beneath roadways and highways:

- Culverts shall be hydraulically designed;
- Culverts shall be located to present a minimum hazard to the public and the environment;
- Culverts shall be designed to be structurally stable and hydraulically efficient;
- Culverts shall be designed to consider construction and maintenance costs, risk of failure, risk of property damage, traffic safety and environmental considerations;
- The detail of documentation for each culvert site shall be commensurate with the risk and importance of the structure;
- Culverts located in floodplains mapped by the Federal Emergency Management Agency shall satisfy the requirements of the National Flood Insurance Program.

8.3 DESIGN CRITERIA

Culverts shall be designed with the following minimum design criteria to integrate hydraulics, economics, safety, environmental considerations, and maintenance.

8.3.1 Structure Type Selection

The choice between a culvert and a bridge at a given site shall be based on the following criteria:

Culverts are used:

- Where more economical than a bridge
- Where debris and ice are not significant
- Where bridges are not required to reduce backwater impacts
- Where environmentally acceptable
- Where floodway encroachments are not critical
- Where overtopping potential and damage due to overtopping is low

Bridges are used:

- Where culverts cannot be used
- Where it is more economical than a culvert
- To avoid floodway encroachments
- To satisfy land use requirements
- To reduce environmental impacts caused by a culvert
- To accommodate ice and large debris

8.3.2 Location, Alignment and Grade

Culverts shall be located in the existing channel in order to avoid major stream relocations beyond the roadway construction limits and to reduce environmental impacts. Where stream channel relocation is necessary, it shall be done without causing abrupt transitions of the stream at either end of the culvert. Consider the temporary diversion of the stream and constructability when locating a culvert.

Culverts shall be aligned with the direction of flow and with the natural grade of the stream. Improper selection of the alignment and grade can decrease hydraulic performance and increase sediment deposition, debris and scour.

On steep terrain, long culverts under high fills should be designed to follow existing stream alignments with both horizontal and vertical bends. This will reduce trench excavation and possibly reduce outlet velocity.

8.3.3 Minimum Size

The minimum size of a culvert shall be determined based on the peak discharge of the design flood from the contributing drainage basin. In some instances culverts may be oversized to limit upstream inundation due to headwater. Culverts shall be sized to accommodate debris and avoid maintenance problems. The minimum culvert diameters based on the roadway classification shall be as shown in Table 8-1.

It should be noted that storm drain and median drain pipes, which are not classified as culverts, should be at least 18 inches in diameter and inlet drain pipes should be at least 12 inches in diameter.

Table 8-1

Minimum Pipe Diameters

Multi-lane Highways	24 inches
Two Lane Highways	18 inches
Driveways	15 inches

8.3.4 STORM FREQUENCY

Highway culverts shall be designed for the minimum flood frequencies shown in Table 4-2 (refer to Chapter 4, Section 4.3.1) and review against a check storm frequency (refer to Chapter 4, Section 4.3.2).

Temporary culverts used to maintain drainage during construction should be sized based on the expected duration of the project and shall be designed for a storm recurrence interval of no less than the 2-year event.

8.3.5 HYDROLOGY

A constant peak discharge shall be assumed for most culvert designs in order to size the structure conservatively. Culverts shall be designed for the peak flow calculated at the inlet end of the culvert. The design peak flow shall be determined by the methods outlined in Chapter 4.

Hydrograph and storage routing methods shall not be used for designing culverts unless unusual circumstances exist. An example would be the use of a smaller culvert for the purposes of inducing flow detention (increased headwater) behind it.

8.3.6 MAXIMUM ALLOWABLE HEADWATER

Allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design storm event. It will be based on the following requirements:

- Non-damaging to upstream property
- Below the roadway subgrade
- HW/D no greater than 1.5
- Equal to the elevation where flow diverts around the culvert

- For replacement culverts, no greater than the existing condition
- In compliance with FEMA and local floodplain regulations

Culverts shall be evaluated based on the check storm criteria in Section 4.3.2 of Chapter 4. Major culvert installations where the headwater may affect insurable structures, developable property, or that are located within a FEMA designated flood zone may need to be analyzed using a backwater analysis program with the Division's approval.

8.3.7 TAILWATER RELATIONSHIP

Tailwater is defined as the depth of water downstream of the culvert. It is measured from the outlet invert and is an important factor in determining the culvert capacity and headwater elevation under outlet control conditions. Tailwater may be caused by the hydraulic resistance (roughness) of the downstream channel or by obstructions such as a low-water crossing, another culvert, or a reservoir. Tailwater in a tributary may also be controlled by backwater from a larger stream. For culverts operating in outlet control, tailwater depths shall be determined for a range of discharges. These are obtained from normal depth calculations, back water calculations for a receiving stream or flood insurance study data.

The tailwater relationship for a culvert located near the confluence with another channel or large water body shall be determined by a joint probability analysis (See Table 5-10 Section 5.3.6.5 of Chapter 5, Storm Drainage Systems). If the design storm events occur concurrently (statistically dependent with coincident peaks), the high water elevation that has the same frequency as the receiving channel or water body shall be used. If the events are statistically independent, the joint probability of the flood magnitudes shall be evaluated and a likely combination resulting in a greater tailwater depth shall be used. Refer to Section 8.4.10 for more details.

8.3.8 END TREATMENTS (INLET OR OUTLET)

End treatments for culverts larger than 36 inches shall consist of headwalls or wingwalls at both ends as shown in the WVDOH Standard Drainage Details. End treatments on all culverts shall consider buoyancy protection. Metal end sections or safety end sections for larger pipes may be used if approved by the engineer.

End treatments shall be located outside of the clear zone on high-speed roads in order to eliminate the possible hazard to an out of control vehicle. For culverts that are skewed to the overlying roadway, headwalls shall be placed perpendicular to the culvert rather than parallel to the roadway. Refer to Section 8.4.12 for more details.

8.3.9 MAXIMUM OUTLET VELOCITY AND SCOUR PROTECTION

The maximum velocity at the culvert outlet shall be determined by evaluating a range of discharges up to the design discharge. The need for scour protection shall be based on the maximum outlet velocity. Protection shall be provided by creating a stable discharge area that reduces the velocity to the pre culvert installation velocity. In other words, the culvert outlet velocity should be equal to or less than the velocity in the outlet channel before the culvert installation. Allowable velocities of the stream bed material shall be used as a guide to determine the need for scour protection. Refer to Table 8-4 for allowable velocities of streambed material.

Outlet scour protection shall consist of channel stabilization with rock lining or equivalent material. On the plans, rock lining shall be designated as “Dumped Rock Gutter” or “Select Embankment” as shown in the Standard Details. The use of rock-lined scour basins shall be based on site-specific conditions. Rock-lined scour basins and energy dissipators shall be designed in accordance with the guidelines provided in the latest edition of HEC-14, published by the FHWA.

8.3.10 MINIMUM VELOCITY

The minimum velocity in the culvert shall be adequate to prevent sedimentation at low flow rates. Culverts shall be designed for a minimum velocity of 2 feet per second when the culvert is flowing at a depth equal to 15% of the diameter of the culvert.

8.3.11 DEBRIS CONTROL

Culverts at locations where excessive sedimentation and debris problems are expected (such as steep streams that are transporting heavy bed load) shall be designed to accommodate debris or proper provisions shall be made for debris maintenance. Access to the culvert for maintenance, personnel, and equipment shall be provided. Debris control shall be designed using the guidelines provided in the latest edition of HEC-9, published by the FHWA.

8.3.12 PIPE MATERIAL SELECTION

Culvert material selection shall be based on the latest version of Design Directive-503 (DD-503), Design of Alternate Pipe Materials, published by the West Virginia Division of Highways.

Culvert material gage and corrugation shall be specified based on the latest version of Design Directive-502 (DD-502), Maximum Fill Height Tables for Various Types of Pipe, published by the West Virginia Division of Highways.

8.3.13 MULTIPLE BARRELS

When considering multiple barrel culverts, stream stability and sediment transport must be evaluated. Where debris and sediment transport are a concerns, a single cell culvert is recommended.

Multiple barrel culverts shall be avoided:

- Where the approach flow is supercritical with high velocity. Such locations shall require a single barrel or special inlet treatment to avoid hydraulic jump effects;
- Where fish passage is required, except where special treatment is provided to ensure adequate low flow, such as lowering one barrel (see Section 8.4.15);
- Where a high potential exists for debris problems and clogging of the culvert;
- Where a meander bend is present immediately upstream.

8.3.14 ENVIRONMENTAL CONSIDERATIONS

Culvert locations shall be selected to minimize impacts to the streams and wetlands whenever practical. Consideration shall be given to constructing culverts in the “dry” by using a temporary diversion channel. Aquatic life movements may be accommodated when required by law or when it is beneficial as mitigation for stream impacts (see Section 8.4.17). If multiple barrels are used, special treatment shall be provided to ensure adequate low flow, such as lowering one barrel (see Section 8.4.15).

8.4 DESIGN CONCEPTS AND GUIDELINES

8.4.1 CULVERT TYPES

Culverts are constructed of materials such as concrete, reinforced concrete, corrugated steel, corrugated aluminum, and high density polyethylene plastic. Common culvert shapes include circular, box, elliptical, arch, and pipe arch. The material and shape are selected based on factors such as roadway profile, channel characteristics, hydraulic performance, strength, construction methods, and corrosion and abrasion resistance.

Conventional culverts with uniform barrel cross-sections throughout their length are considered in this chapter. Culvert inlets and outlets may consist of the culvert barrel projecting from the roadway fill, mitered to the embankment slope, or with end treatments such as headwalls, wing-walls with apron slabs, or standard end sections of concrete or metal.

8.4.2 CULVERT HYDRAULICS

An exact theoretical analysis of culvert hydraulics is extremely complex because the flow is usually non-uniform with regions of both gradually varying and rapidly varying flow. An exact analysis involves determining change in flow type for various flows and tailwater elevations, backwater and drawdown calculations, energy and momentum balance, application of hydraulic model studies, and determination of hydraulic jump locations.

8.4.3 CONTROL SECTION AND MINIMUM PERFORMANCE

The procedures in HDS-5 were developed to simplify culvert hydraulic calculations and systematically analyze culvert flow on the basis of a “control section”. A control section is a location where there is a unique relationship between the flow rate and the upstream water surface elevation. Many different flow conditions exist over time but at a given time the flow is either governed by the inlet geometry (inlet control) or a combination of the inlet geometry, the culvert barrel characteristics and the tailwater elevation (outlet control). Control may oscillate from inlet to outlet; however, the design method is based on a “minimum performance”. That is, while the culvert may operate more efficiently at times (i.e., more flow for a given headwater level), it will never operate at a lower level of performance than the calculated minimum.

The HDS-5 design method uses equations, charts, and nomographs that were developed from numerous hydraulic laboratory tests and theoretical calculations. Due to the error introduced in the test data as a result of scatter, it should be assumed that culvert sizes calculated with this method are accurate to within plus or minus 10 percent, (HDS-5, Sept 2001, Chapter III, Section A, 3rd paragraph, page 23) in terms of head.

8.4.4 INLET AND OUTLET CONTROL

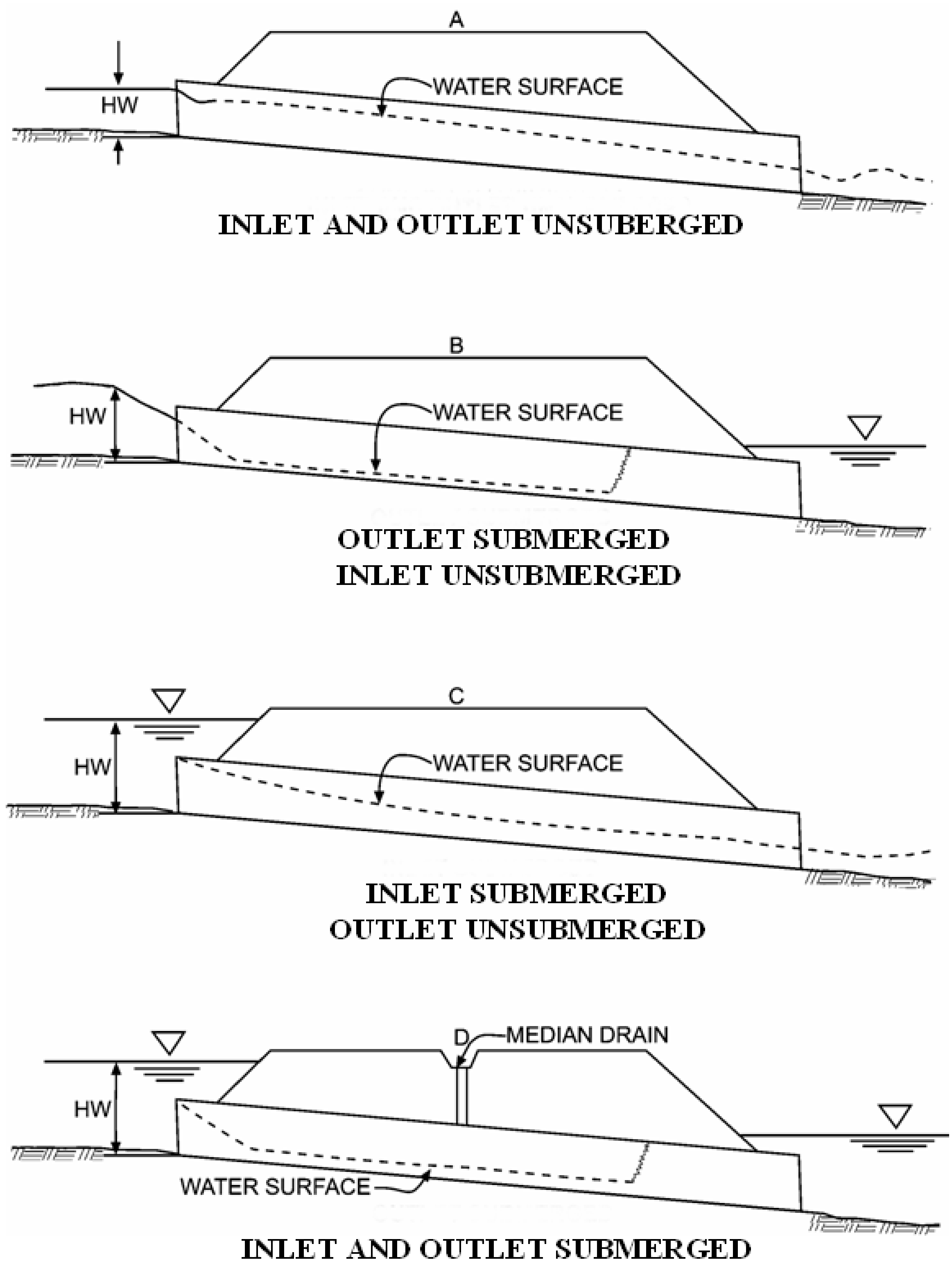
A culvert flowing in inlet control has shallow, high-velocity, supercritical flow with the control section located at the upstream end of the barrel. Inlet control is influenced by the headwater depth and inlet area, edge configuration and shape. Figure 8–1 shows several examples of inlet control flow with either a submerged or an unsubmerged inlet. The submerged inlet operates essentially as an orifice and an un-submerged inlet operates as a weir.

The inlet edge configuration is a major factor of inlet control performance and it can be modified to improve performance. Modified inlets with beveled edges can reduce the flow contraction. This may decrease the headwater for a given barrel size or allow a smaller pipe for a given headwater.

A culvert flowing in outlet control will have a deep, low velocity subcritical flow with the control section located at the downstream end of the culvert. The factors influencing outlet control are barrel roughness, tailwater elevation, headwater, edge configuration, barrel area, shape, length and slope. The greater depth of the tailwater depth or downstream channel depth is the control at the outlet, whichever is greater. Figure 8–2 shows several examples of outlet control flow.

Figure 8–1

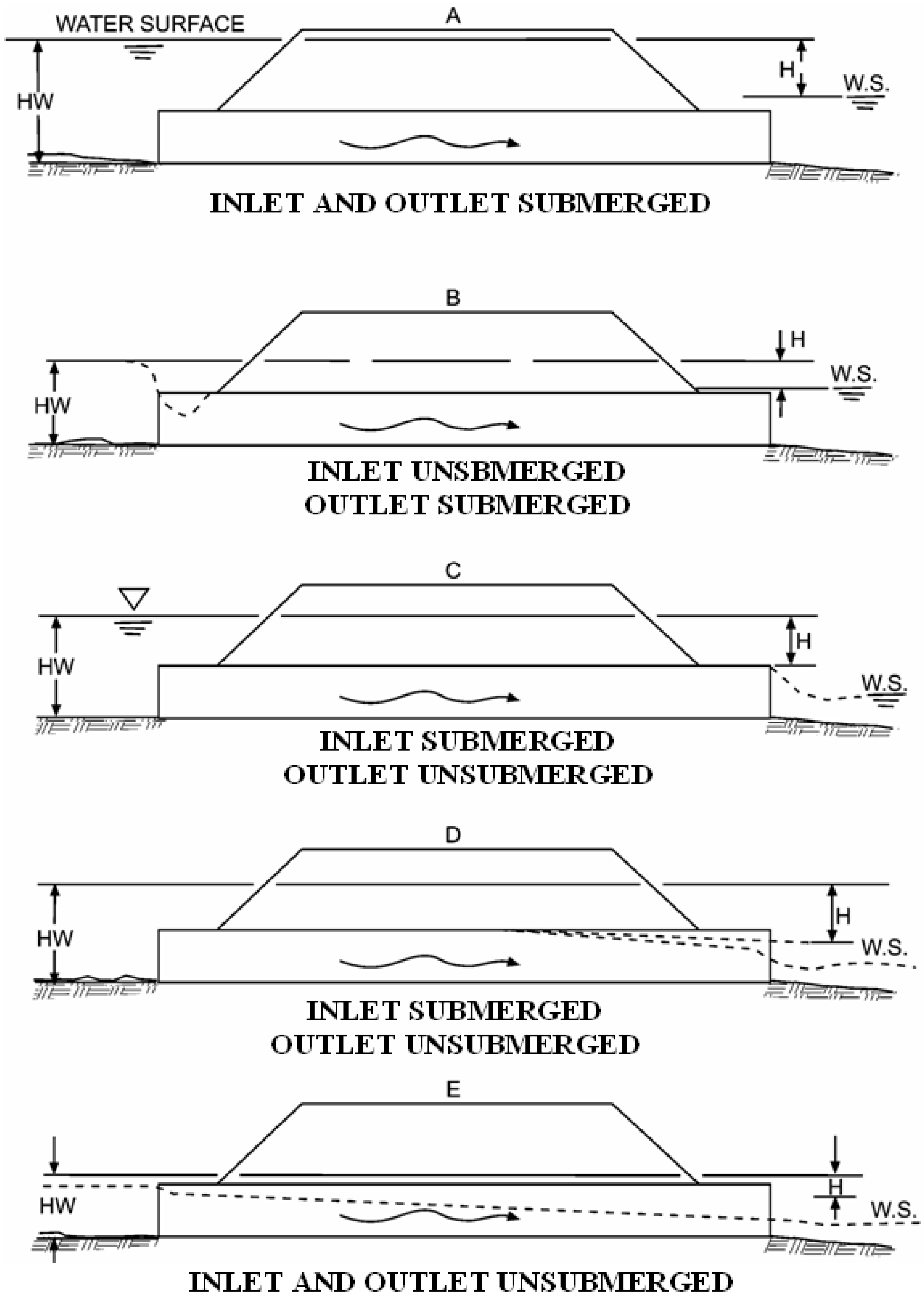
Types of Inlet Control



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Figure 8–2

Types of Outlet Control



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

8.4.5 INLET AND OUTLET CONTROL EQUATIONS

The two basic conditions of inlet control depend upon whether the inlet end of the culvert is submerged by the upstream headwater. If the inlet is not submerged, the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice.

The unsubmerged and submerged inlet control headwater design equations are provided below. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at the critical depth, adjusted with two correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply and is the only documented form of equation for some of the inlet control nomographs. A constant slope value of 2 percent was used for the development of the nomographs. This is due to the small effect of the slope and the conservatively high resultant headwater for sites with slopes exceeding 2 percent.

UNSUBMERGED

Form (1) $$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M - 0.5S$$

Note: For mitered inlets the slope correction factor is $+0.7S^2$ instead of $-0.5S^2$ as shown in the equation below.

The following equations are applicable up to $\frac{Q}{AD^{0.5}} = 3.5$.

Form (1) $$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M + 0.7S$$

Form (2) $$\frac{HW_i}{D} = K\left[\frac{K_u Q}{AD^{0.5}}\right]^M$$

SUBMERGED

$$\frac{HW_i}{D} = c\left[\frac{K_u Q}{AD^{0.5}}\right]^2 + Y - 0.5S$$

The above equation applies above about $\frac{Q}{AD^{0.5}} = 4.0$

HW_i= Headwater depth above inlet control section invert (ft)

D = Interior height of culvert barrel (ft)

H_c = Specific head at critical depth ($d_c+V_c^2/2g$) (ft)

Q = Discharge (ft^3/s)

A = Cross-sectional area of the barrel (ft^2)

K,M,c,Y Constants from Table 8-2

K_u = 1.0 for english units

S = Culvert barrel slope (ft/ft) = 0.02

Outlet control flow conditions are calculated based on energy balance. In its most basic form, the head loss H_L or the total energy required to pass a given quantity of water through a culvert flowing under outlet control with the barrel flowing full throughout its length, is made up of three major parts: an entrance loss (H_e), the friction loss through the culvert (H_f), and the exit loss (H_o).

$$H_L = H_e + H_f + H_o$$

The culvert barrel velocity is calculated as: $V = \frac{Q}{A}$

Where: V is the average velocity in the culvert barrel (ft/s)

Q is the flow rate (ft^3/s)

A is the full cross-sectional area of flow (ft^2)

The velocity head is expressed as: $H_v = \frac{V^2}{2g}$

Where: g = acceleration due to gravity (32.2 ft/s^2)

The entrance loss (H_e) is expressed as a coefficient times the velocity head:

$$H_e = K_e\left(\frac{V^2}{2g}\right)$$

Table 8-2

Constants for Inlet Control Design Equations

CONDUIT	CHART	INLET EDGE TYPE	UNSUBMERGED		SUBMERGED	
			K	M	c	Y
CORRUGATED METAL PIPE	Chart 8-1	HEADWALL	0.0078	2.00	0.0379	0.69
		MITERED TO SLOPE	0.0210	1.33	0.0463	0.75
		PROJECTING	0.0340	1.50	0.0553	0.54
CONCRETE PIPE	Chart 8-4	SQUARE EDGE WITH HEADWALL	0.0098	2.00	0.0398	0.67
		GROOVE END WITH HEADWALL	0.0018	2.00	0.0292	0.74
		GROOVE END PROJECTING	0.0045	2.00	0.0317	0.69
HIGH DENSITY POLYETHYLENE PLASTIC PIPE	Chart 8-6	HEADWALL	0.0078	2.00	0.0379	0.69
		MITERED TO SLOPE	0.0210	1.33	0.0463	0.75
		PROJECTING	0.0340	1.50	0.0553	0.54
CONCRETE BOX CULVERTS	Chart 8-14	30° TO 75° WINGWALL FLARE	0.026	1.00	0.0347	0.81
		90° AND 15° WINGWALL FLARE	0.061	0.75	0.0400	0.80
		0° FLARE OR SIDE EXTENSION	0.061	0.75	0.0423	0.82
CORRUGATED METAL PIPE ARCH STANDARD SIZES	Chart 8-19	90° HEADWALL	0.0083	2.0	0.0379	0.69
		MITERED TO SLOPE	0.0300	1.0	0.0463	0.75
		PROJECTING	0.0340	1.5	0.0496	0.57
STRUCTURAL PLATE CORR. METAL PIPE ARCH 18 INCH CORNER RADIUS	Chart 8-21	90° HEADWALL	0.0083	2.0	0.0379	0.69
		MITERED TO SLOPE	0.0300	1.0	0.0463	0.75
		PROJECTING	0.0340	1.5	0.0496	0.57
HORIZONTAL ELLIPTICAL CONCRETE PIPE	Chart 8-32	SQUARE EDGE WITH HEADWALL	0.0100	2.0	0.0398	0.67
		GROOVE END WITH HEADWALL	0.0018	2.5	0.0292	0.74
		GROOVE END PROJECTING	0.0045	2.0	0.0317	0.69

Table 8-3

Entrance Loss Coefficients K_e

Outlet Control, Full or Partly Full Entrance Head Loss

$$H_e = K_e\left[\frac{V^2}{2g}\right]$$

Type of Structure and Design of 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 = D/12	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^0 or 45^0 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	0.5
Beveled edges, 33.7^0 or 45^0 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 D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30^0 to 75^0 to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10^0 to 25^0 to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End Sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

The friction loss in the barrel (H_f) can be expressed as:

$$H_f = \left[\frac{29\, n^2 L}{R^{1.33}} \right] \frac{V^2}{2g}$$

Where: n = Manning's Roughness Coefficient

L = Length of the culvert barrel (ft)

R = Hydraulic radius of the full culvert barrel (ft) = A / P

A = Cross-sectional area of the barrel (ft^2)

P = Perimeter of the barrel (ft)

V = Velocity in the barrel (ft/s)

The standard corrugated metal pipe flowing full nomograph was created using an n value of 0.024, however according to Design Directive 503 a different n value shall be used. Corrugated metal pipes less than 24 inches in diameter shall have an n value of 0.015. Corrugated metal pipes greater than 24 inches in diameter shall have an n value of 0.023 or greater (see Table 503-2 in Design Directive 503). This rule applies because most manufactured corrugated pipe has helical corrugations, not annular corrugations.

The exit loss (H_o) for sudden expansion such as an endwall is:

$$H_o = 1.0 \left[\frac{V^2}{2g} - \frac{V_d^{\,2}}{2g} \right]$$

Where: V_d is the channel velocity downstream of the culvert (ft/s)

Since the downstream velocity is usually neglected, the exit loss becomes equal to the full flow velocity head in the culvert barrel.

$$H_o = H_v = \frac{V^2}{2g}$$

Inserting the above relationships for entrance loss, friction loss, and exit loss into energy or head loss equation, the following equation for total head loss is obtained:

$$H_L = \left[1 + k_e + \frac{29\; n^2\; L}{R^{1.33}} \right] \frac{V^2}{2\; g}$$

The outlet control headwater design equation is:

$$HW_o = H_L + h_o - L\,S_o$$

See Section 8.5 for information about the use of the equation.

Minor losses such as bend losses (H_b), junction losses (H_j), and losses at grates (H_g) should be included in energy or head loss equation if appropriate. See Chapter 5 section 5.3.6.8 for information on minor losses.

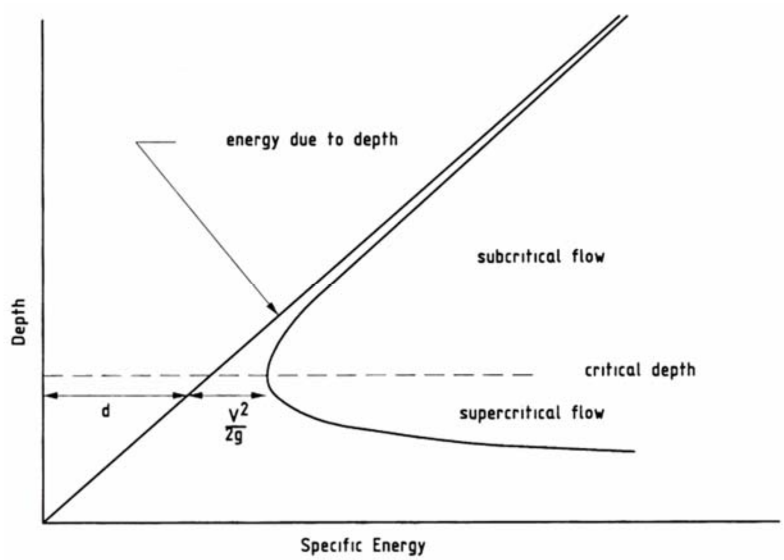
8.4.5.1 KEY PARAMETERS FOR CULVERT FLOW

Specific Energy

Specific energy (E) is the energy or head relative to the channel bottom. For a mild sloped channel with uniform flow (velocity and depth remain constant) the specific energy is defined as the depth plus the velocity head. If you look at the graph for depth vs. specific energy there is one depth at which the specific energy is at a minimum. This depth is the critical depth for the amount of flow or discharge.

$$E = \frac{V^2}{2g} + d$$

$$Q = V\,A \quad or \quad V = \frac{Q}{A}$$



The continuity equation transforms the specific energy equation in terms of depth, flow, and flow area.

$$E = \frac{Q^2}{2gA^2} + d$$

(see Section 8.4.5 and this entire section for term explanations)

Where: d = flow depth

It is important to define and distinguish between three important flow depths and how they pertain to a hydraulic pipe design.

Normal Depth

Normal depth is defined as the depth of uniform, steady flow under a constant discharge. In a uniform flow regime the losses due to boundary friction are balanced by the force of gravity in the direction of the flow. In other words, friction and gravity forces in the direction of flow are equal but act in opposite directions. This creates a hydraulic condition where the discharge, cross-sectional area, and velocity are constant throughout the length of the channel or pipe. The slope of the pipe invert, the slope of the water surface, and the slope of the energy grade line are equal and parallel to each other in this hydraulic condition.

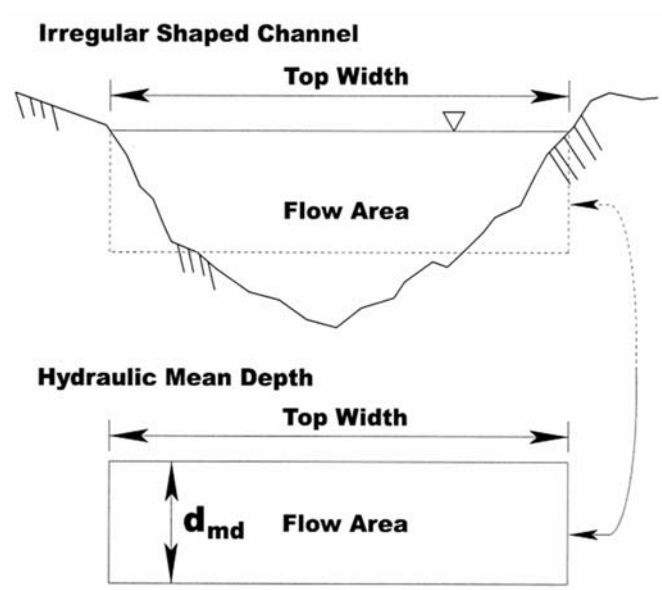
Normal depth is a function of discharge, size of channel, shape of channel, slope of channel, and frictional resistance to flow. It can be calculated using the familiar Manning's Equation.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad Q = VA = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

Hydraulic Mean Depth

The hydraulic mean depth is defined as the area of the flow cross section divided by the water surface top width. It is a method of characterizing an irregular shaped channel in terms of a rectangular shaped channel.

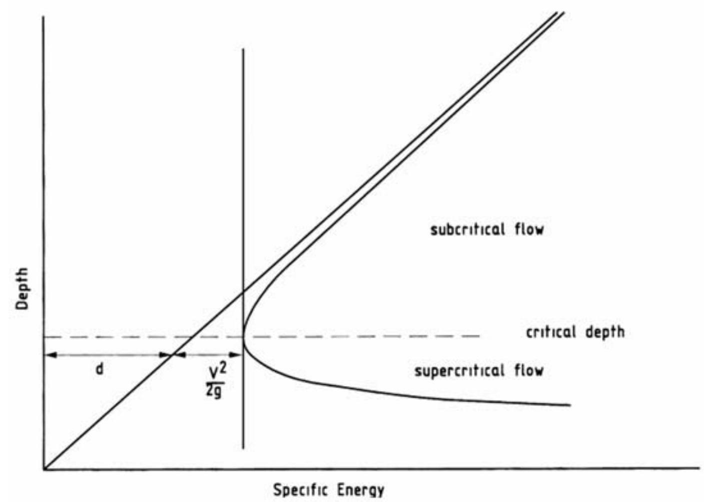
Where: T = flow top width



$$d_{md} = \frac{A}{T}$$

Critical Depth

The critical depth is defined as the depth at the point of minimum specific energy for a constant discharge. In most cases the occurrence of critical flow determines the location of the control point within that flow. The equation for determining critical depth is reached by taking the 1st derivative of the energy equation with respect to depth and setting it equal to zero (finding the point of zero slope w.r.t. the y axis).

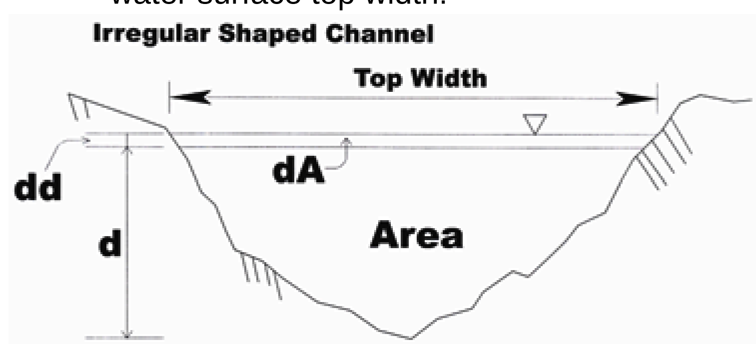


$$E = \frac{Q^2}{2gA^2} + d$$

$$E = \frac{Q^2}{2g}\frac{1}{A^2} + d$$

$$\frac{dE}{dd} = \frac{Q^2}{2g}\;\frac{-2}{A^3}\;\frac{dA}{dd} + 1$$

If we take small enough slices of our channel, the change in area with respect to the change in depth is equal to the water surface top width:



$$\frac{dA}{dd} = T$$

$$\frac{dE}{dd} = 1 - \frac{Q^2}{g}\;\frac{T}{A^3}$$

$$-\frac{Q^2}{g}\;\frac{T}{A^3} = -1$$

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

Depth of flow determines the flow area (A) and the flow top width (T). To solve for critical depth, find the flow (Q) and its corresponding depth (d) that create the equality in this equation.

This equation applies to all sizes and shapes of pipe and is the source for the critical depth charts in this chapter. The calculation for determining the critical depth curves end at 94% of the diameter for the case of circular pipes since this depth gives the maximum amount of discharge. The depth yielding the maximum amount of discharge will vary for arch pipes due to the many differences in cross sectional geometry. The same limit was used for the calculation of the critical depth curves for arch pipes to preserve consistency. Determining the critical depth beyond this 94% limit shall be taken from a "sketched" continuation of the calculated curve up to the diameter or rise of the pipe.

Critical Slope

Every culvert flow rate has a critical slope that corresponds to the critical depth.

$$S_C = \frac{14.56\, n^2\, d_{md}}{R^{\frac{4}{3}}}$$

The value of the critical slope can be compared against the slope of the culvert invert to determine the state of flow (see Chapter 5 Section 5.3.6.9). If S_c is greater than the slope of the culvert invert then the flow is subcritical and the control section is the outlet. If S_c is equal to the slope of the culvert invert then the flow is critical inside of the culvert and the control section is at the inlet.

Froude Number

When the flow is at critical depth, the specific energy is at a minimum and the Froude Number is equal to one. A detailed discussion of the Froude Number is provided in Chapter 5, Section 5.3.6.9. The derivation is provided here.

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad Q = V\,A \qquad \frac{V^2 A^2}{g} = \frac{A^3}{T}$$

$$\frac{V^2}{g} = d_{md}$$

The Froude Number: $\dfrac{V^2}{g\, d_{md}} = 1$ or $\dfrac{V}{\sqrt{g\, d_{md}}} = 1$

8.4.5.2 HIGH DENSITY POLYETHYLENE PLASTIC PIPE

The inlet and outlet control equations and their corresponding nomographs do not address the subject of designing a culvert using High density polyethylene plastic pipe (HDPEPP). HDPEPP has a smooth interior with a corrugated exterior. It was determined in the development of design directive 503; Design of Alternate Pipe Materials, that HDPEPP should be treated as a corrugated metal pipe in an inlet control situation. The constants K and M in an unsubmerged situation and c and Y in a submerged situation for circular corrugated metal pipe shall apply for HDPEPP in determining inlet control. This application shall be used for the headwall, mitered to slope, and projecting inlet edge description.

In an outlet control situation the entrance loss coefficient K_e shall be the same as that for a concrete pipe. This application shall be valid for projecting from fill and headwall and wingwalls for a square or cut end, mitered to conform to fill slope, and an end section conforming to fill slope.

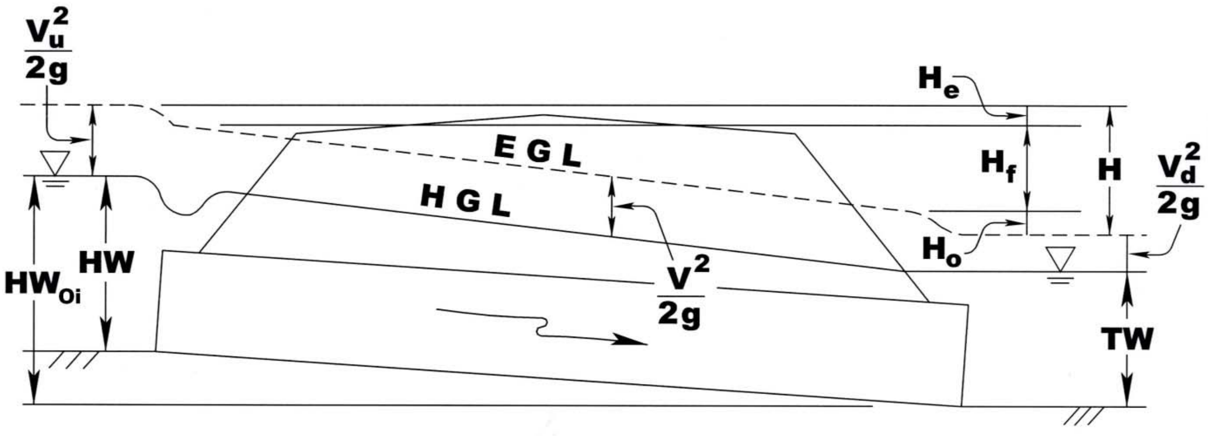
The roughness of the interior of the pipe varies with the type of backfill. When the non-compacted cementitious material, referred to as controlled low-strength material (see DOH specification 219) is used as backfill (type F trench see WVDOH Typical Sections and Related Details page 54) the interior of the pipe remains smooth and the Mannings roughness coefficient is 0.013. When any other type of backfill is used the exterior corrugations tend to protrude into the interior and the roughness coefficient is 0.015. Since the nomographs for total head loss (H_L) use different roughness coefficients, only the equations on the nomographs can be used to determine total head loss for HDPEPP pipe in determining an outlet control headwater.

8.4.6 Full Flow Energy and Hydraulic Grade lines

Figure 8–3 shows the energy grade line (EGL) and the hydraulic grade line (HGL) for full flow in a culvert barrel. The EGL represents the total energy at any point along the culvert barrel. The HGL is the water surface and the depth to which water would rise in vertical tubes connected to the sides of the barrel. The headwater (HW) and tailwater (TW) conditions as well as the entrance (H_e), friction (H_f), and exit (H_o) losses are also shown. HW is the depth from the inlet invert to the hydraulic grade line and HW_{oi} is the headwater depth above the outlet invert. V_u is the approach velocity and V_d is the downstream velocity.

Figure 8–3

Full flow energy and hydraulic grade lines



8.4.7 Roadway Overtopping

Roadway overtopping begins when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve resulting in flow that is similar to flow over a broad-crested weir. This overtopping flow is calculated using the weir equation:

$$Q = k\ C_d\ L\ H_w^{\ 1.5}$$

Where Q = Overtopping flow rate (cubic feet per second)

C_d = Overtopping discharge coefficient (weir coefficient)

k = over-embankment flow adjustment factor

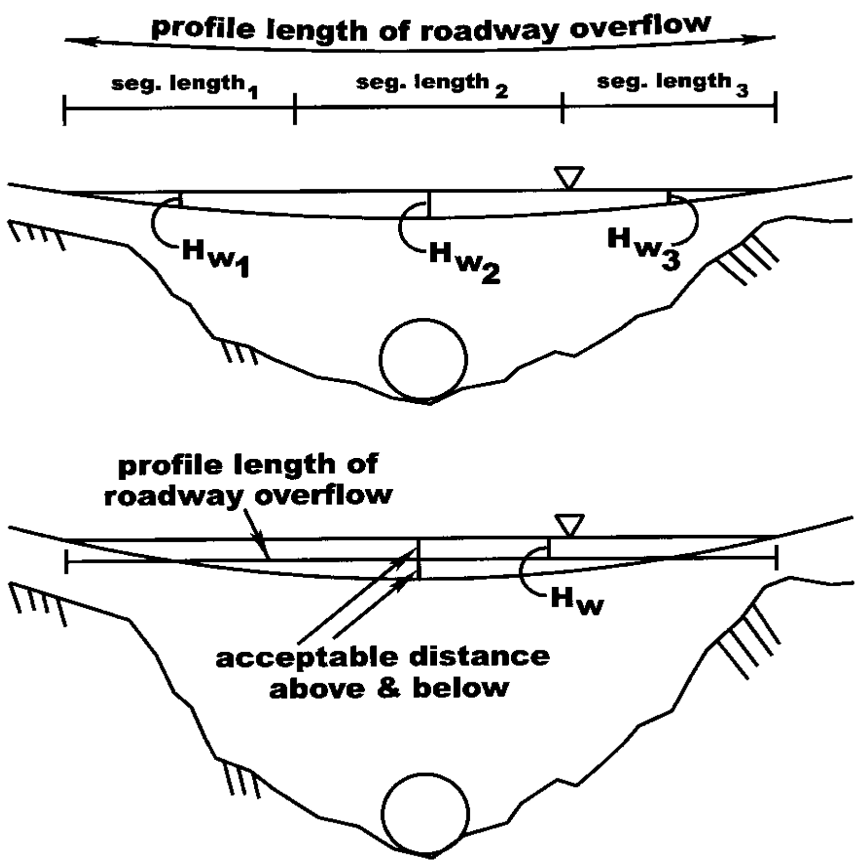
L = Profile length of the roadway overflow (ft)

H_W = Headwater depth measured above the roadway (ft)

The length of overflow and the headwater depth along the roadway are difficult to determine when the overflow is defined by a sag vertical curve. The sag vertical curve can be broken into a series of horizontal segments, and the flow over each segment is calculated for a given headwater using the weir equation (Figure 8–4). The given headwater is determined at the elevation along the vertical curve in the center of the horizontal segment. The overtopping flow rates for each segment are then added together, resulting in the total flow over the roadway.

Figure 8–4

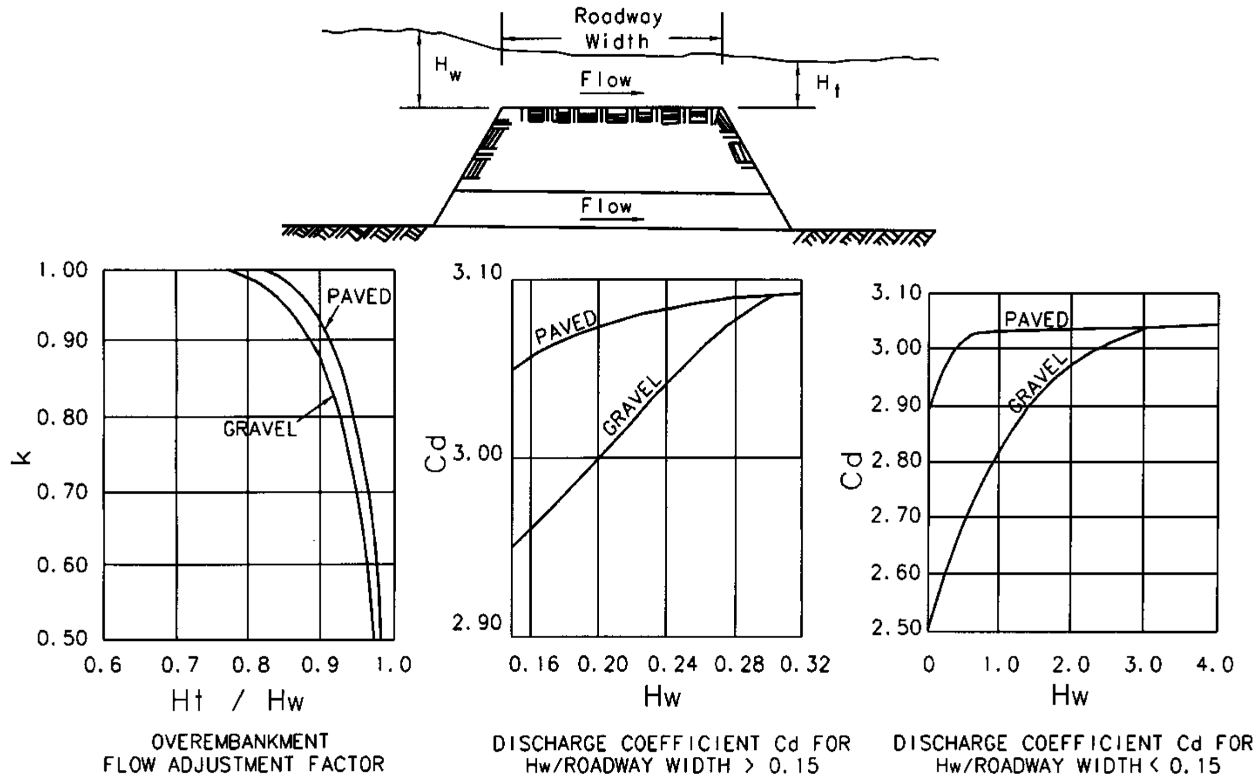
Roadway Overtopping



The sag vertical curve can also be adequately represented by a single horizontal line (one segment) with an acceptable variation above and below the horizontal line. The length of the overflow can be taken as this segment length or it can be based on the roadway profile. In effect, this method utilizes an average depth of the upstream pool above the roadway for the overflow calculation. Values of the weir coefficient (C_d) in English Units can be found in Figure 8–5 (Chart 60B from HDS-5). The roadway overflow plus the culvert flow must equal the design flow. A trial and error process is necessary to determine the amount of total flow passing through the culvert and the amount of flow over the roadway. Computer programs such as HY-8 are recommended when evaluating roadway overtopping.

Figure 8–5

Roadway Overtopping / Discharge Coefficient

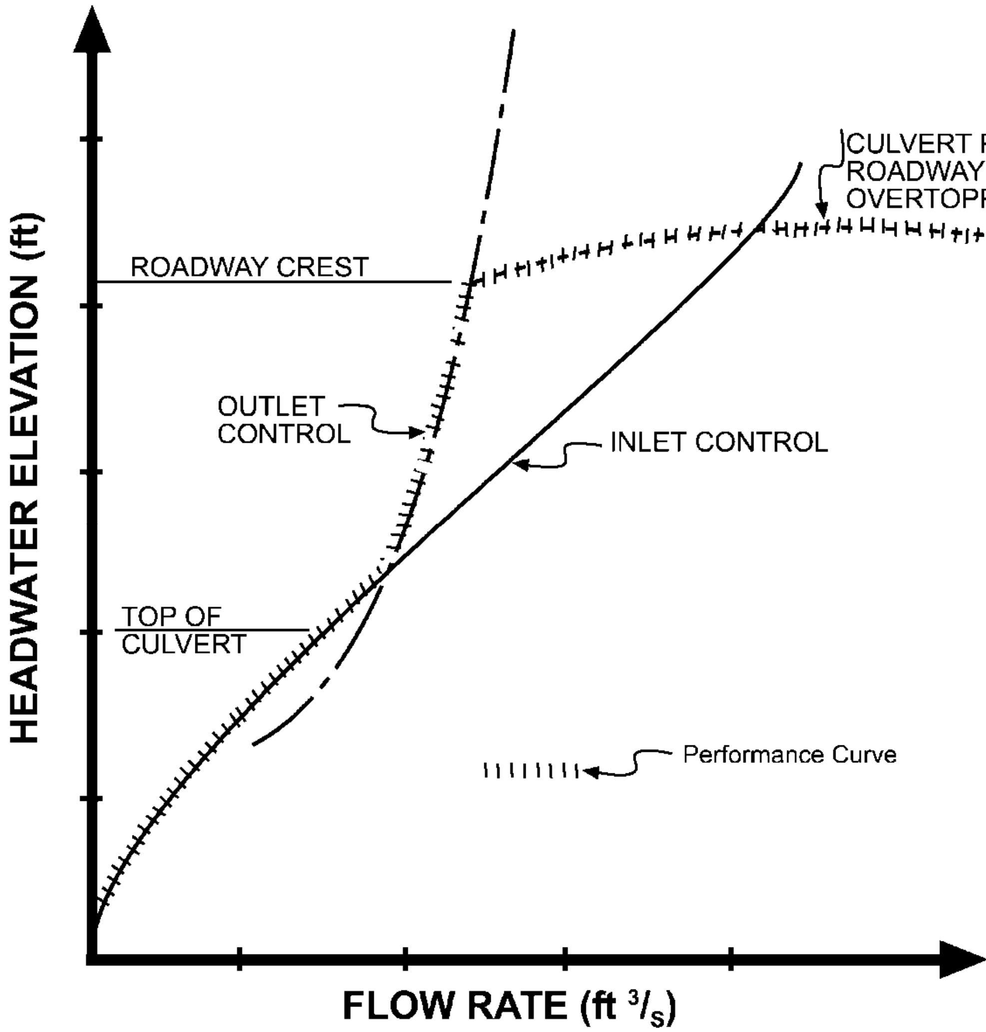


8.4.8 PERFORMANCE CURVES

Performance curves are plots of flow rate versus headwater depth or headwater elevation. Since the control section can exist at the inlet, outlet, or the throat of the culvert, a performance curve is possible for each control section including roadway overtopping. The overall performance curve is made up of the controlling portions of the individual curves for each control section. It can be used to determine the headwater depth or elevation for any flow rate, or to examine the performance of the culvert over a range of flow rates. Figure 8–6 depicts a typical culvert performance curve.

Figure 8–6

Overall Culvert Performance Curve



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

An overall performance curve can be developed as follows:

1. Select a range of flows falling above and below the design discharge and calculate the corresponding inlet and outlet control headwater elevations.
2. Plot and combine the inlet and outlet control performance curves to define a single curve for the culvert.
3. For culvert headwaters that overtop the roadway crest elevation, use the weir equation (see Section 8.4.7) to calculate flow rates over the roadway.
4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall performance curve.

8.4.9 ALLOWABLE HEADWATER

The economic design of culverts with headwater requires that consideration be given to the following effects:

- Hydraulic uplift or buoyancy, which is especially significant in permeable soils, and/or pipes with no headwalls. The possibility of uplift is increased when the culvert entrance becomes blocked with debris.
- Exfiltration in pipes due to pressure.
- Erosion of the embankment due to falling headwater.
- Danger to fills due to seepage especially in hillside locations.
- Debris protection.
- Maintenance.
- Damage to upstream and downstream property.
- Hazards to life.
- Public image.
- Acquisition of land affected by headwater. Areas inundated beyond levels of former flooding may need to be acquired.
- Possible future development of the land upstream.

Allowable headwater depth criteria are provided in Section 8.3.6. The check storm should be evaluated in accordance with Section 4.3.2 (see Chapter 4).

The area upstream of a culvert where ponding might occur can impact the design of the culvert. If the upstream ponding area is limited, the allowable headwater may need to be reduced. Conversely, if the upstream area has a large storage capacity, the allowable headwater elevation could be increased thus reducing the required culvert size. In the latter case, the Division of Highways shall acquire the right to pond water on the affected area to prevent future development and maintain the area for ponding. This will require the purchase of right of way, permanent ponding easement, or permanent drainage easement. The delineation of the ponding area will require mapping with at least 2-foot interval contours.

8.4.10 TAILWATER CONDITIONS

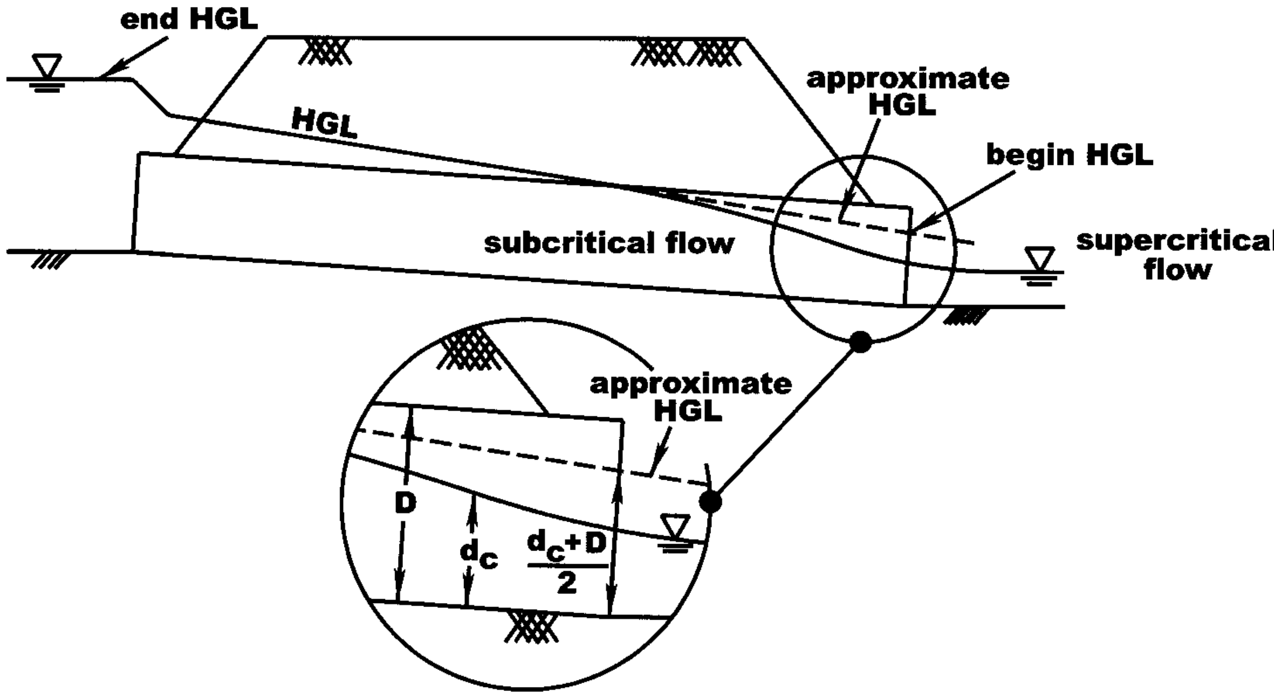
The tailwater elevation above the outlet invert at the design flow rate may be obtained from backwater or normal depth calculations, or from field observations. A field inspection and a review of flood insurance studies should be made to check the effect of downstream controls on tailwater conditions at the outlet. Backwater computations from a downstream control point can be tedious and may require additional survey of the area. Normal depth computations utilize Manning's Equation and the geometric properties of the outlet channel to determine a tailwater elevation, assuming the channel is reasonably uniform in cross section, slope and roughness. Once this elevation is determined, it is compared to the critical depth of flow for the culvert.

The critical depth of flow is a computable occurrence in the flow regime. Where this depth occurs, it has an effect on the tailwater condition for a culvert. In the case of a hydraulic drop, flow changes from a subcritical state (slow, deep flow) to a supercritical state (shallow, fast flow). The depth of flow must pass through the critical depth to make this change in flow state.

In the case of the culvert flowing full at the outlet (as in Figure 8–3), the tailwater elevation is easy to determine. In the case of the culvert flowing partly full at the outlet, the tailwater is determined by comparing the observed or computed tailwater (from backwater or normal depth methods) with the critical depth. Based on numerous backwater calculations by the FHWA, it was determined that for partly-full flow, a downstream extension of the full-flow hydraulic grade line pierces the vertical plane of the culvert outlet at a point halfway between critical depth (d_c) and the top of the barrel (culvert diameter, D). This means the change in the flow state occurs within the culvert and the tailwater elevation at the culvert outlet is $(d_c+D)/2$ above the outlet invert (Figure 8–7). This is the value used to set the beginning of the hydraulic grade line at the outlet for the calculation of headwater in an outlet control situation.

Figure 8–7

Hydraulic Grade Line Approximation



If the observed or computed tailwater elevation exceeds (d_c+D)/2, then it is used to set the beginning of hydraulic grade line. The headwater elevation (or end of the hydraulic grade line) is then determined by adding the exit loss, the head due to friction losses through the culvert barrel, the head due to entrance losses, and subtracting the change in the invert elevation (slope of culvert x length of the culvert) to (d_c+D)/2 or the tailwater, whichever is greater (see Section 8.5).

This approximate method by the FHWA works best when the barrel flows full over at least part of its length. When the barrel is partly full over its entire length, the method becomes increasingly inaccurate as the headwater falls further below the crown of the culvert. Adequate results are obtained down to a headwater of 0.75D. For lower headwaters backwater calculations are required to obtain accurate headwater elevations. Computer programs such as HEC-2, HEC-RAS or WSPRO should be utilized.

8.4.11 Culvert Slope

The culvert length and longitudinal slope should be based on existing site conditions and topography. The flow characteristics of the existing channel in the area of the proposed culvert should be examined to properly position the culvert vertically. The culvert invert should be as near as possible to the existing channel bottom and follow the existing stream bed alignment as close as is practical.

Where extremely steep grades are encountered, vertical breaks in grade can be introduced to reduce outlet velocities and minimize outlet protection requirements. The use of grade breaks in culverts should be coordinated with the WVDOH Engineering Division.

8.4.12 END TREATMENTS

Design of the inlet and outlet is a very important aspect of the overall design of a culvert. End treatments such as headwalls, wingwalls, end sections, and improved inlets can increase hydraulic efficiency, prevent buoyancy effects and reduce erosion. Culvert outlets are also important because of the potential for erosion caused by the increased flow velocity through the culvert. Other types of end treatments include scour protection with rock lining and energy dissipators.

8.4.12.1 HEADWALLS OR WINGWALLS

See Section 8.3.8 for guidelines on where to use headwalls, wingwalls or other end sections. Safety of the road user is an important consideration in the design and location of drainage structures. It is important to locate the headwall, wingwall or end section outside the clear zone on high-speed roads to eliminate the possible hazard to an errant vehicle. Where end treatments must be within the clear zone, they should be designed or modified to be traversable or present a minimal obstruction to an errant vehicle where debris is not a concern. The end section can be made traversable by using sloped grates or safety slope end sections. If a major drainage feature cannot effectively be redesigned or relocated, shield it using a suitable traffic barrier. Refer to the AASHTO Roadside Design Guide for further information on traffic safety issues associated with drainage features.

Wingwalls retain the roadway embankment and improve the hydraulic efficiency by reducing the inlet and outlet loss coefficients. Cut-off walls at the entrance or outlet of a culvert are used to prevent piping and subsequent undermining along the culvert barrel (see WVDOH Standard Details, Volume I, Sheet DR2).

8.4.12.2 IMPROVED CULVERT INLETS

Improved inlets are refinements to the geometry of the culvert entrance in order to increase the hydraulic performance (Standard Details Sheet DR2, sheet 1 of 4). These types of inlets should be considered for exceptionally long culverts operating under inlet control or when an existing culvert operating under inlet control is lengthened and hydraulic performance needs to be increased. There are three types of improved inlets:

- Beveled-Edge
- Side-Tapered

- Slope-Tapered

The bevel-edged inlet acts to decrease the flow contraction at the inlet and generally increases the culvert capacity by 5 to 20 percent depending on the type of entrance edge, wingwalls and depth of headwater.

The side-tapered inlet has an enlarged face area with tapered sidewalls that transition to the culvert barrel. This type of inlet provides an increase in flow capacity of 25 to 40 percent over that of a conventional culvert with a square edged inlet.

The slope-tapered inlet incorporates a steeper slope or fall in the enclosed entrance portion of the culvert. The increase in capacity with this inlet depends on the amount of fall available, but up to a 100 percent increase in capacity can be achieved over a conventional culvert with a square edged inlet.

The designer should refer to HDS-5 for detailed guidance on the design of improved inlets.

8.4.12.3 Scour Protection and Energy Dissipators

A pre-formed basin lined with rock is called a scour basin. The geometry of a rock lined scour basin can be determined using the Energy Dissipator Module of HY-8 version 7.1, which is based on the methods presented in HEC-14 published by the FHWA. Version 7.0 does not have this capability but future versions will.

High velocity culverts or culverts where a hydraulic jump cannot be avoided at the outlet may require an energy dissipator device to reduce the velocity. Energy dissipators work on the principle of inducing a hydraulic jump, controlling it within a stilling basin and finally transitioning the reduced velocity flow to the downstream channel. HEC-14 presents a variety of energy dissipator designs including the CSU basin, USBR impact basin, SAF stilling basin, rock lined scour basin and the VPI tumbling flow dissipator. The need for maintenance is an important consideration for such energy dissipators. The design should be based on HEC-14.

The following guide should be used for selecting the most appropriate outlet protection. It is based on a comparison of the pre-existing stream velocity, culvert outlet velocity and the maximum allowable velocity for the soil. The allowable velocities of the channel bed material are listed in Table 8-4. V_o is the culvert outlet velocity in feet/second.

- If V_o < allowable streambed material velocity, no protection is needed
- If V_o > allowable streambed material velocity, use dumped rock gutter or select embankment
- If V_o > 15fps, use rock lined basin or energy dissipator

Table 8-4

Allowable Velocities of Streambed Material

SOIL TEXTURE	ALLOWABLE VELOCITIES (feet/sec)
Fine sand and sandy loam (A-3)	2.5
Silt soils (A-4, A-5, A-6)	3.0
Silt or clayey gravel and sand (A-2)	3.5
Clayey soils (A-6, A-7)	4.0
Clay, fine gravel	5.0
Cobbles	5.5
Shale	6.0

8.4.13 SEDIMENT/BEDLOAD

A major concern with culverts involves the adverse effects of sediment and bedload deposition. Excessive deposition can partially block the culvert inlet, the barrel itself or the outlet and reduce the flow carrying capacity of the culvert. This can also result in a potential flood hazard or develop into a costly maintenance problem.

Culvert locations where potential sediment problems are anticipated require a sediment transport analysis. Whether sediment will be deposited or be scoured will depend on the ability of the upstream channel and the culvert to transport sediment under varying hydraulic conditions. There are four types of methods to evaluate sediment deposition and scour in a culvert: statistical, simplistic, complex and tractive shear. A description of these methods can be found in the 2005 AASHTO Model Drainage Manual, Chapter 9, Appendix C. These methods estimate the rate of sediment deposition versus the rate of scour or clean out under varying hydraulic conditions. This estimate predicts the potential for problems caused by sediment.

In most cases, the simplistic method of assessment will be adequate unless there are extenuating circumstances that dictate a more complex study. It is based on extreme conditions and it assumes the culvert barrel will fill more than the stream bed would if the culvert where not present. The existing channel flow line is assumed to be the limit of deposition except for aggrading channels. This method results in a ratio that describes the sediment movement. This ratio is the sediment transport ratio and is determined by the following equation:

$$R_G = \left(\frac{V_1}{V_2}\right)^3 \left(\frac{n_1}{n_2}\right)^4 \left(\frac{y_2}{y_1}\right)^{\frac{5}{3}}$$

Where $V_{1,2}$ = Average velocity in uniform flow (ft / s)

$n_{1,2}$ = Manning's roughness value

$y_{1,2}$ = Average depth of uniform flow (ft)

In this expression the subscript 1 refers to the reach upstream of the culvert and the subscript 2 refers to a location within the culvert. If this ratio is greater than 1 the deposition will occur in the vicinity of section 2. If this ratio is much greater than 1 expect nearly all of the sediment carried by the stream to be deposited in the vicinity of section 2. Since V_2 is within the culvert and an average velocity is taken throughout the length of the culvert, the location of section 2 can be taken at any point within the culvert.

In some instances, environmental considerations may require countersinking one or more culvert barrels. The purpose of countersinking a culvert is to allow the pipe barrel to fill with streambed material up to the profile of the streambed that existed prior to the culvert's installation. This is done to accommodate the passage of fish and other stream biota (see Section 8.4.17). Sediment transport calculations will be required to ensure that the desired depth of streambed material will be maintained in the culvert. Baffles may be required to hold the streambed material in the culvert during the design flow.

8.4.14 DEBRIS

Debris is defined as any natural or manmade material in the stream that has the potential to block the culvert opening and prevent it from performing its function. Debris potential at a site is dependent on the land use in the contributing watershed and the floodplain characteristics upstream of the culvert. A field reconnaissance of the upstream watershed should be conducted with particular attention given to the presence of shrubs and trees on eroded banks, stream susceptibility to flash floods and storage of manmade debris in the floodplain.

Accumulation of debris at a culvert's inlet or within the barrel can cause failure. The result will be increased headwater depths and flooding which can cause damage to upstream property and possible roadway overtopping. Accumulation can be reduced by avoiding skewed culverts and providing a smooth and well-designed inlet.

When a high potential for debris accumulation exists, the culvert entrance protection should be designed using the Federal Highway Administration's Hydraulic

Engineering Circular No. 9, “Debris-Control Structures”. Protection should be provided where experience or field observations indicate that the watercourse will transport a heavy volume of controllable debris. Debris protection design should be submitted as a part of the culvert design.

The type of debris protection will depend on the individual location. Debris interceptors can be placed at the entrance to the culvert or upstream of the culvert. Upstream interceptors come in the form of debris racks, floating drift booms and debris basins. Culvert entrance interceptors include debris risers and debris cribs. Normally debris protection will not be considered necessary for culverts that carry runoff from natural watersheds but it should be provided in areas where debris is a known problem. Problem areas could be where timbering or strip mining operations exist upstream, locations in mountainous or steep regions where the culvert is under high fill and where clean out access is limited. Maintenance access must be provided to allow for clean out of the debris control device.

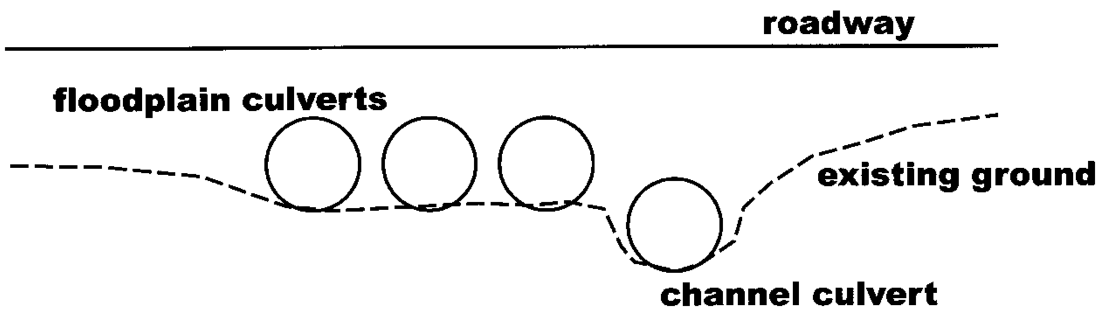
8.4.15 MULTIPLE CELL CULVERTS

Traditional culverts are sized to carry a low-frequency design discharge and are usually accompanied by channel modification that results in localized channel instability. The more common high-frequency events will flow with high-velocity, shallow flow through the culvert which can hinder fish passage. Traditional culverts on roadway embankments also block the floodplain and result in high-velocity flow concentration through the culvert, which can scour the outlet channel and cause “perching” of the culvert. Perched culverts can also impede fish passage.

Multiple cell culverts were developed to improve channel stability as well as facilitate fish passage. They should not be confused with traditional multiple-barrel culverts. Multiple cell culverts consist of one or more cells placed in the channel to convey flows up to the dominant discharge (or channel forming discharge) and one or more cells positioned in the floodplain to convey overbank flow up to the design discharge without increasing the water surface elevation of the 100-year discharge. The width of the channel culvert or total width of the channel culverts shall be equal to the bankfull width of the channel. This arrangement reduces the flow concentration through the culvert, which reduces channel scour.

Figure 8–8

Multiple Cell Culverts



The designer is cautioned that multiple cell culverts are not appropriate for all streams; therefore a stream stability assessment must be conducted to determine if they are appropriate. Multiple cell culverts should be considered only on stable streams that have active floodplains. A bridge is more appropriate for unstable streams or streams with no floodplain.

Scour protection of the floodplain cell outlets should be considered to prevent headcuts or erosion. It should be recognized that multiple cell culverts are a relatively new concept and their use is not widespread.

8.4.16 *Computational Methods*

Culverts can be designed using the design charts and nomographs from HDS-5 or with computer applications. Computer programs afford the designer the advantages of increased accuracy over the use of charts and graphs as well as the ability to perform numerous iterations of comparative designs quickly and easily. The results of computer applications however should be spot-checked for accuracy using the design charts and nomographs.

The HY-8 computer program is the most widely used program available to the designer today:

- HY-8 Culvert Analysis Microcomputer Program developed by the FHWA. This program is based on the methods presented in HDS-5.
- Commercially developed software is also acceptable.

8.4.17 *Accommodating Aquatic Life Movements*

Background

Decreasing Salmon populations lead fisheries biologists to believe that roadway culverts were preventing adult salmon from spawning. More recently, concern over movements of other aquatic species has lead researchers and regulators to expand their focus to all aquatic organisms. This includes all organisms that live in streams, or rely on streams as the pathway for necessary life cycle movements. Culverts have been determined to be barriers to passage of aquatic organisms in three ways:

- A high drop-off at the downstream end of the culvert keeps fish from entering the culvert. With a few exceptions, culverts are generally installed so that the invert of the pipe matches the bottom of the stream elevation. The high drop-off condition develops over time as a result of two possible causes. First, the streambed elevation downstream of the culvert has been lowered due to erosion after the culvert was installed. The second possible cause is that the water velocities are high enough to cause a scour hole at the outlet end of the culvert.
- Steep, smooth culverts have flow that is too shallow and too swift for fish to swim through.
- Long culverts exceed the endurance limit of the fish by not providing pools for fish to rest in.

Research and regulation began in the 70's. Most of the research in the U.S. has been focused along the northern Pacific coast, but efforts are increasing in the northeast and mid-Atlantic regions.

<u>Legal basis</u>

USACE Section 404 Nation Permit General Conditions, part C. 4. – Aquatic Life Movements. "No activity may substantially disrupt the necessary life-cycle movements of those species of aquatic life indigenous to the waterbody, including those species that normally migrate through the area, unless the activity's primary purpose is to impound water. Culverts placed in streams must be installed to maintain low flow conditions."

West Virginia 401 Water Quality Certifications Special Conditions for Nationwide 14 Linear Transportation Projects states that "The culvert barrel must be properly countersunk at the outlet." Reference is also made to "Appendix A for recommendations on proper culvert installation." These recommendations are from Oregon and Washington, and have not been widely accepted or implemented in West Virginia.

Proposed Design Methods and Policy

The WVDOH has established a goal to accommodate aquatic life movements (ALM) where it is beneficial, practical and feasible. In working toward this goal over the last few years, WVDOH has installed few experimental culverts, which have yielded mixed results. The following design method and policy is proposed to implement ALM on a limited basis.

Step 1: Determine the culvert size based on WVDOH Drainage Manual to ensure that hydraulic design requirements are met. Remember to consider FEMA requirements if the stream crossing is in a mapped flood zone.

Step 2: Collect field data at the site to determine whether ALM is applicable.

- List aquatic organisms present at site. Note state and federal Rare, Threatened and Endangered (RTE) species in conjunction with the Environmental Section of the Engineering Division's environmental document for the project.
- Consider whether the stream is perennial, intermittent, or ephemeral.
- Determine whether the stream is listed as a mussel stream.
- Determine whether the stream is a trout stream or an intermittent tributary of a trout stream.
- Measure bankfull width, bankfull maximum depth and channel slope.

Step 3: Based on the data collected in step 2, determine whether ALM is appropriate.

- If the stream supports aquatic life, then ALM should be considered. Regulation of this issue is still developing. Building a stream crossing to accommodate ALM may be required, or it may be considered as mitigation of stream impacts.
- If ALM is not appropriate, then size the stream crossing based on the hydraulic culvert design performed in step 1.

Step 4: Determine whether the stream crossing should have an open bottom structure such as a bridge or 3-sided structure, or a closed cell culvert with natural stream bed material. Based on WVDOH calculations, and current practice of other agencies, a stream slope of 5% is the upper limit for closed cell culverts with natural stream bed material. If the channel slope is less than 5%, follow steps 5 through 10 for closed cell culvert design. If the channel slope is greater than 5%, follow steps 11 through 14 for open bottom structure design.

Culverts

Step 5: Set the culvert width equal to the bankfull channel width. Investigate which structure types will be appropriate for this width.

Step 6: Determine what measures will be needed to ensure that the culvert will have adequate substrate and structure to allow ALM. Some culverts may be set at a low elevation and be expected fill in naturally over time. Others will require streambed material placement inside the culvert.

Step 7: For culverts that require streambed material placement inside the culvert, determine the size and thickness of the material to fill the bottom of the culvert. Based on current information:

$$D_{50} = 15.6dS$$

Where D_{50} = the median size of well graded stream bed material

d = bankfull maximum depth (ft)

S = channel slope

The thickness of the stream bed material should be approximately $2D_{50}$.

Step 8: Determine the culvert height based on steam bed material thickness and bankfull maximum depth.

Step 9: Determine which structure sizes and types are acceptable based on fill heights and hydraulic requirements in step 1.

Step 10: Complete final design of culvert.

Bridges and three-sided structures

Step 11: Set the structure width equal to or greater than the bankfull channel width. Investigate which structure types will be appropriate for this width.

Step 12: Perform bridge scour analysis.

Step 13: Obtain core borings.

Step 14: Design the structure and foundations.

All stream crossings

Step 15: Prepare drawings and quantity tables for the permit applications.

Ongoing research

- FWHA has commissioned Washington State University to develop “Fish Passage for Bridges and Culverts”, HEC-26
- Mark Hudy, USDA Forest Service/James Madison University is conducting research in Pocahontas County.
- West Virginia University and Marshall are conducting research commissioned by WVDOH.
- Other Universities and state and federal agencies.

8.5 CULVERT DESIGN PROCEDURE

The following is a step-by-step procedure to determine the minimum size of a culvert. While it is possible to follow the design method without an understanding of culvert hydraulics, it is not recommended as it can result in an inadequate and possibly unsafe structure. Therefore the designer is advised to become familiar with the detailed procedures here. More information is provided in HDS-5.

Step 1: Assemble site data and culvert project file

a. Minimum data are:
   - USGS site and location maps
   - Embankment cross sections
   - Roadway profile
   - Existing channel profile
   - Photographs
   - Field visit (check for downstream controls, sediment, debris, erosion, high water marks, etc.)
   - Surveyed elevations of nearby structures and design data of nearby hydraulic structures

b. Studies by other agencies including:
   - Small dams (NRCS, USGS, etc)
   - Floodplain (NRCS, FEMA, USGS, NOAA, USACE, etc.)
   - Storm drain (local or private)

c. Environmental constraints including:
   - Commitments contained in Environmental documents
   - Environmental mitigation
   - Aquatic life movement, see Section 8.4.17

d. Review Design Criteria in Section 8.3, and the design directives

Step 2: Calculate design discharge (Q):

a. Determine design frequency based on the design criteria for the roadway classification

b. Determine Q from Form Form 4-1 or Form 4-2 (Chapter 4)

c. Divide Q by the total number of barrels, if more than one barrel is used

Step 3: Determine tailwater conditions based on the downstream channel and stream flow:

a. Review Chapter 7

b. Minimum data are cross sections, estimate of Manning's roughness coefficient for stream, geometry of channel and investigation of downstream controls.

Step 4: Summarize data on Form 8-1(see Section 8.6):

a. Fill in data from step 2

b. Fill in all other information including station, location, description, project number, etc.

c. Determine maximum allowable headwater (in feet), which is the vertical distance from the culvert inlet invert (flow line) to the allowable water surface elevation in the headwater pool or approach channel upstream of the culvert

Step 5: Select design alternative

a. Choose culvert material, shape, entrance type, and trial size (for example, using inlet control nomographs, assume HW/D = 1.5 with the design discharge and determine a preliminary size). If the trial size is too large in dimension because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge equally between the number of barrels used. Consider raising the embankment height or the use of a pipe arch or box culvert with a width greater than the height. Final selection should be based on an economic analysis of the alternatives.

b. Review the West Virginia Standard Specifications and Standard Details to ensure compliance and evaluate need for special details or special provisions.

Step 6: Determine inlet control headwater depth (HW_i)

Use the inlet control nomographs in Section 8.6 for the selected culvert shape and material.

a. Locate the size or height on the scale

b. Locate the discharge

- For a circular or arch shape use discharge
- For a box shape use Q per foot of width

c. Locate HW/D ratio

- Use a straight edge
- Extend a straight line from the culvert size through the flow rate.
- Mark the first HW/D scale. Extend a horizontal line to the scale for the end treatment to be used and read HW/D and note it on Form 8-1

d. Calculate headwater depth (HW_i)

- Multiply HW/D by D (pipe diameter if circular shape, height of culvert if box shape or arch shape is used) to obtain HW.
- Neglecting the approach velocity HW_i = HW
- Including the approach velocity HW_i = HW-approach velocity head.

Step 7: Determine outlet control headwater depth at inlet (HW_o)

a. Calculate the normal depth (d_n) in feet above the outlet invert using the design flow rate (single section) or using a backwater profile for the downstream channel. A measurable tailwater (TW) should also be noted here.

b. Calculate critical depth (d_c) using appropriate charts in Section 8.6.

- Locate flow rate and read d_c (A is the area of flow)
- d_c cannot exceed D

c. Calculate (d_c + D)/2

d. Determine (h_o)

h_o = the larger of the measurable TW, normal depth or (d_c + D)/2

e. Determine the Entrance Loss Coefficient (k_e) used to determine the entrance head loss (H_e). Coefficient k_e is multiplied by the velocity head ($V^2/2g$) to determine the head loss at the entrance to a culvert operating full or partially full with control at the outlet. Entrance loss coefficients for various inlet configurations are provided in Table 8-3.

f. Determine the total head losses (H_L).

- Use outlet control nomographs.
- Locate appropriate k_e scale.
- Locate culvert length (L) or (L1):
  - Use (L) if Manning's n matches the n value of the culvert and
  - Use (L1) to adjust for a different culvert n value.

$$L1 = L\,(n_1/n)^2$$

- Mark point on turning line:
  - Use a straight edge and
  - Connect culvert size with the length on the appropriate k_e scale and mark a point on the turning line.
- Read (H_L).
  - Use a straight edge,
  - Connect discharge (Q) and mark on the turning line,
  - Read (H_L) on Head Loss Scale.

g. Calculate outlet control headwater (HW_o).

$$HW_o = H_L + ho - LS_o$$

Where, S_o = slope of culvert

L = length of culvert

Therefore, LS_o is the difference in elevation of the invert in and invert out of the culvert (fall through the culvert).

Step 8: Determine controlling headwater (HW).

a. Compare HW_i and HW_o. The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

b. If outlet control governs and the HW is higher than is acceptable, select a larger trial size and return to Step 7.

Step 9: Compute Outlet Velocity and Depth.

a. If inlet control governs, outlet velocity can be assumed to be equal to the mean velocity in open channel flow within the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of the selected culvert.

b. If outlet control governs:

- Use dc if dc > TW
- Use TW if dc < TW < D
- Use D if D < TW
- Calculate flow area A

- Calculate exit velocity V = Q/A.

Step 10: Determine need for Culvert Outlet Protection.

a. Determine mean and maximum allowable flow velocities for the natural stream

b. Design protection based on Section 8.4.12.3

Step 11: Review Results:

Analyze the design alternative with regard to constraints and assumptions made in the design process. If any of the following constraints are not met, repeat steps 5 through 10 with another alternative design:

a. Allowable headwater is not exceeded;

b. Check storm criteria in Chapter 4 are complied with;

c. Culvert barrel material has adequate cover;

d. Actual length of the culvert is close to the approximated length;

e. Culvert end treatments can be accommodated by site conditions.

f. Allowable velocity is not exceeded.

If the above constraints are satisfied,

- Record final selection of culvert with size, type, required headwater, outlet velocity, and economic justification under recommendation on Form 8-1.
- Prepare report if needed and file with the background information.

8.6 COMPUTATION FORMS AND DESIGN CHARTS

Nomographs, charts, computation forms, step-by-step procedures, and example problems pertaining to all aspects of hydraulic design of highway culverts can be found in HDS-5. WVDOH design forms and charts are included in this section. Refer to the HDS-5 for complex situations not covered by the charts included in this section.

Form 8-1

Culvert Design Form

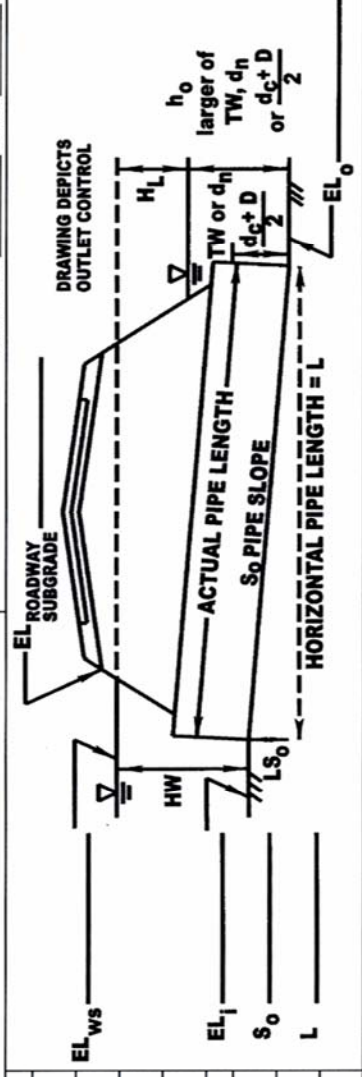
DRAINAGE COMPUTATION FORM 8 - 1
CULVERT DESIGN

PROJECT NAME ______________________
PROJECT NUMBER ______________________

STATION ______________________
DESIGNER __________ DATE __________
REVIEWER __________ DATE __________

OUTLET CHANNEL TAILWATER		
	RETURN PERIOD	TW OR NORMAL DEPTH
DESIGN STORM		
CHECK STORM		

CULVERT FLOWS		
	RETURN PERIOD	FLOW
DESIGN STORM		
CHECK STORM		

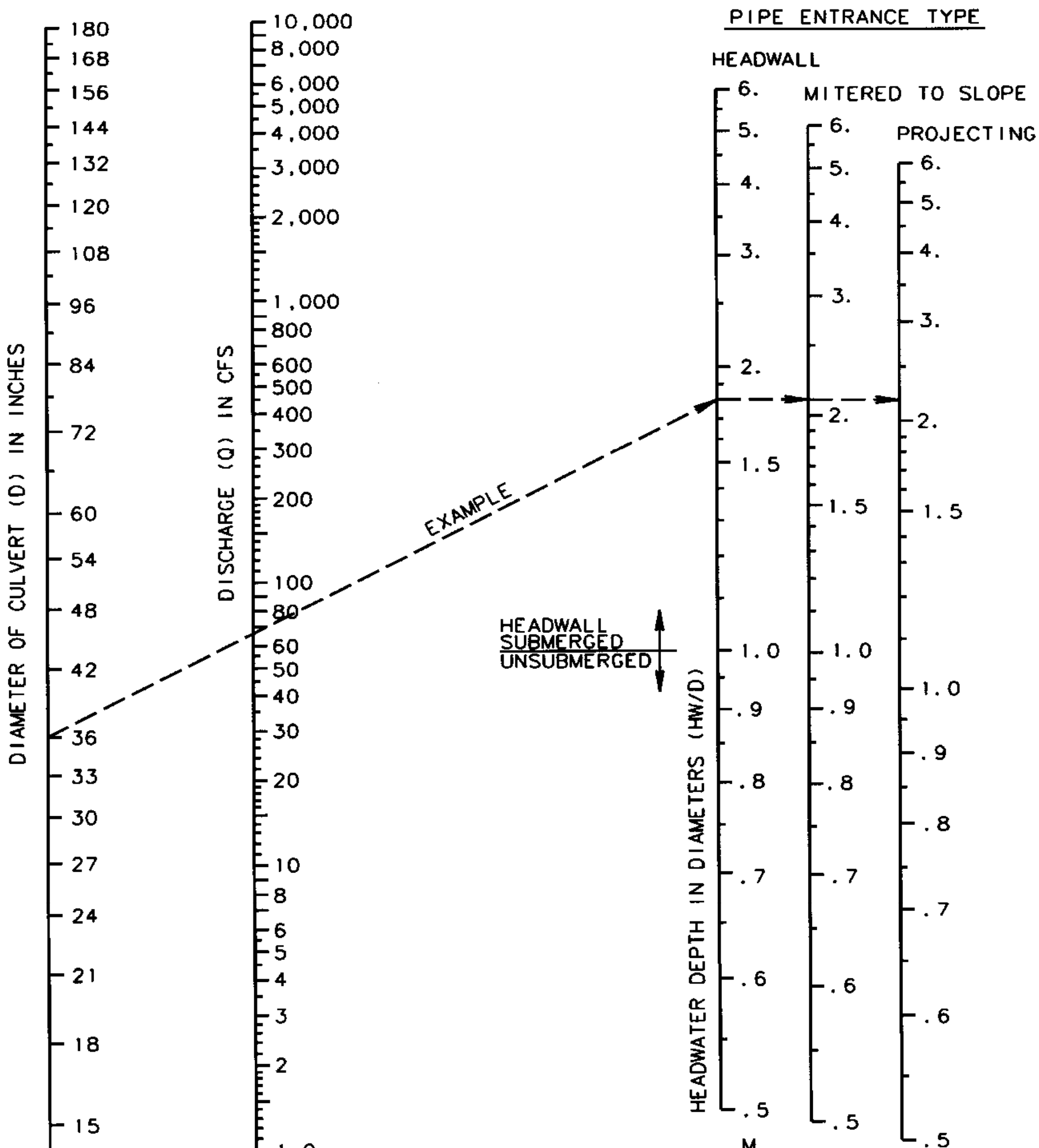


CULVERT DESCRIPTION				INLET CONTROL		OUTLET CONTROL										
MATERIAL, ENTRANCE, ROUGHNESS	CULVERT DIAMETER OR SHAPE	TOTAL FLOW	MAX ALLOWABLE HEADWATER	HEADWATER TO DEPH RATIO	HW AT INLET DUE TO INLET CONTROL	TAILWATER OR NORMAL DEPTH IN CHANNEL	CRITICAL DEPTH IN PIPE	PIPE CRITICAL DEPTH & DIAMETER AVERAGE	WSEL AT THE PIPE OUTLET	ENTRANCE LOSS COEFFICIENT	HEAD LOSS DUE TO FLOW THROUGH THE CULVERT	FALL THROUGH THE CULVERT	HW AT INLET DUE TO OULET CONTROL	CONTROLLING HEADWATER	OUTLET VELOCITY	REMARKS
		Q	H_{MAX}	HW_i / D	HW_i	TW, d_n	d_C	$(d_C+D)/2$	h_o	k_e	H_L	$L S_o$	HW_o	HW	V	
		cfs	ft		ft	ft	ft	ft	ft		ft	ft	ft	ft	ft / s	

RECOMMENDATION:

Chart 8-1

C.M. Pipe Culverts with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^M - 0.5S^2$

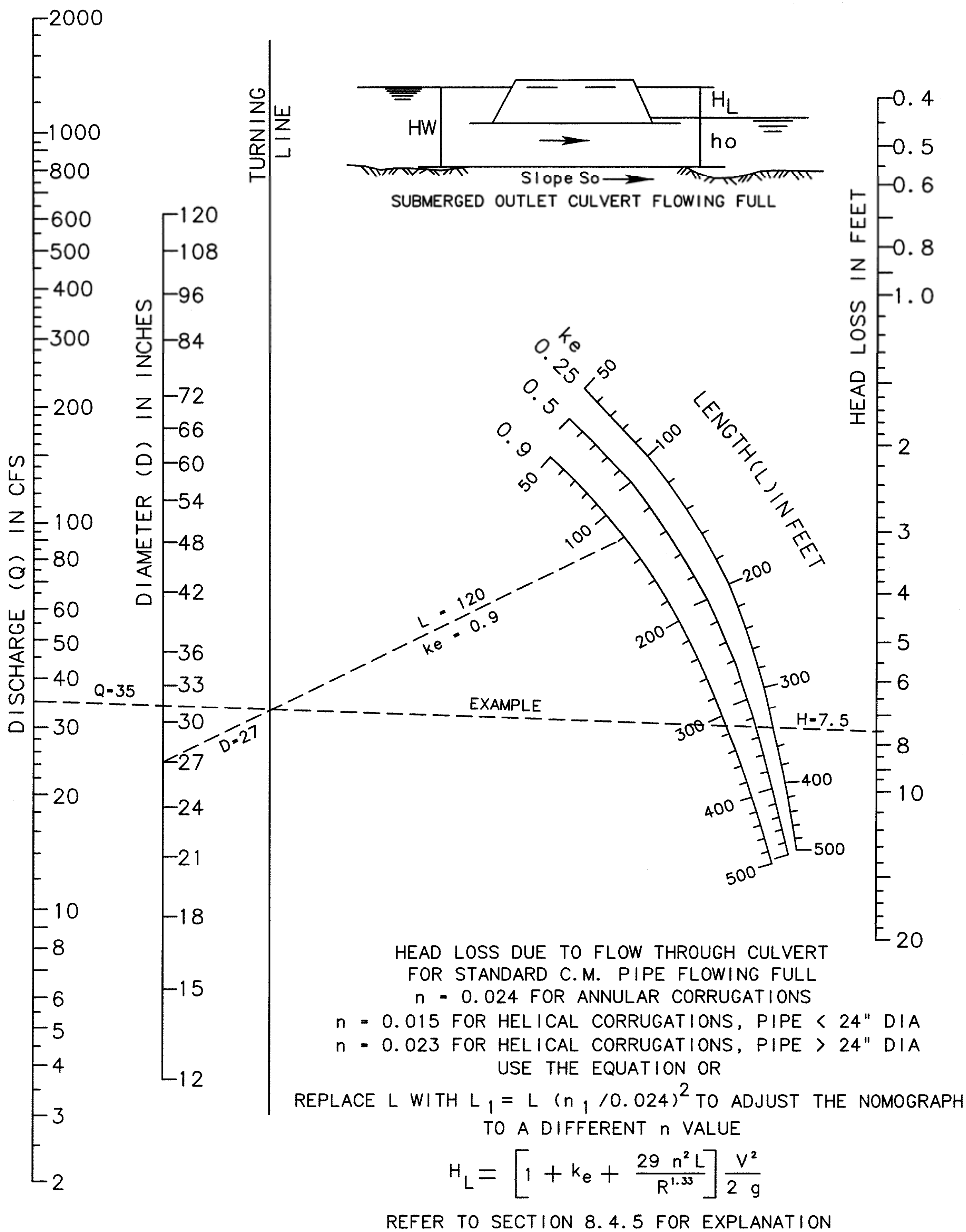
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S^2$

SEE TABLE 8-3 AND TABLE 8-4 FOR K, M, c, Y
REFER TO SECTION 8.4.5 FOR EXPLANATION

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-2

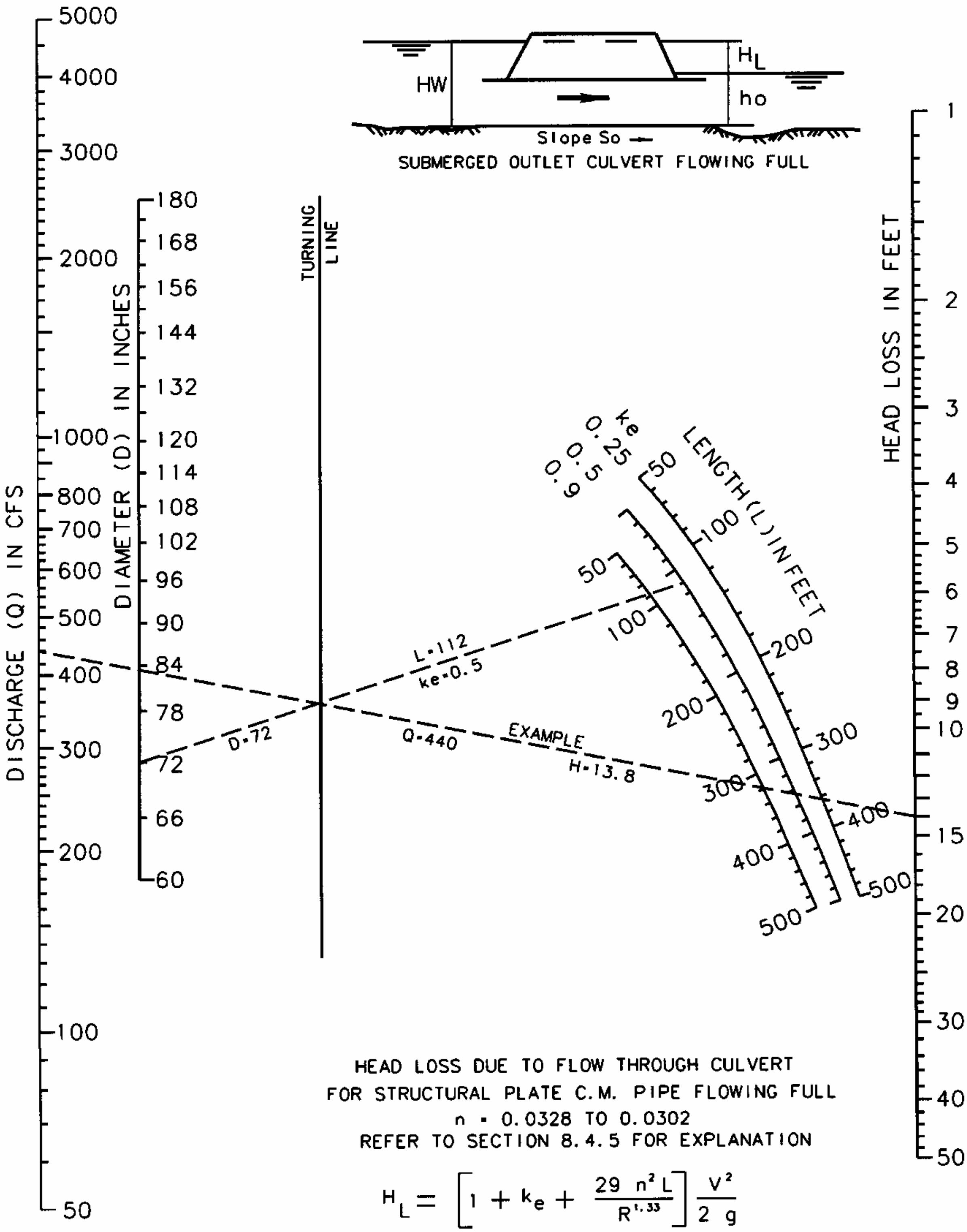
Standard C.M. Pipe Flowing Full



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-3

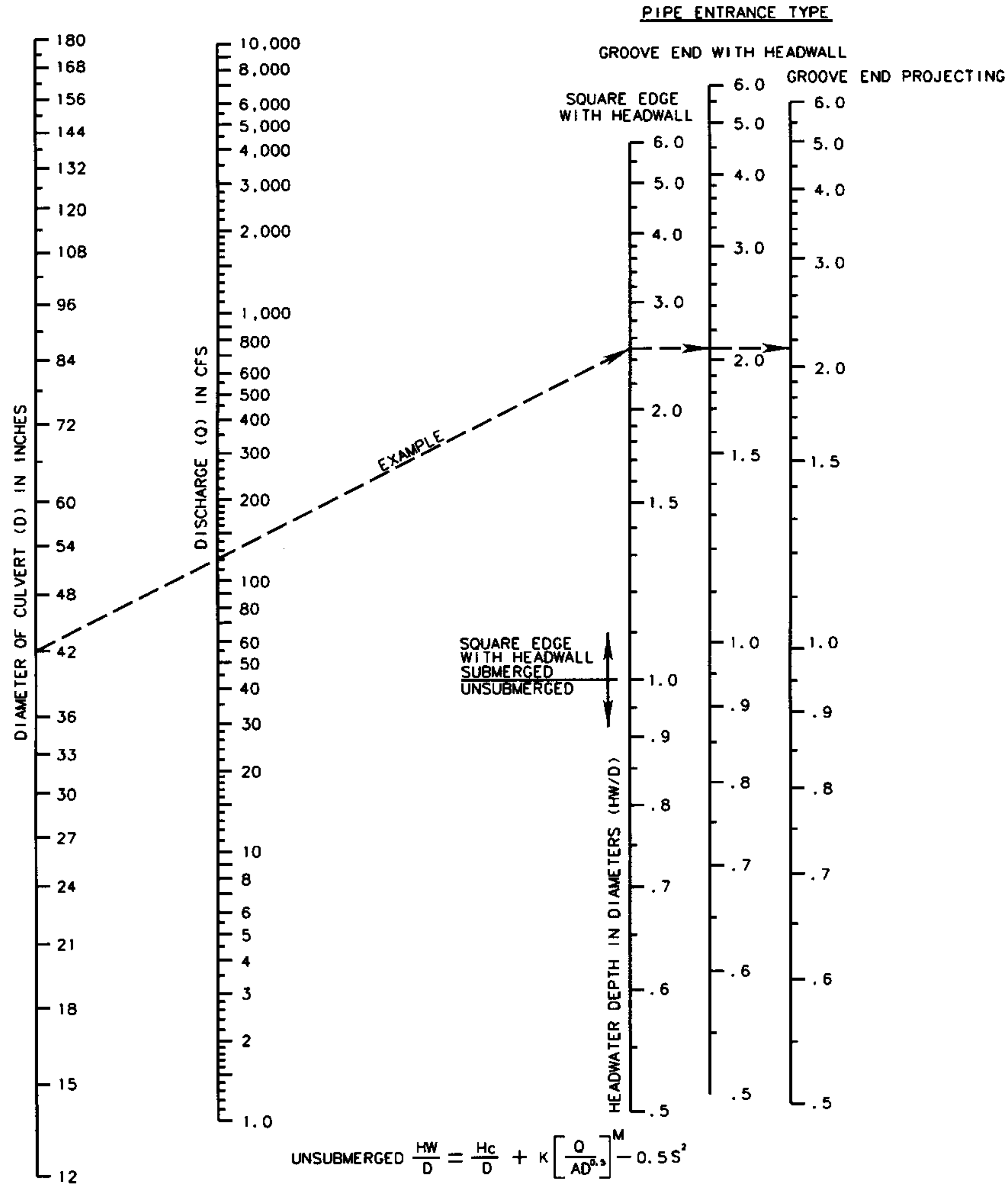
Structural Plate C.M. Pipe Flowing Full



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-4

Concrete Pipe Culverts with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^2$

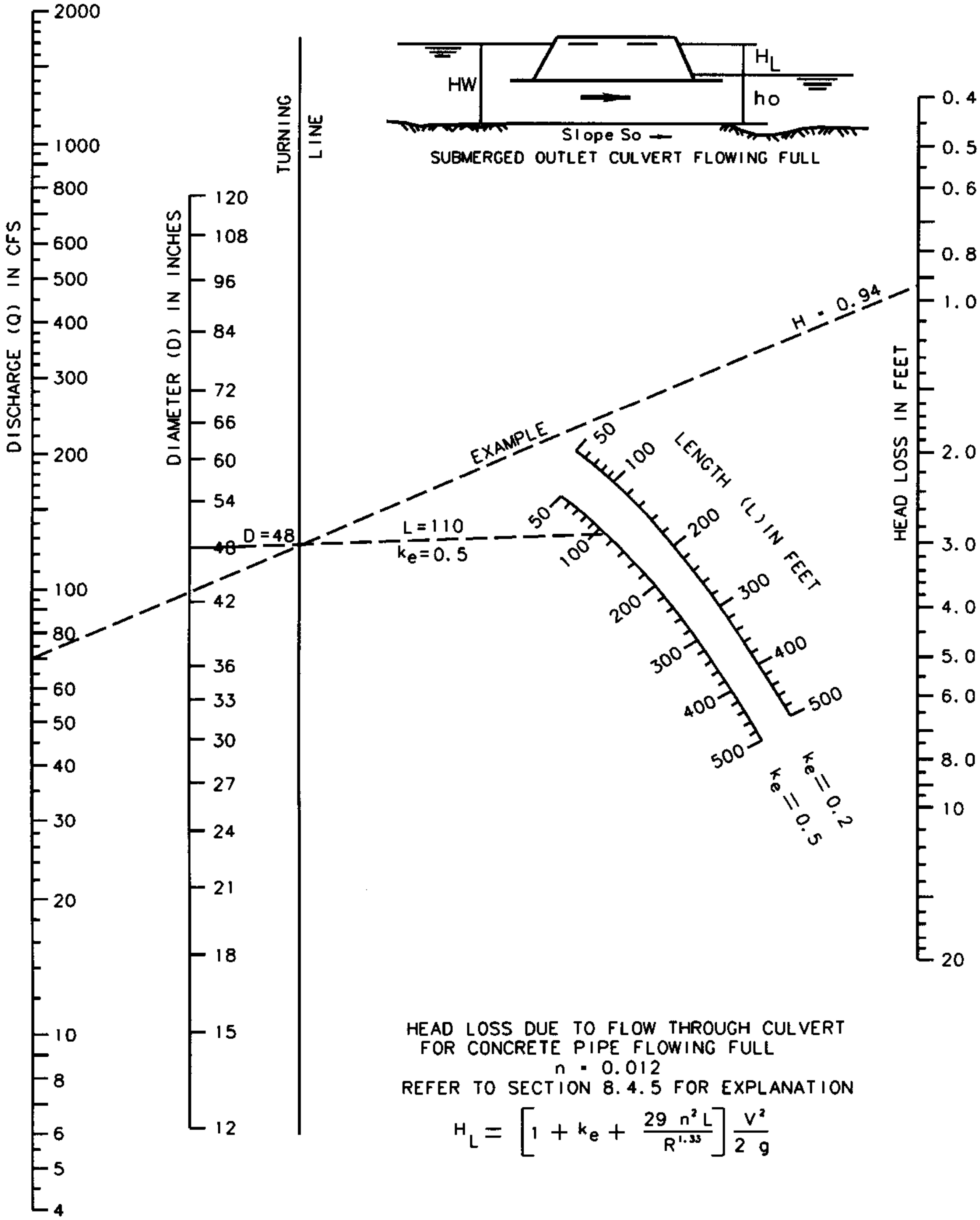
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^2$

SEE TABLE 8-3 AND TABLE 8-4 FOR K, M, c, Y

REFER TO SECTION 8.4.5 FOR EXPLANATION

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-5
Concrete Pipe Flowing Full

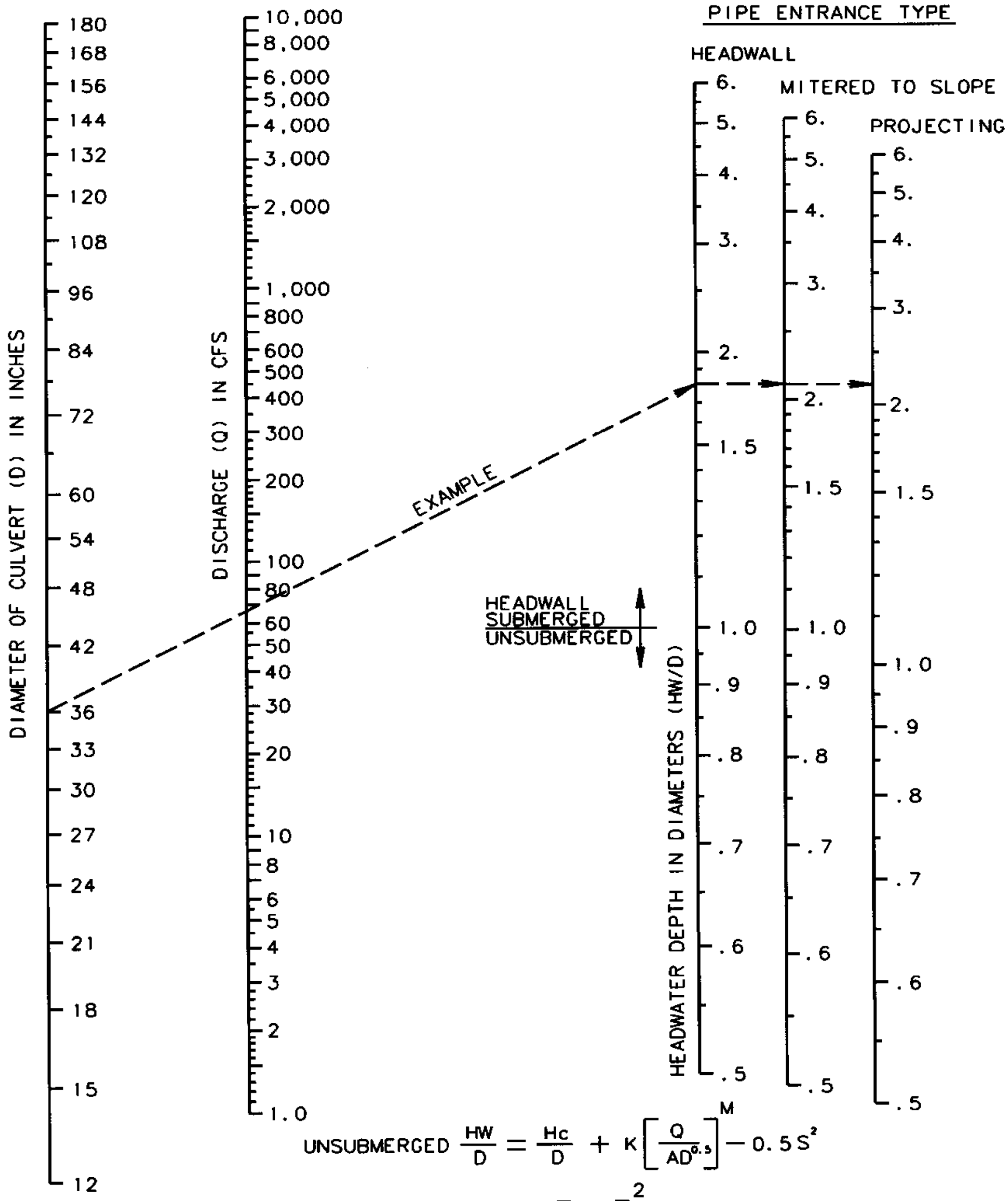


$$H_L = \left[1 + k_e + \frac{29\ n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-6

HDPEP Pipe Culverts with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^{2}$

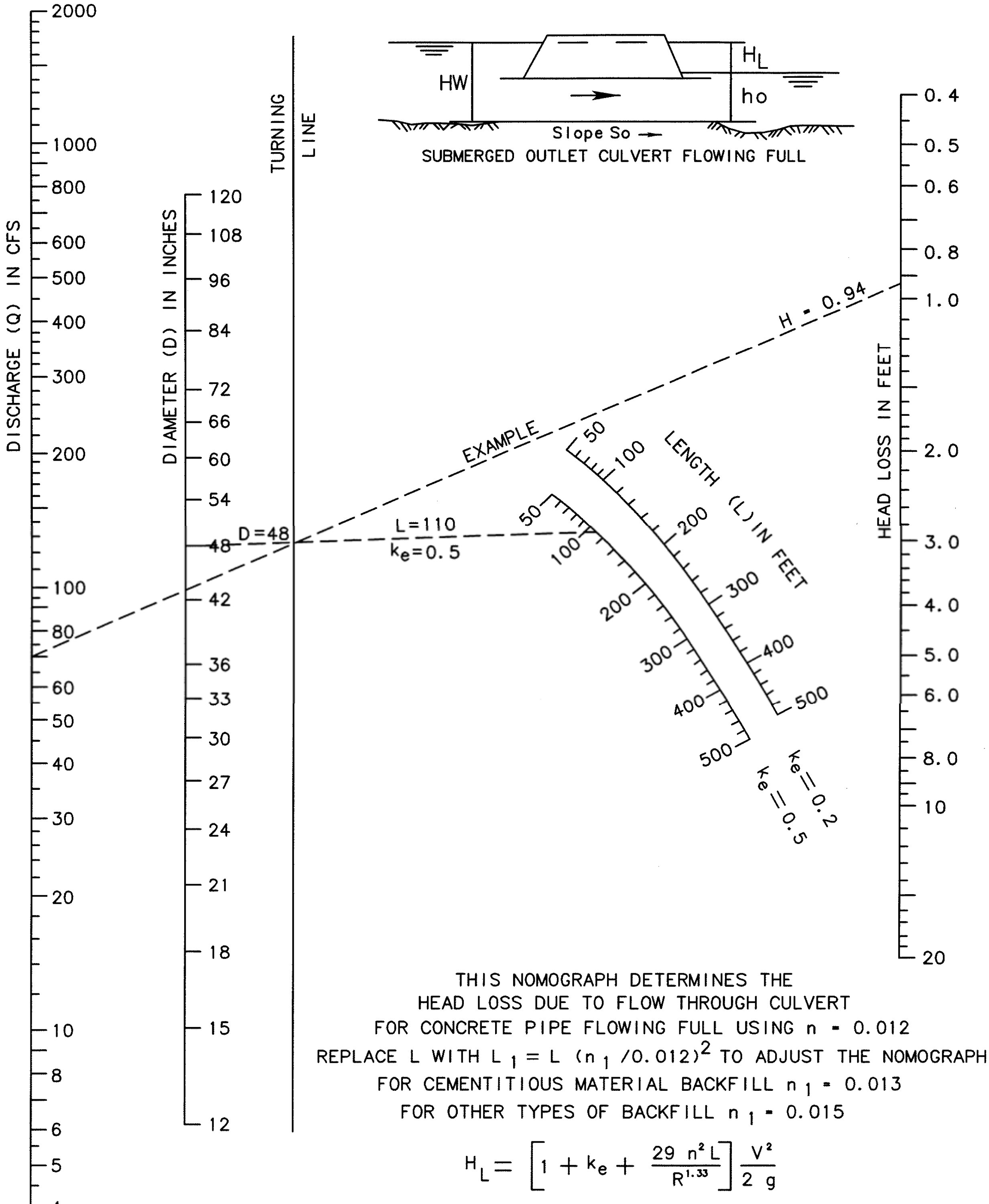
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^{2}$

SEE TABLE 8-3 AND TABLE 8-4 FOR K, M, c, Y
REFER TO SECTION 8.4.5 FOR EXPLANATION

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-7

HDPEP Pipe Flowing Full



THIS NOMOGRAPH DETERMINES THE
HEAD LOSS DUE TO FLOW THROUGH CULVERT
FOR CONCRETE PIPE FLOWING FULL USING n = 0.012
REPLACE L WITH $L_1 = L\ (n_1/0.012)^2$ TO ADJUST THE NOMOGRAPH
FOR CEMENTITIOUS MATERIAL BACKFILL $n_1 = 0.013$
FOR OTHER TYPES OF BACKFILL $n_1 = 0.015$

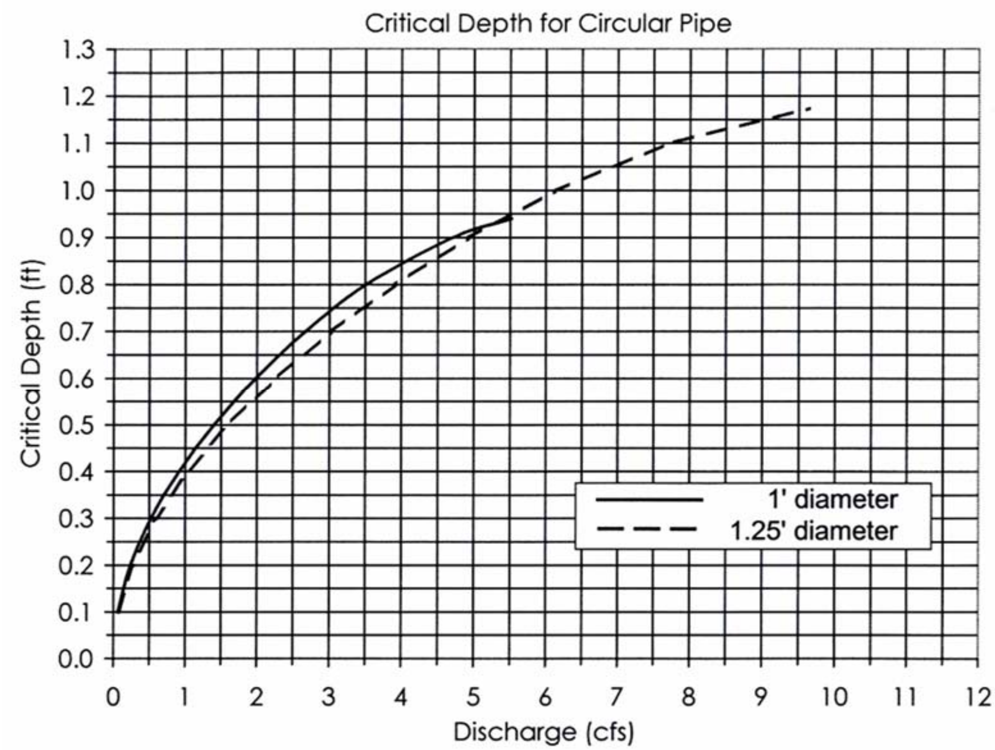
$$H_L = \left[1 + k_e + \frac{29\ n^2 L}{R^{1.33}}\right] \frac{V^2}{2\ g}$$

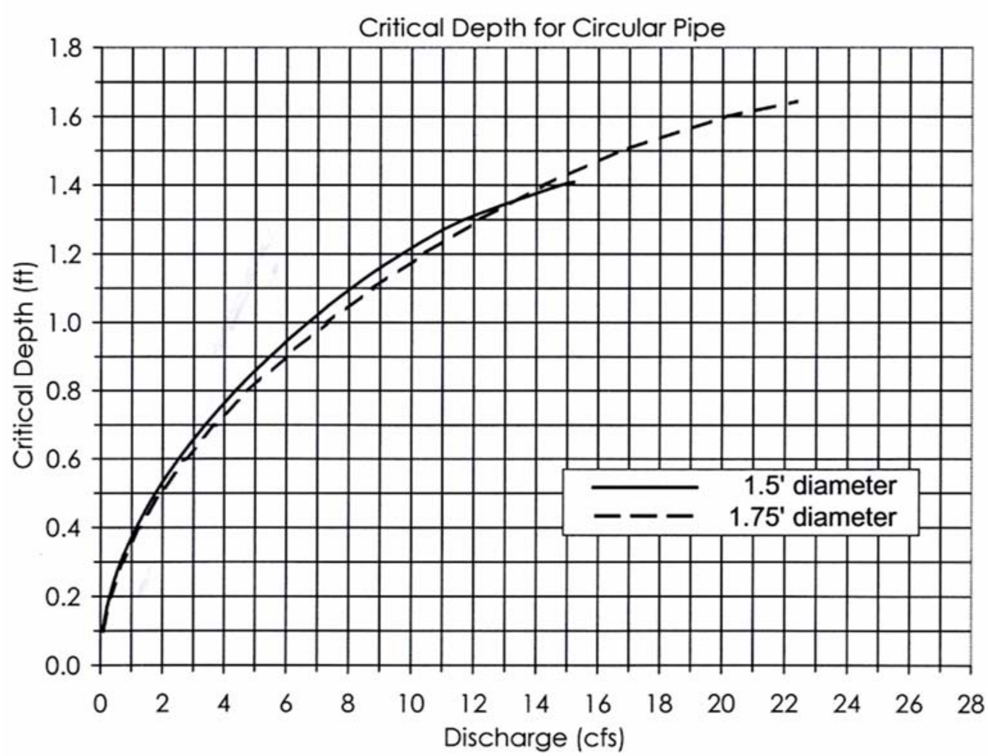
REFER TO SECTION 8.4.5.2 FOR EXPLANATION

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-8

Critical Depth for Circular Pipe

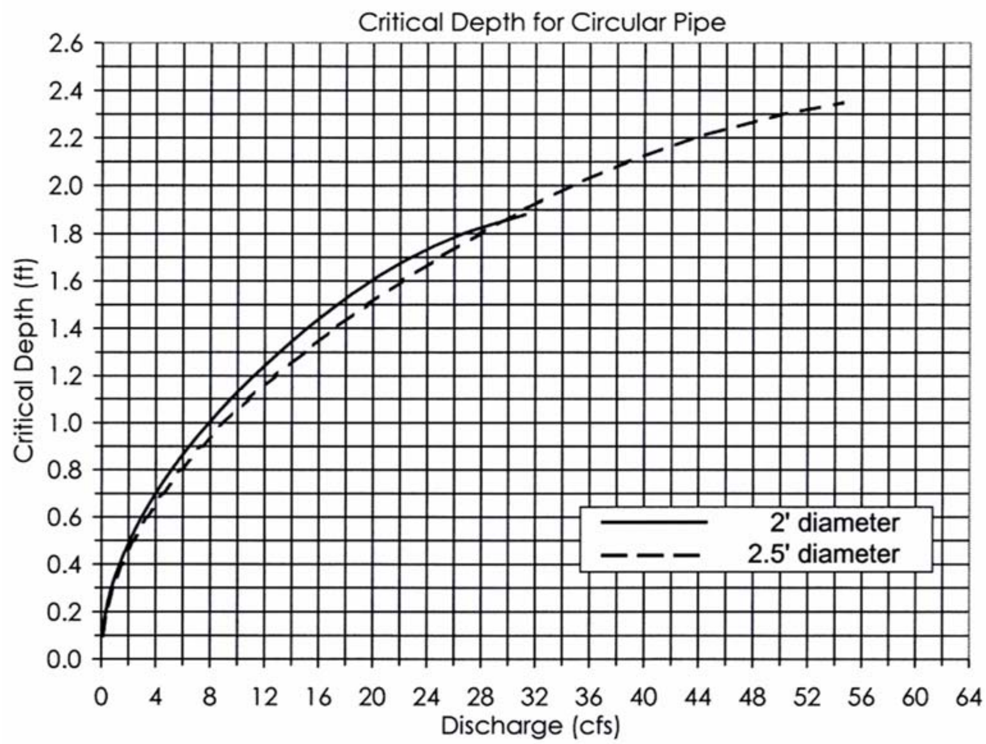




Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-9

Critical Depth for Circular Pipe



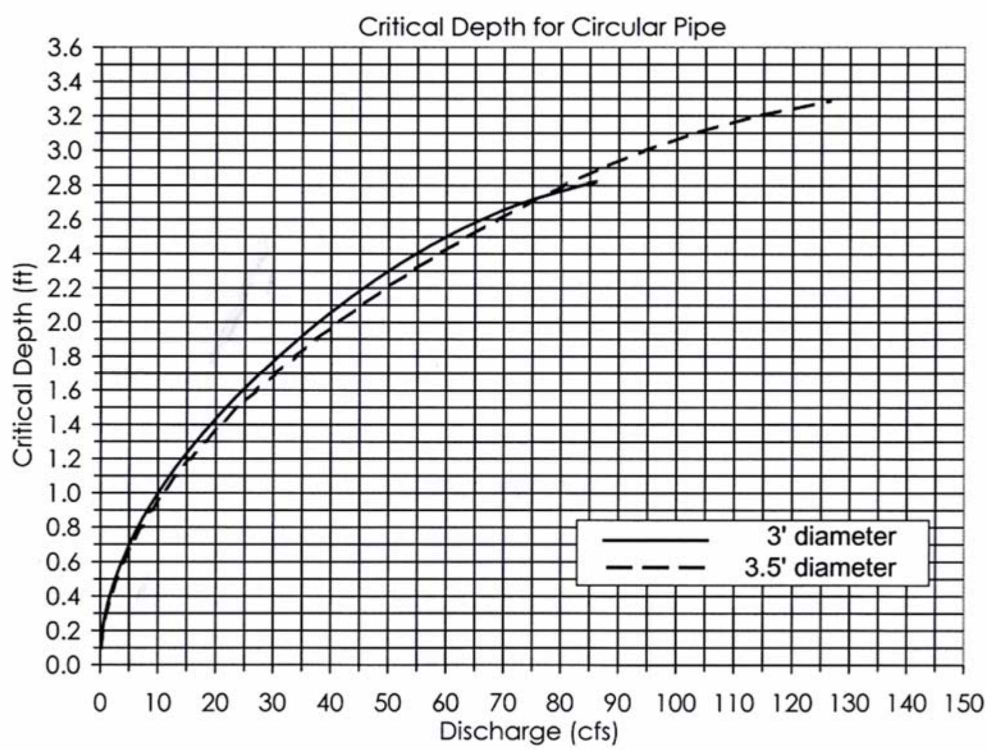


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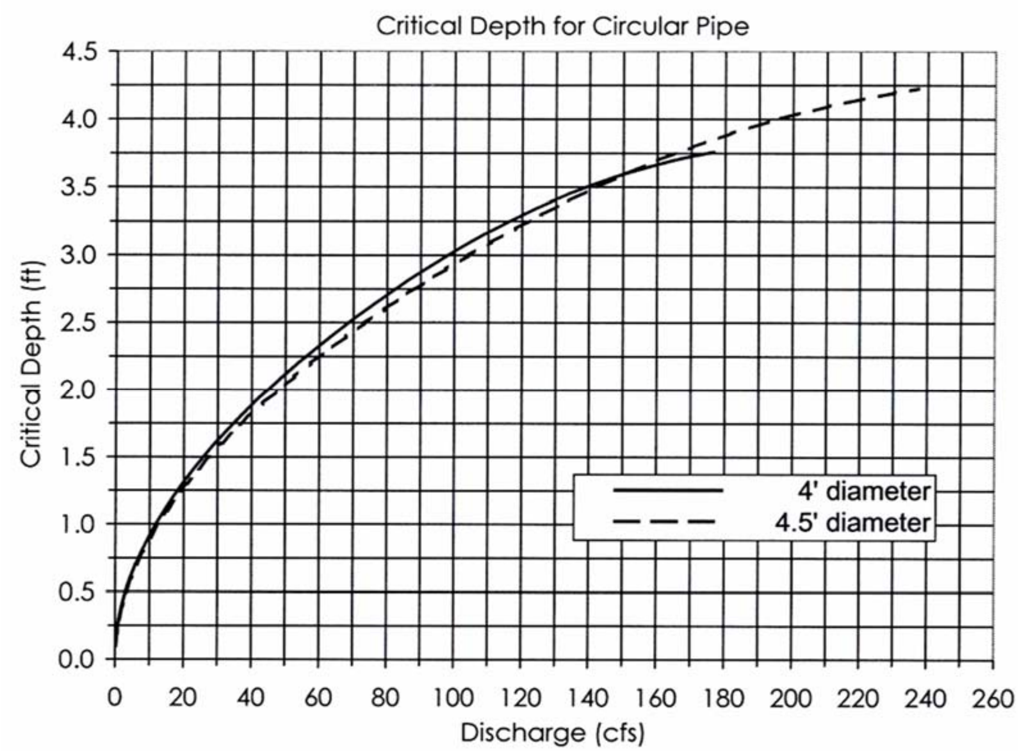
Chart 8-10

Critical Depth for Circular Pipe



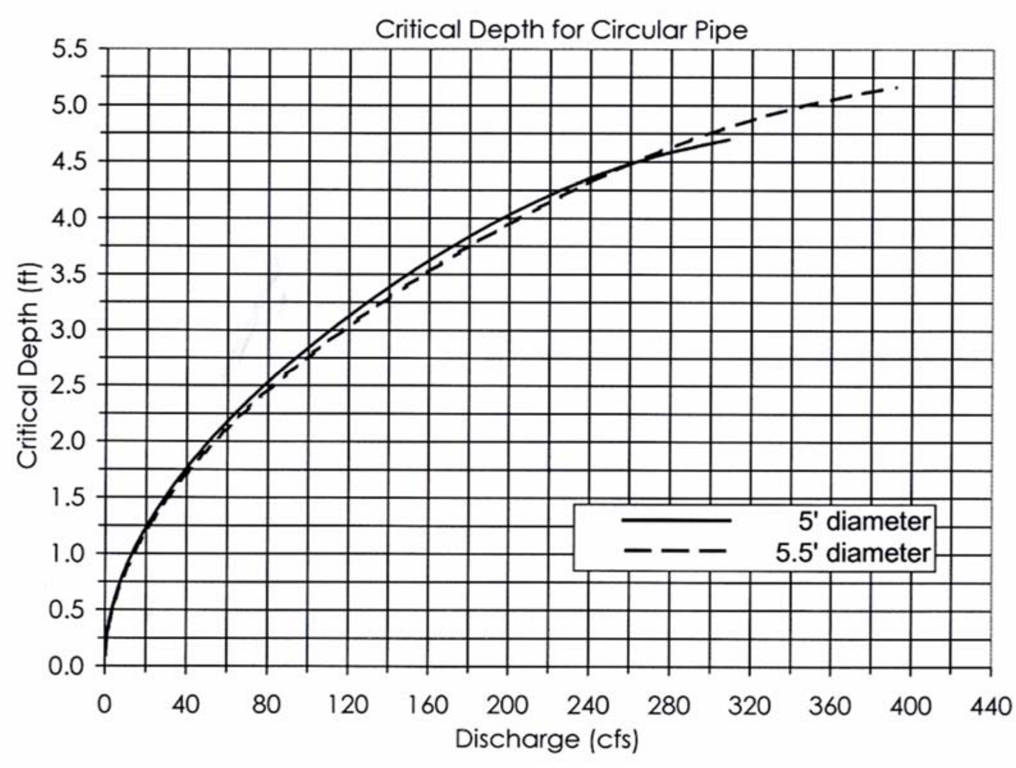


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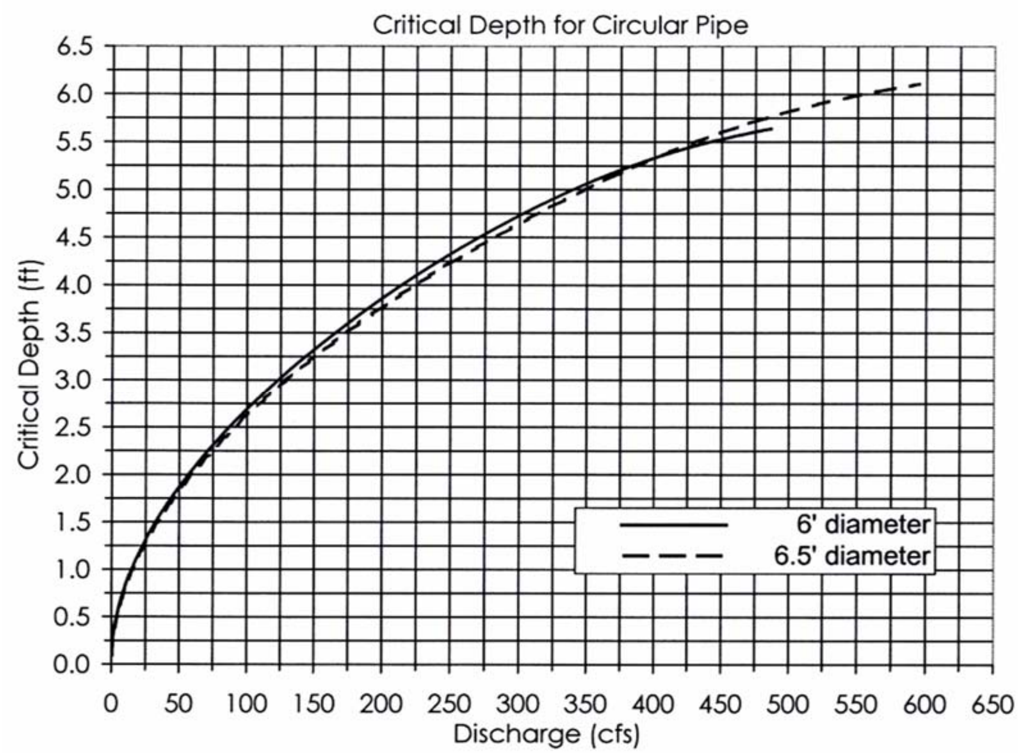
Chart 8-11

Critical Depth for Circular Pipe



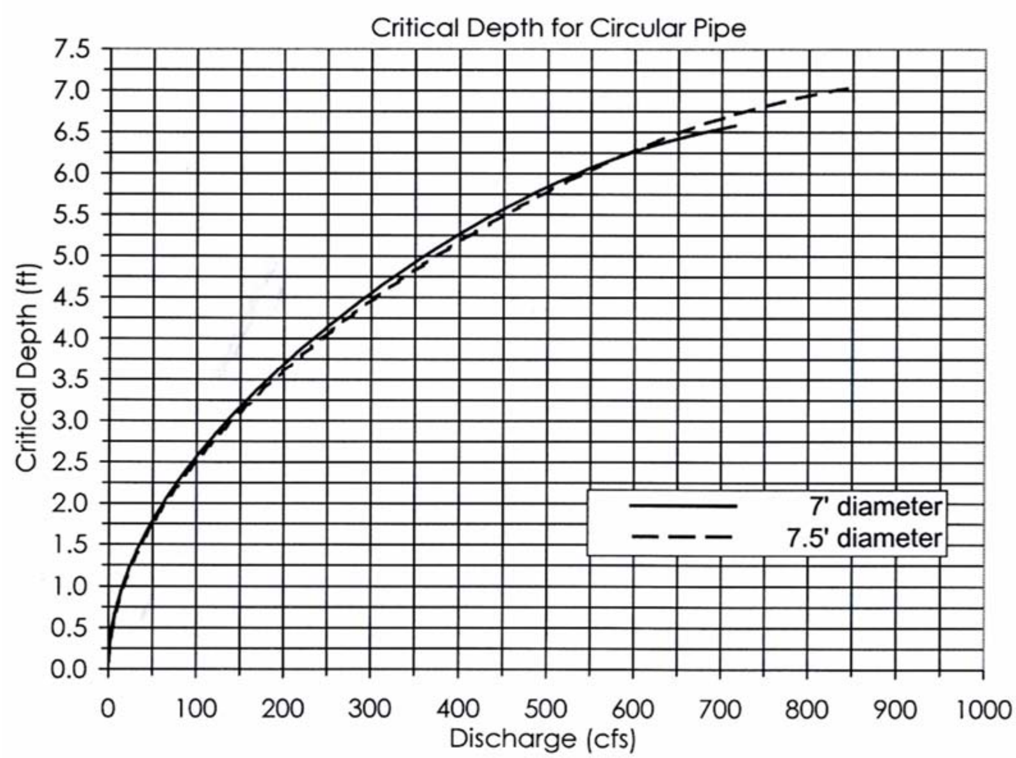


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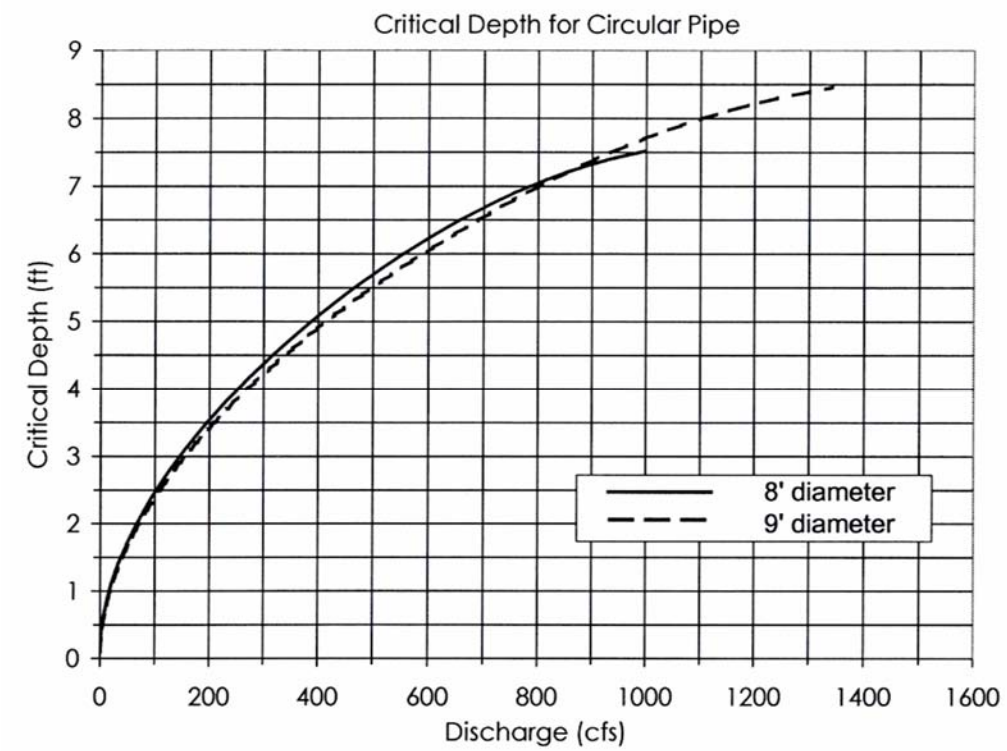
Chart 8-12

Critical Depth for Circular Pipe



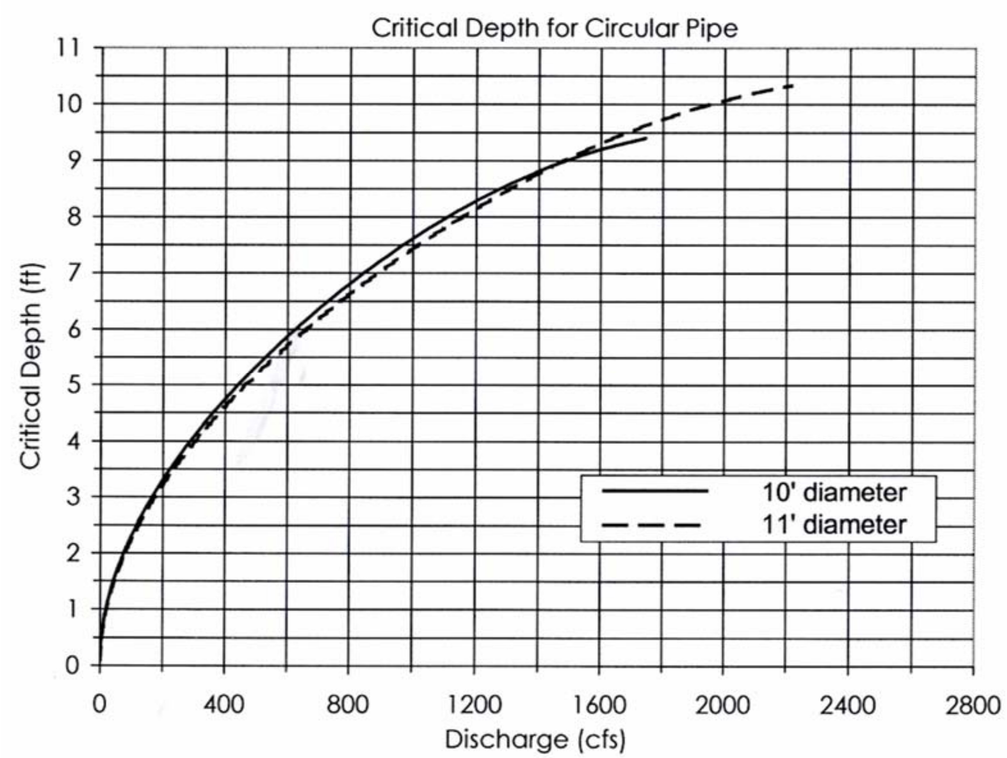


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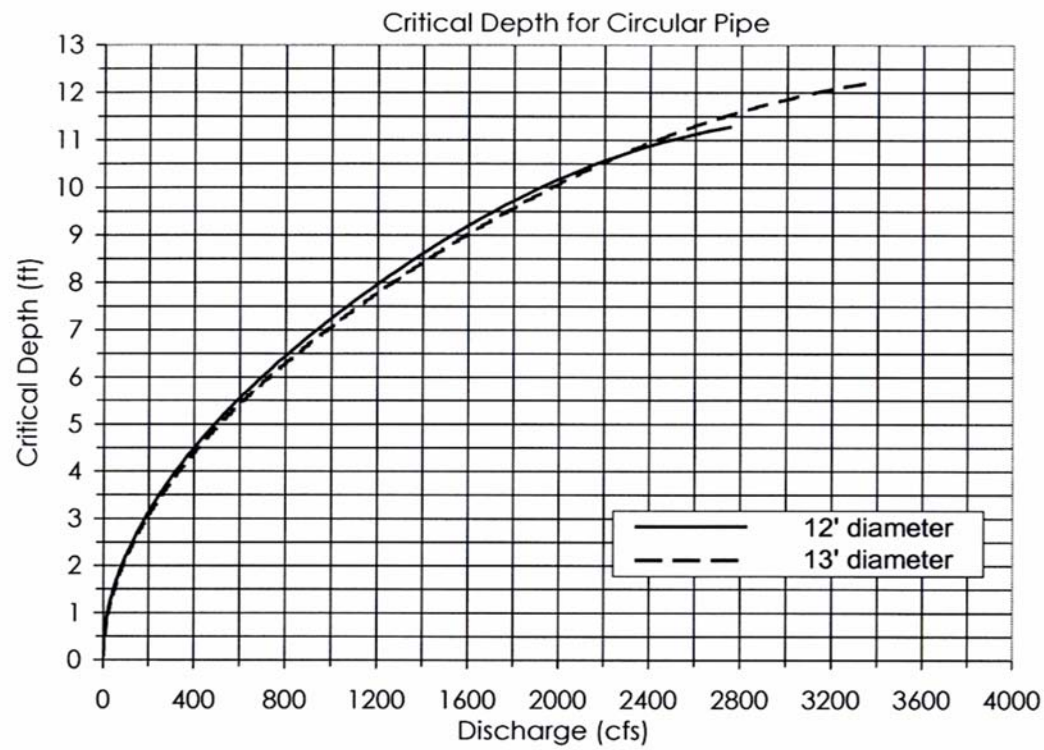
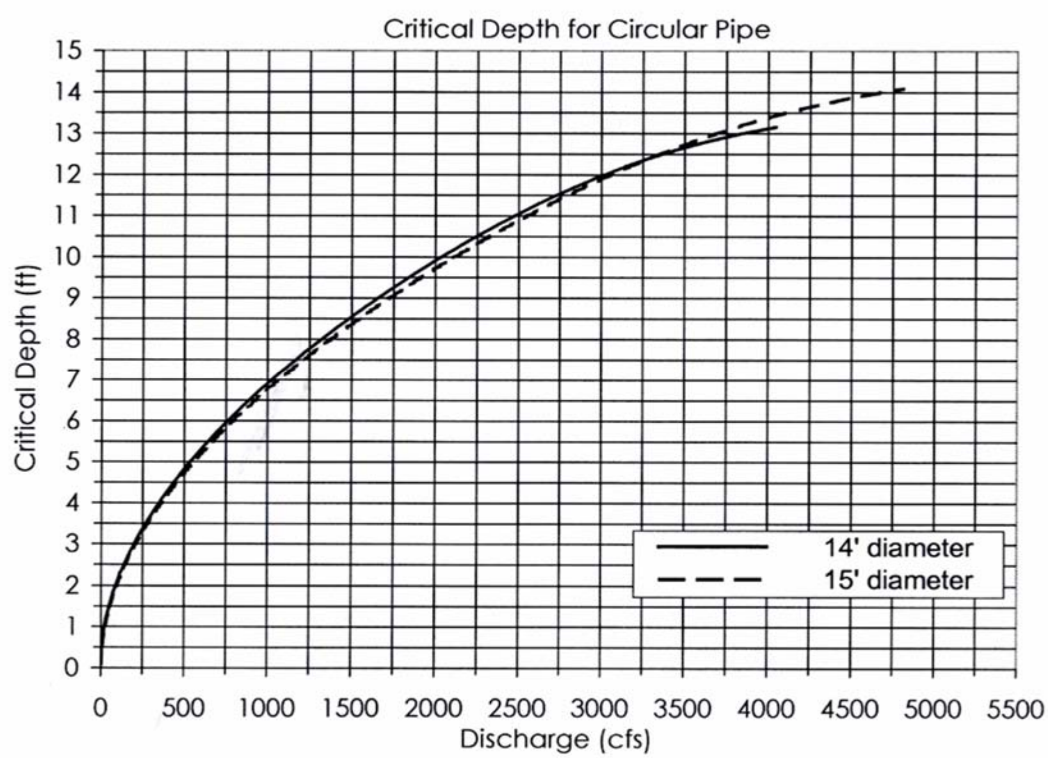
Chart 8-13

Critical Depth for Circular Pipe

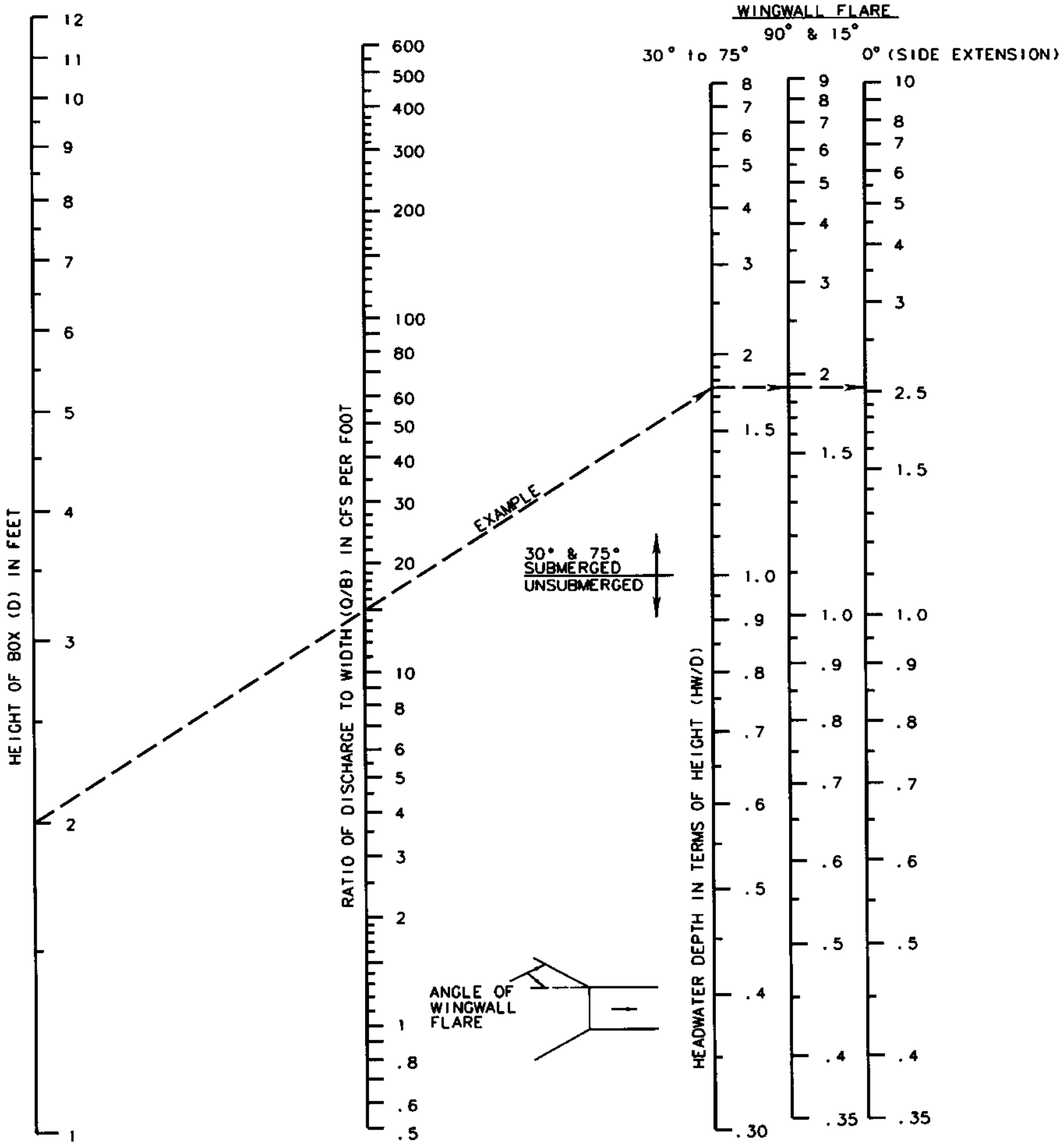




Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-14

Box Culverts with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^2$

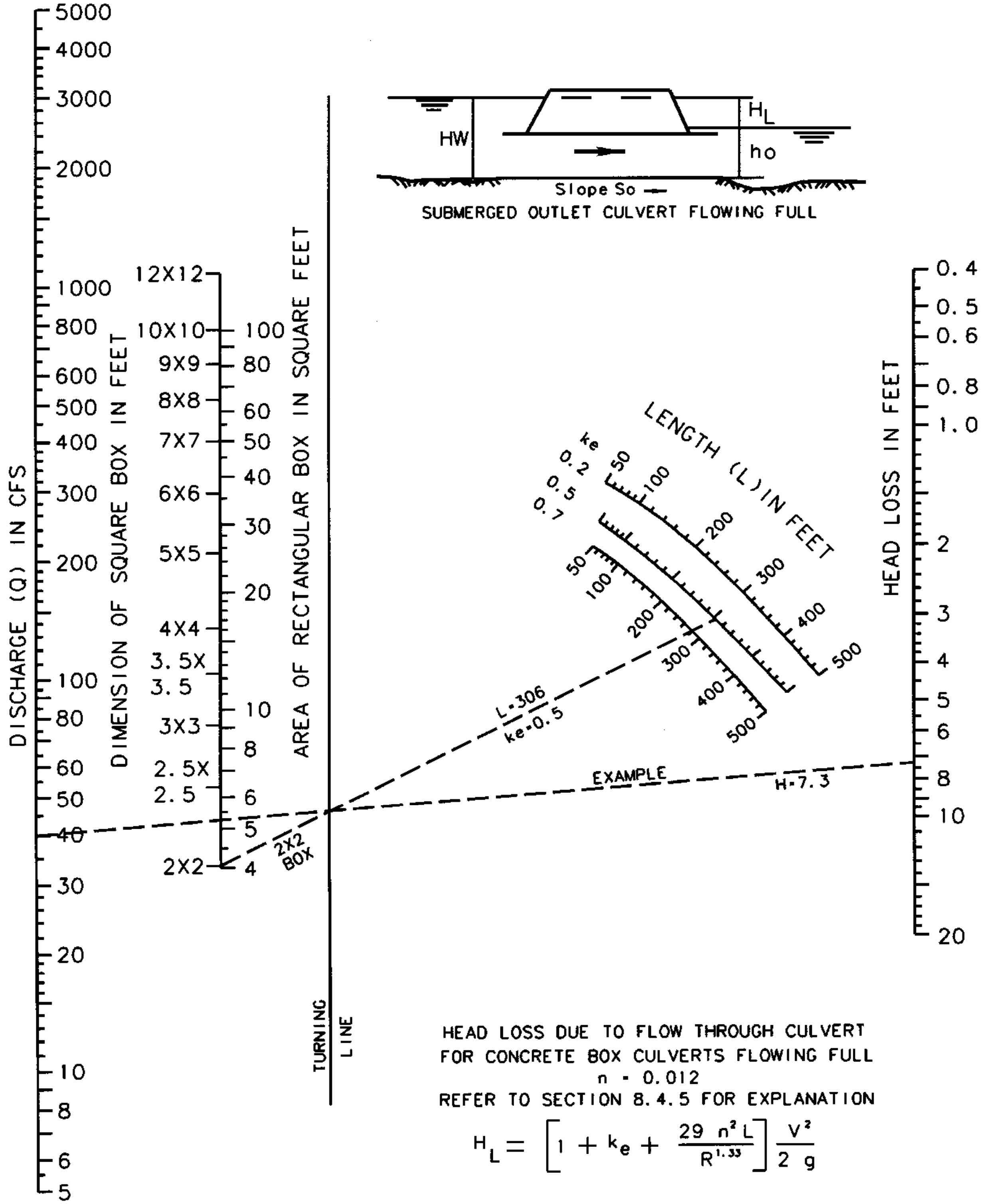
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^2$

SEE TABLE 8-3 AND TABLE 8-4 FOR K, M, c, Y
REFER TO SECTION 8.4.5 FOR EXPLANATION

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-15

Concrete Box Culverts Flowing Full

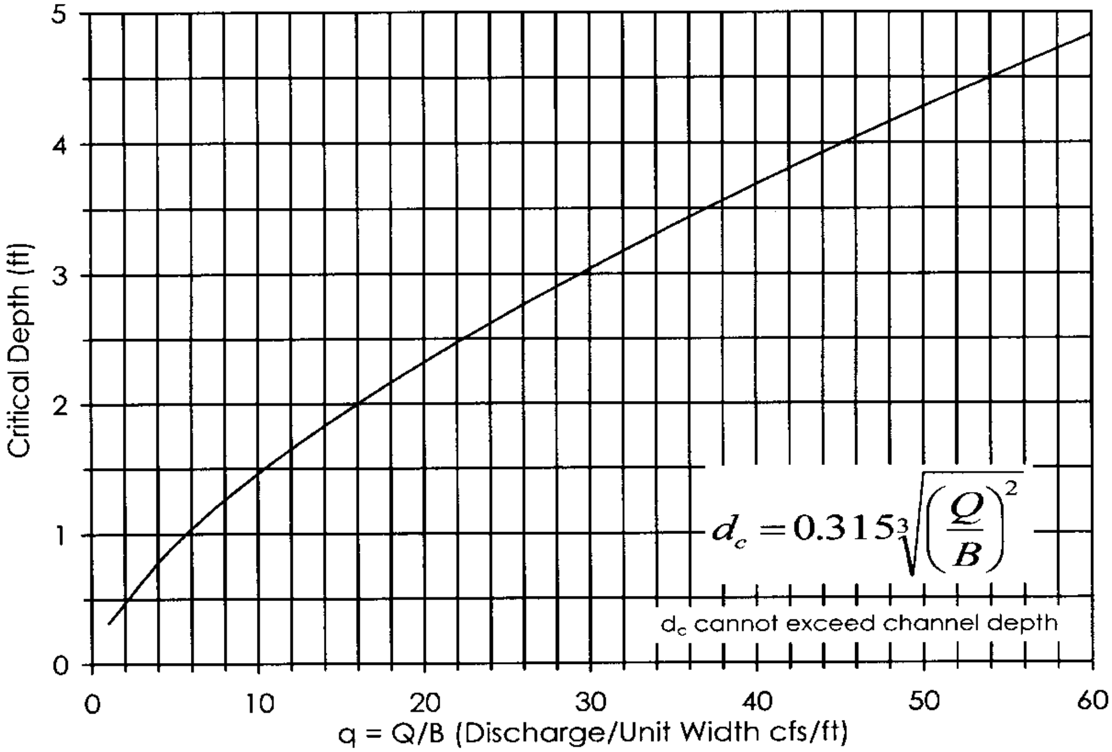


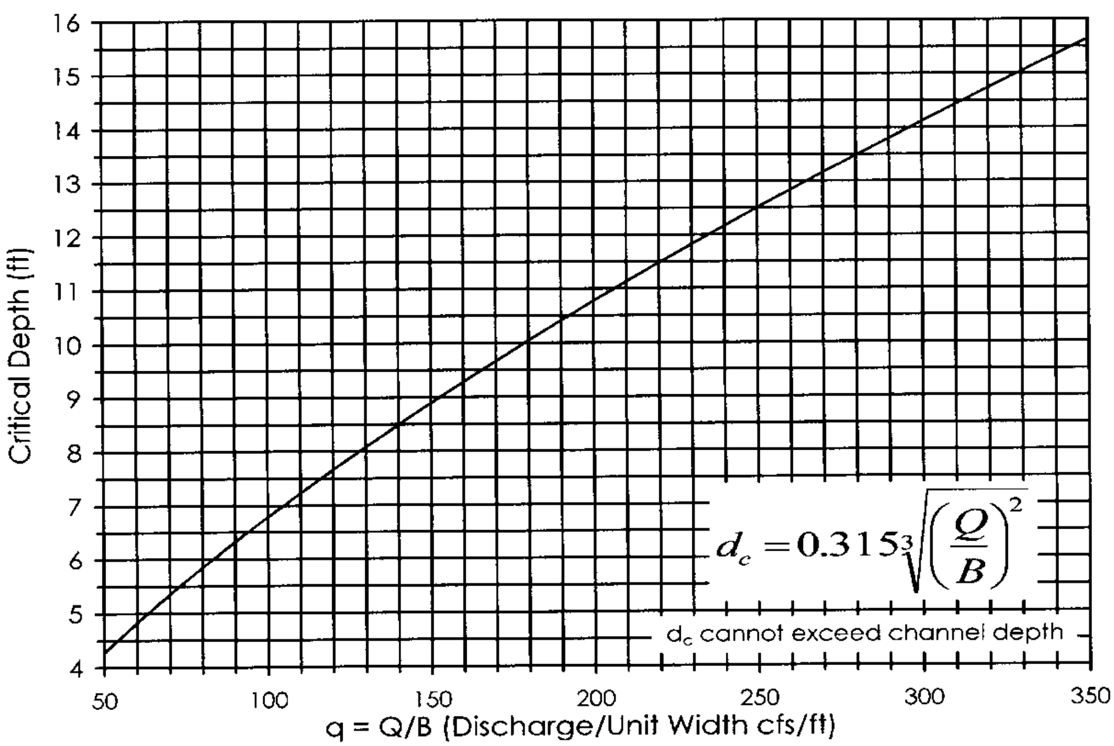
$$H_L = \left[1 + k_e + \frac{29\ n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-16

Critical Depth for Box Culverts

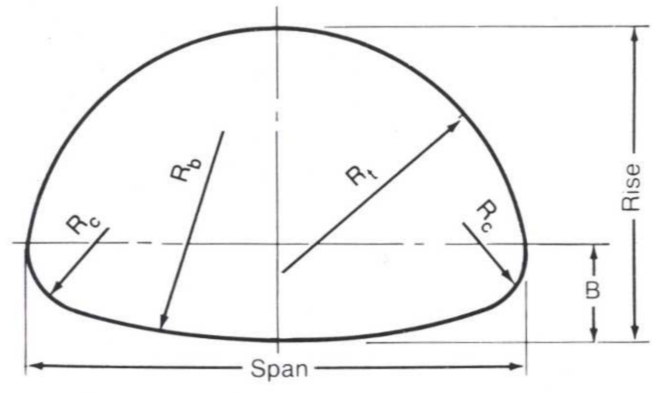




Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-17

Standard Arch Pipe Details

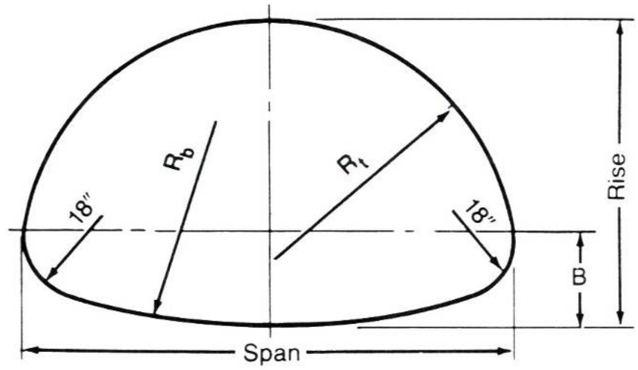


Corrugated Steel Pipe Arch												
NOMINAL SIZE	Rise			Span			Corrugation	Area	B	R_c	R_t	R_b
	in	ft	ft - in	in	ft	ft - in	in x in	ft_2	in	in	in	in
13 x 17	13	1.08	1 - 1.0	17	1.42	1 - 5.0	2 2/3 x 1/2	1.1	4 1/8	3 1/2	8 5/8	25 5/8
15 x 21	15	1.25	1 - 3.0	21	1.75	1 - 9.0	2 2/3 x 1/2	1.6	4 7/8	4 1/8	10 3/4	33 1/8
18 x 24	18	1.50	1 - 6.0	24	2.00	2 - 0.0	2 2/3 x 1/2	2.2	5 7/8	4 7/8	11 7/8	34 5/8
20 x 28	20	1.67	1 - 8.0	28	2.33	2 - 4.0	2 2/3 x 1/2	2.9	6 1/2	5 1/2	14	42 1/4
24 x 35	24	2.00	2 - 0.0	35	2.92	2 - 11.0	2 2/3 x 1/2	4.5	8 1/8	6 7/8	17 7/8	55 1/8
29 x 42	29	2.42	2 - 5.0	42	3.50	3 - 6.0	2 2/3 x 1/2	6.5	9 3/4	8 1/4	21 1/2	66 1/8
33 x 49	33	2.75	2 - 9.0	49	4.08	4 - 1.0	2 2/3 x 1/2	8.9	11 3/8	9 5/8	25 1/8	77 1/4
38 x 57	38	3.17	3 - 2.0	57	4.75	4 - 9.0	2 2/3 x 1/2	11.6	13	11	28 5/8	88 1/4
43 x 64	43	3.58	3 - 7.0	64	5.33	5 - 4.0	2 2/3 x 1/2	14.7	14 5/8	12 3/8	32 1/4	99 1/4
47 x 71	47	3.92	3 - 11.0	71	5.92	5 - 11.0	2 2/3 x 1/2	18.1	16 1/4	13 3/4	35 3/4	110 1/4
52 x 77	52	4.33	4 - 4.0	77	6.42	6 - 5.0	2 2/3 x 1/2	21.9	17 7/8	15 1/8	39 3/8	121 1/4
57 x 83	57	4.75	4 - 9.0	83	6.92	6 - 11.0	2 2/3 x 1/2	26.0	19 1/2	16 1/2	43	132 1/4
41 x 53	41	3.42	3 - 5	53	4.42	4 - 5	3 x 1 or 5 x 1	11.7	15 1/4	10 3/16	28 1/16	73 7/16
46 x 60	48 1/2	4.04	4 - 1/2	58 1/2	4.88	4 - 10 1/2	3 x 1 or 5 x 1	15.6	20 1/2	18 3/4	29 3/8	51 1/8
51 x 66	54	4.50	4 - 6	65	5.42	5 - 5	3 x 1 or 5 x 1	19.3	22 3/4	20 3/4	32 5/8	56 1/4
55 x 73	58 1/4	4.85	4 - 10 1/4	72 1/2	6.04	6 - 1/2	3 x 1 or 5 x 1	23.2	25 1/8	22 7/8	36 3/4	63 3/4
59 x 81	62 1/2	5.21	5 - 2 1/2	79	6.58	6 - 7	3 x 1 or 5 x 1	27.4	23 3/4	20 7/8	39 1/2	82 5/8
63 x 87	67 1/4	5.60	5 - 7 1/4	86 1/2	7.21	7 - 2 1/2	3 x 1 or 5 x 1	32.1	25 3/4	22 5/8	43 3/8	92 1/4
67 x 95	71 3/4	5.98	5 - 11 3/4	93 1/2	7.79	7 - 9 1/2	3 x 1 or 5 x 1	37.0	27 3/4	24 3/8	47	100 1/4
71 x 103	76	6.33	6 - 4	101 1/2	8.46	8 - 5 1/2	3 x 1 or 5 x 1	42.4	29 3/4	26 1/8	51 1/4	111 5/8
75 x 112	80 1/2	6.71	6 - 8 1/2	108 1/2	9.04	9 - 1/2	3 x 1 or 5 x 1	48.0	31 5/8	27 3/4	54 7/8	120 1/4
79 x 117	84 3/4	7.06	7 - 3/4	116 1/2	9.71	9 - 8 1/2	3 x 1 or 5 x 1	54.2	33 5/8	29 1/2	59 3/8	131 3/4
83 x 128	89 1/4	7.44	7 - 5 1/4	123 1/2	10.29	10 - 3 1/2	3 x 1 or 5 x 1	60.5	35 5/8	31 1/4	63 1/4	139 3/4
87 x 137	93 3/4	7.81	7 - 9 3/4	131	10.92	10 - 11	3 x 1 or 5 x 1	67.4	37 5/8	33	67 3/8	149 1/2
91 x 142	98	8.17	8 - 2	138 1/2	11.54	11 - 6 1/2	3 x 1 or 5 x 1	74.5	39 1/2	34 3/4	71 5/8	162 3/8
96 x 150	102	8.50	8 - 6	146	12.17	12 - 2	3 x 1 or 5 x 1	81.0	41	36	76	172
101 x 157	107	8.92	8 - 11	153	12.75	12 - 9	3 x 1 or 5 x 1	89.0	43	38	80	180
105 x 164	113	9.42	9 - 5	159	13.25	13 - 3	3 x 1 or 5 x 1	98.0	45	40	82	184
110 x 171	118 1/2	9.88	9 - 10 1/2	165	13.75	13 - 9	3 x 1 or 5 x 1	107.0	47	41	85	190

Source: Handbook of Steel Drainage Products, American Iron & Steel Inst., 1994

Chart 8-18

Arch Pipe with 18 inch Corner Radius Details

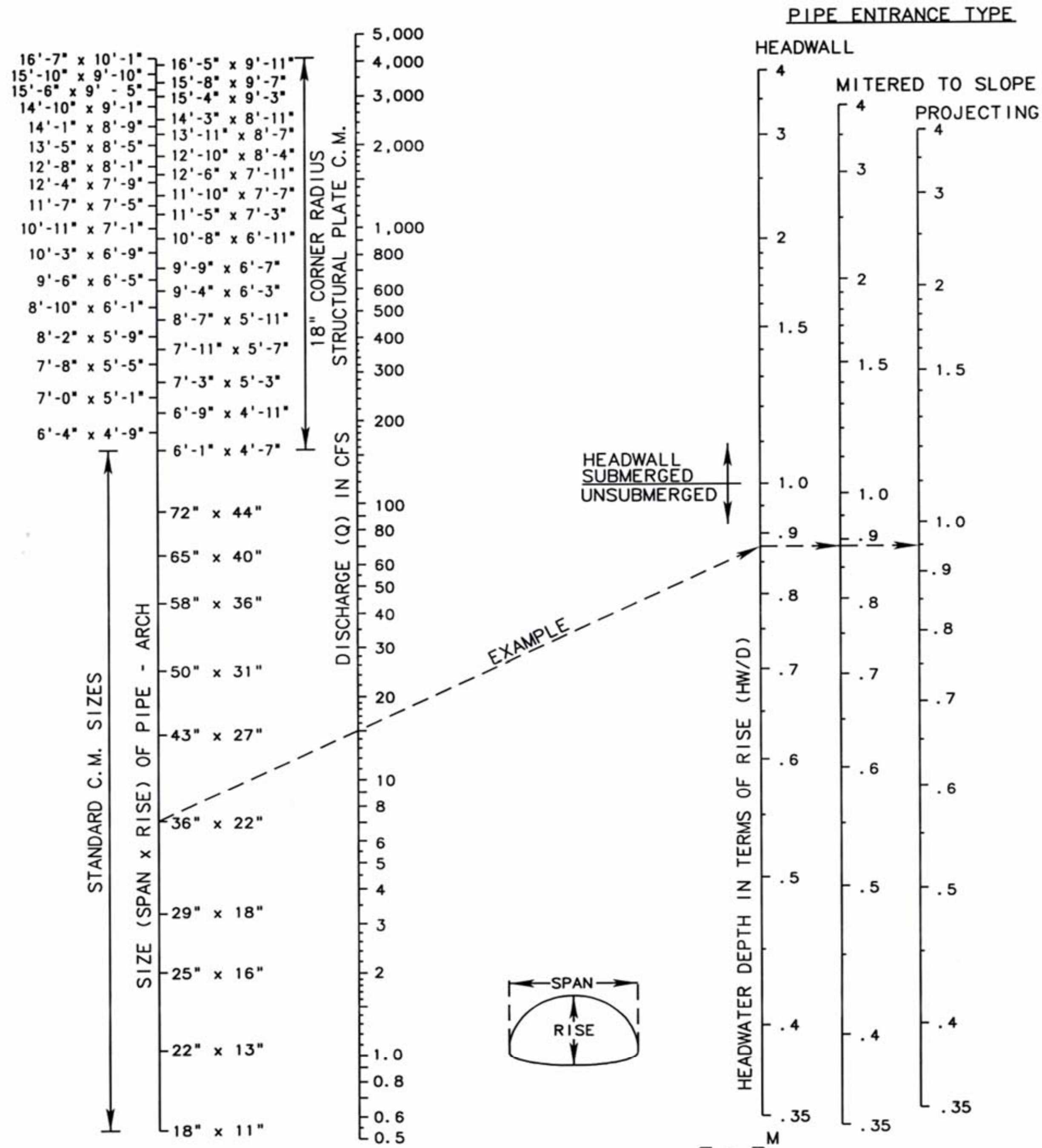


Corrugated Steel Pipe Arch 18 inch corner radius											
Rise			Span			Corrugation	Area	B	R_c	R_t	R_b
in	ft	ft - in	in	ft	ft - in	in x in	ft_2	in	in	ft	ft
55	4.58	4 - 7.0	73	6.08	6 - 1.0	6 x 2	22	21.0	18	3.07	6.36
57	4.75	4 - 9.0	76	6.33	6 - 4.0	6 x 2	24	20.5	18	3.18	8.22
59	4.92	4 - 11.0	81	6.75	6 - 9.0	6 x 2	26	22.0	18	3.42	6.96
61	5.08	5 - 1.0	84	7.00	7 - 0.0	6 x 2	28	21.4	18	3.53	8.68
63	5.25	5 - 3.0	87	7.25	7 - 3.0	6 x 2	31	20.8	18	3.63	11.35
65	5.42	5 - 5.0	92	7.67	7 - 8.0	6 x 2	33	22.4	18	3.88	9.15
67	5.58	5 - 7.0	95	7.92	7 - 11.0	6 x 2	35	21.7	18	3.98	11.49
69	5.75	5 - 9.0	98	8.17	8 - 2.0	6 x 2	38	20.9	18	4.08	15.24
71	5.92	5 - 11.0	103	8.58	8 - 7.0	6 x 2	40	22.7	18	4.33	11.75
73	6.08	6 - 1.0	106	8.83	8 - 10.0	6 x 2	43	21.8	18	4.42	14.89
75	6.25	6 - 3.0	112	9.33	9 - 4.0	6 x 2	46	23.8	18	4.68	12.05
77	6.42	6 - 5.0	114	9.50	9 - 6.0	6 x 2	49	22.9	18	4.78	14.79
79	6.58	6 7.0	117	9.75	9 9.0	6 x 2	52	21.9	18	4.86	18.98
81	6.75	6 - 9	123	10.25	10 - 3	6 x 2	55	23.9	18	5.13	14.86
83	6.92	6 - 11	128	10.67	10 - 8	6 x 2	58	26.1	18	5.41	12.77
85	7.08	7 - 1	131	10.92	10 - 11	6 x 2	61	25.1	18	5.49	15.03
87	7.25	7 - 3	137	11.42	11 - 5	6 x 2	64	27.4	18	5.78	13.16
89	7.42	7 - 5	139	11.58	11 - 7	6 x 2	67	26.3	18	5.85	15.27
91	7.58	7 - 7	142	11.83	11 - 10	6 x 2	71	25.2	18	5.93	18.03
93	7.75	7 - 9	148	12.33	12 - 4	6 x 2	74	27.5	18	6.23	15.54
95	7.92	7 - 11	150	12.50	12 - 6	6 x 2	78	26.4	18	6.29	18.07
97	8.08	8 - 1	152	12.67	12 - 8	6 x 2	81	25.2	18	6.37	21.45
100	8.33	8 - 4	154	12.83	12 - 10	6 x 2	85	24.0	18	6.44	26.23
101	8.42	8 - 5	161	13.42	13 - 5	6 x 2	89	26.3	18	6.73	21.23
103	8.58	8 - 7	167	13.92	13 - 11	6 x 2	93	28.9	18	7.03	18.39
105	8.75	8 - 9	169	14.08	14 - 1	6 x 2	97	27.6	18	7.09	21.18
107	8.92	8 - 11	171	14.25	14 - 3	6 x 2	101	26.3	18	7.16	24.80
109	9.08	9 - 1	178	14.83	14 - 10	6 x 2	105	28.9	18	7.47	21.19
111	9.25	9 - 3	184	15.33	15 - 4	6 x 2	109	31.6	18	7.78	18.90
113	9.42	9 - 5	186	15.50	15 - 6	6 x 2	113	30.2	18	7.83	21.31
115	9.58	9 7	188	15.67	15 8	6 x 2	118	28.8	18	7.89	24.29
118	9.83	9 10	190	15.83	15 10	6 x 2	122	27.4	18	7.96	28.18
119	9.92	9 11	197	16.42	16 5	6 x 2	126	30.1	18	8.27	24.24
121	10.08	10 1	199	16.58	16 7	6 x 2	131	28.7	18	8.33	27.73

Source: Handbook of Steel Drainage Products, American Iron & Steel Inst., 1994

Chart 8-19

Standard C.M. Pipe Arch Culverts with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^2$

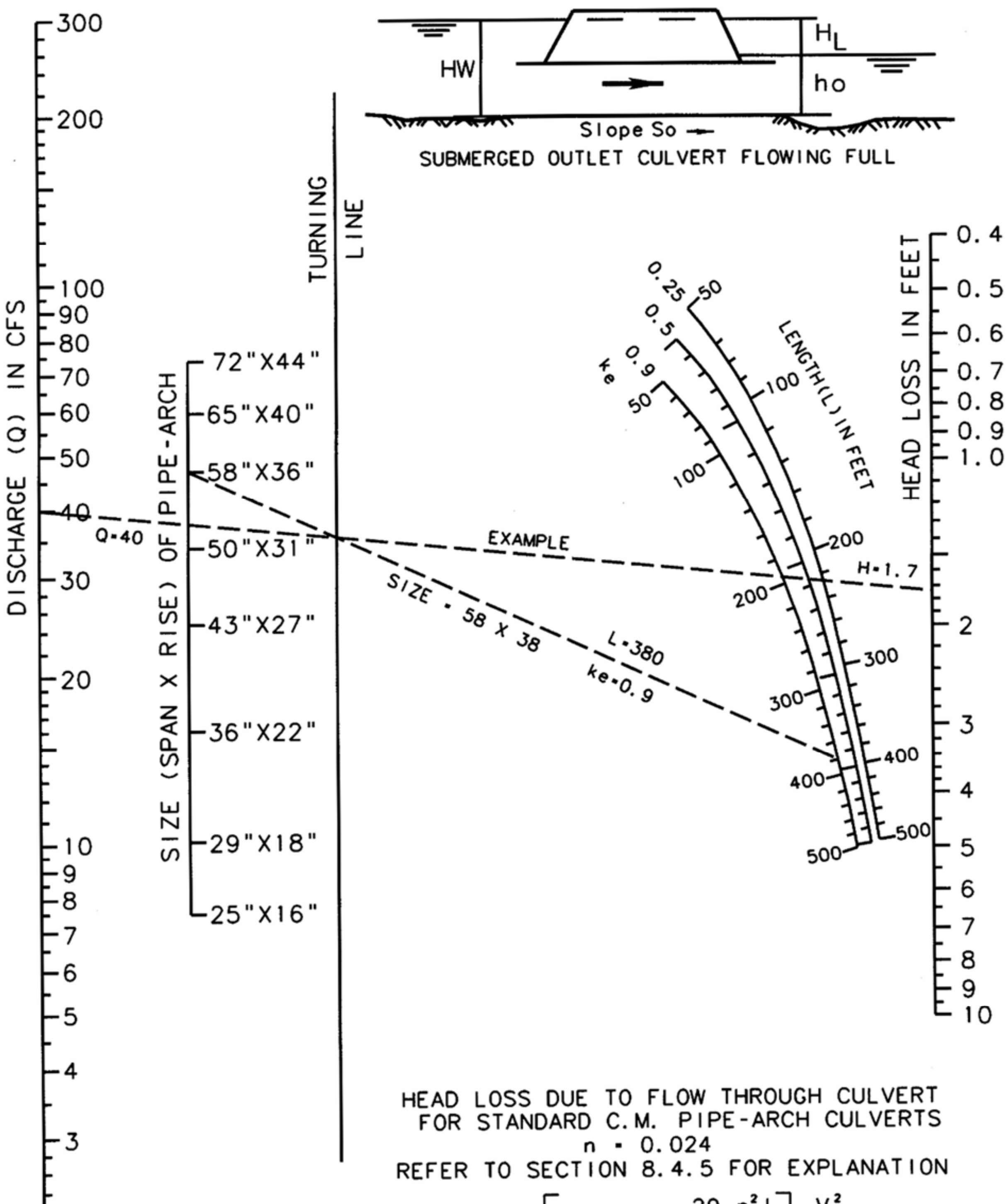
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^2$

SEE TABLE 8-3 AND TABLE 8-4 FOR K, M, c, Y
REFER TO SECTION 8.4.5 FOR EXPLANATION

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-20

Standard C.M. Pipe Arch Flowing Full

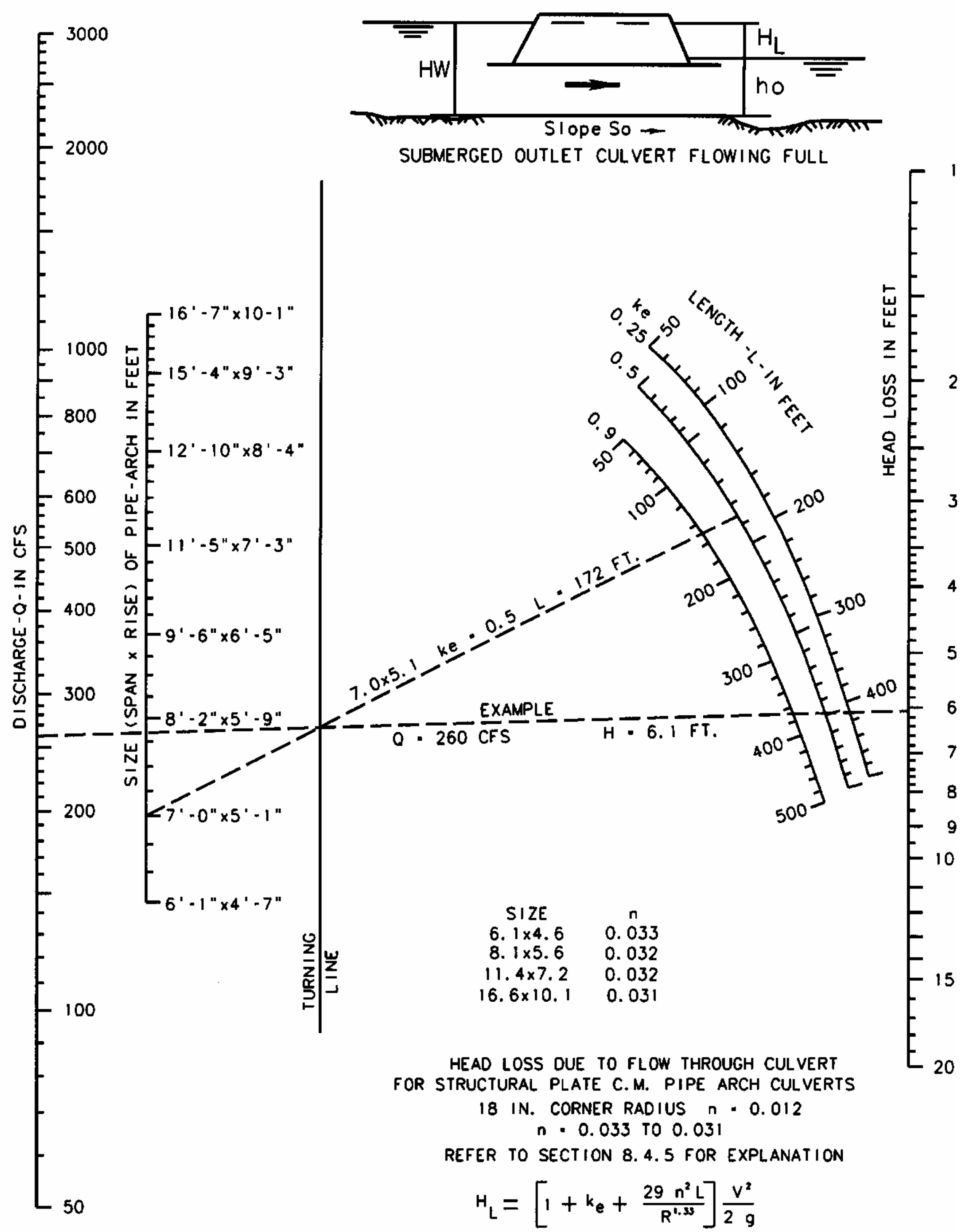


$$H_L = \left[1 + k_e + \frac{29\ n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-21

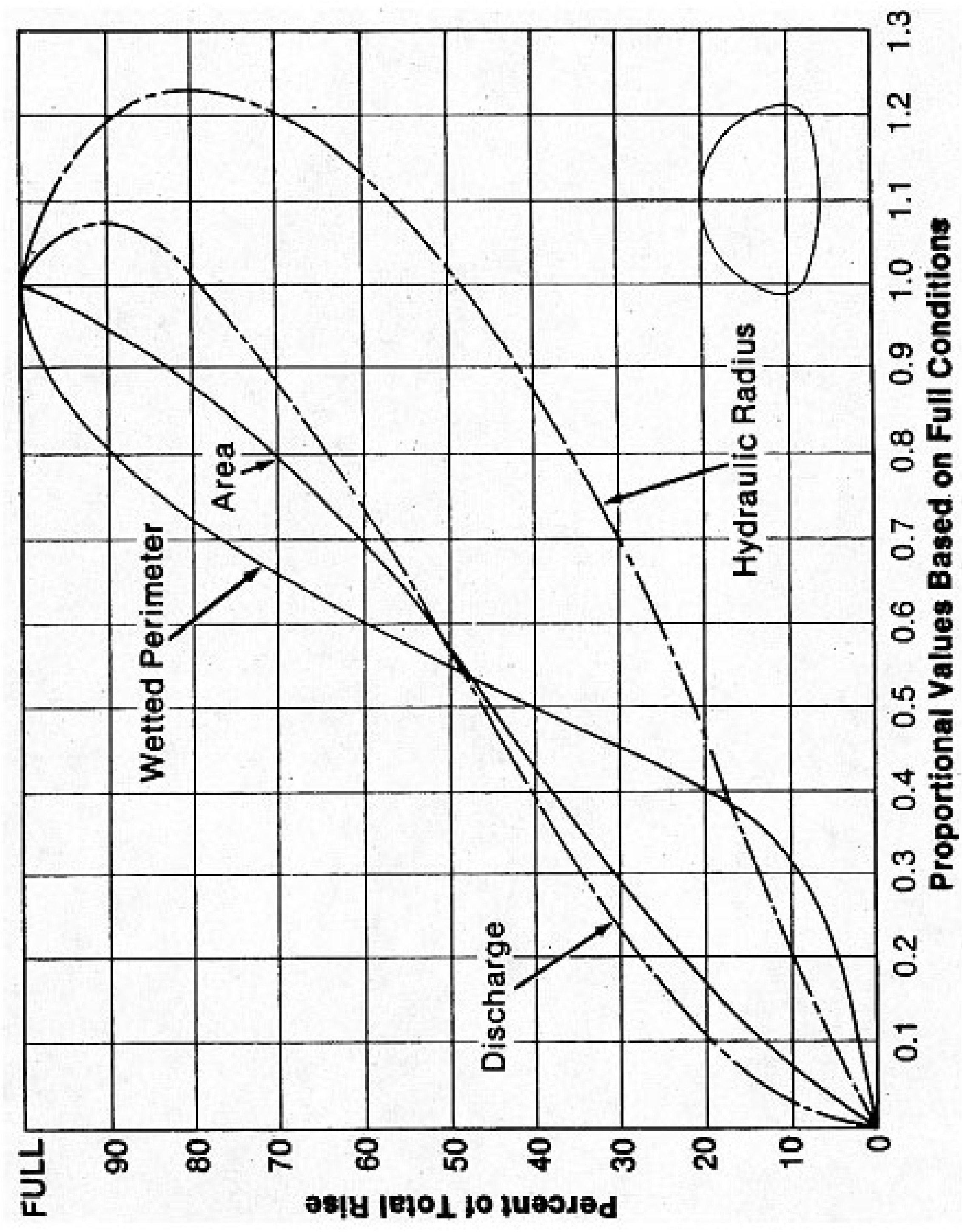
Str. Plate Pipe Arch (18” corner rad.) Flowing Full



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-22

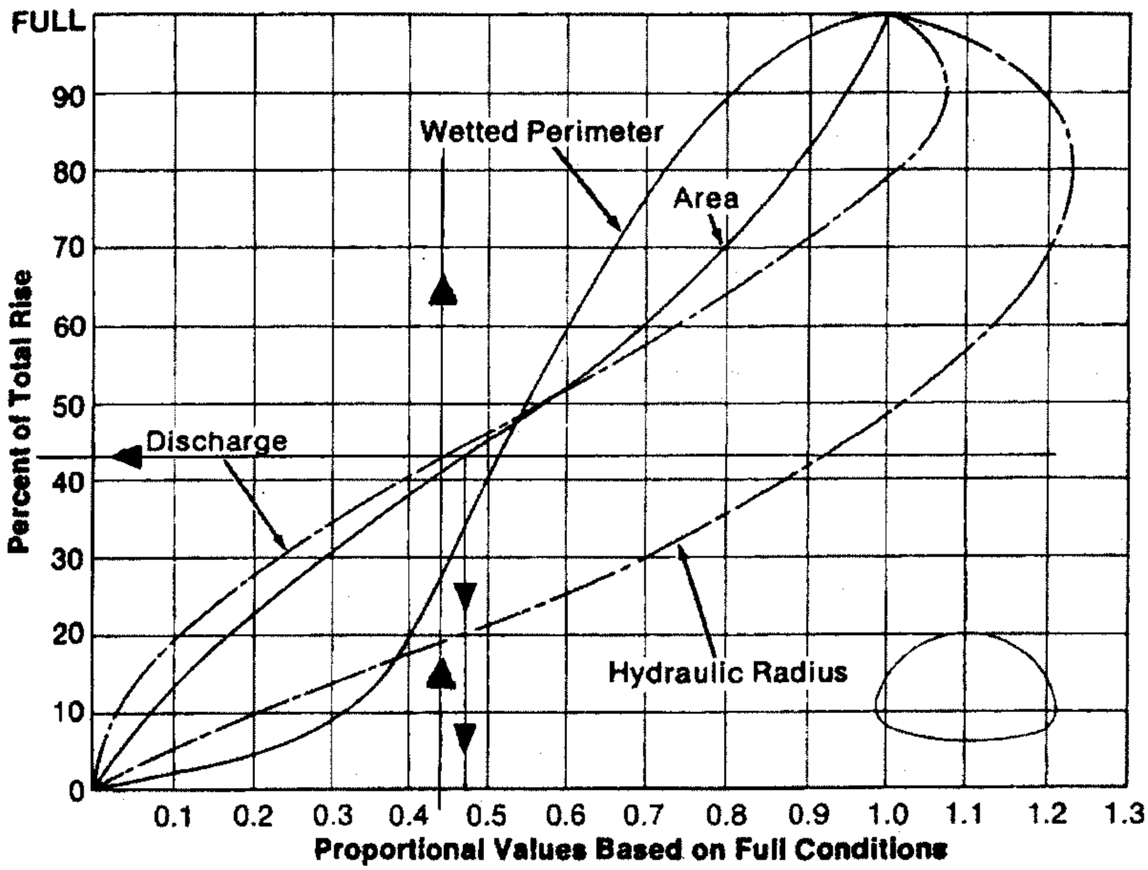
Hydraulic Elements for Partially Full C.M. Pipe Arch



Source: Handbook of Steel Drainage Products, American Iron & Steel Inst., 1994

Chart 8-23

Hydraulic Elements for Partially Full C.M. Pipe Arch Example



24 X 35 pipe, n = 0.024, Slope = 0.02, Q_{full} /S $^{0.5}$ = 192 (Table 8-5)

Given: D_{full} = 24 in, Q = 12 cfs

$Q_{full} = 192*(0.02)^{0.5} = 27.2$ cfs

Required: depth at Q, velocity at Q

Solution: Q / Q_{full} = 12/27.2 = 0.44

From figure for hydraulic elements, d / d_{full} = 0.435

d = d_{full} x 0.435 = 10.4 in

From figure for hydraulic elements, A / A_{full} = 0.465 (A_{full} from Table 8-5)

A = A_{full} x 0.465 = 2.1 ft^2

V = Q / A = 12 / 2.1 = 5.7 ft/s

Table 8-5

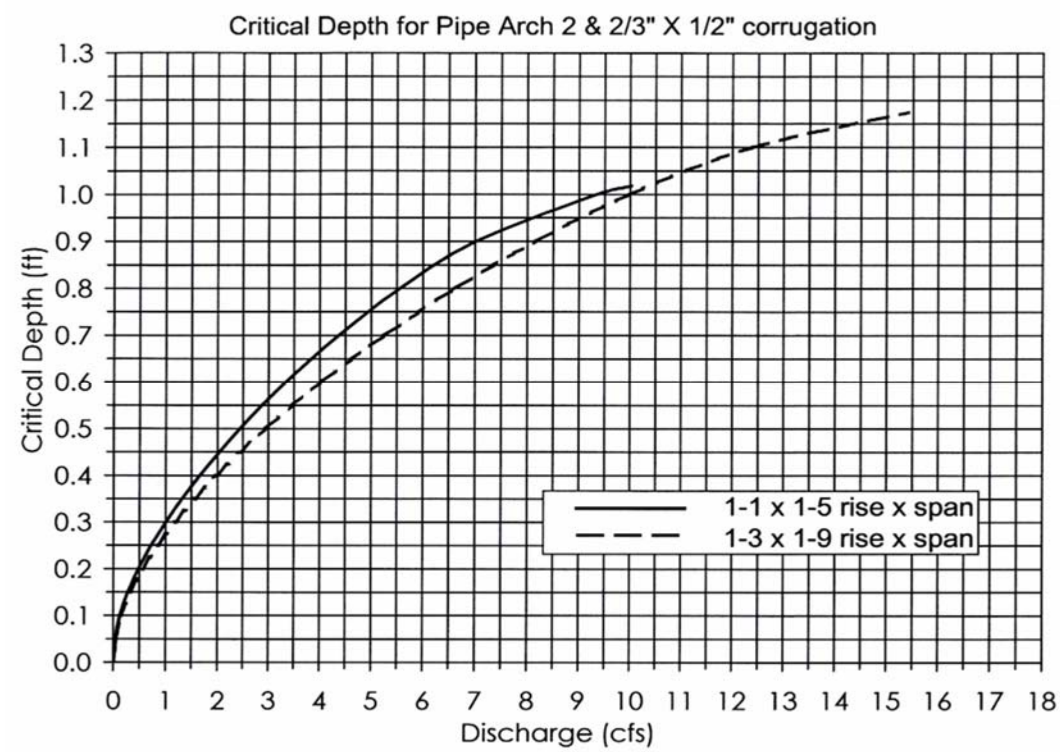
Full Flow Values for C.M. Pipe Arch

Pipe Size Rise X Span	Corrugations	Equivalent Full Flow Area for Circular Diameter inches	A Area ft^2	R Hydraulic Radius ft	Value of Full Flow Conveyance K $\frac{Q}{\sqrt{S}} = \frac{1.486}{n} A R^{\frac{2}{3}}$ n = 0.024	n = 0.027	n = 0.033
13" X 17"	2 2/3 X 1/2	15	1.1	0.280	29	26	21
15" X 21"	2 2/3 X 1/2	18	1.6	0.340	48	43	35
18" X 24"	2 2/3 X 1/2	21	2.2	0.400	74	66	54
20" X 28"	2 2/3 X 1/2	24	2.9	0.462	107	95	78
24" X 35"	2 2/3 X 1/2	30	4.5	0.573	192	171	140
29" X 42"	2 2/3 X 1/2	36	6.5	0.690	314	279	229
33" X 49"	2 2/3 X 1/2	42	8.9	0.810	479	426	348
38" X 57"	2 2/3 X 1/2	48	11.6	0.924	681	606	496
43" X 64"	2 2/3 X 1/2	54	14.7	1.040	934	830	679
46" X 60"	3 X 1	54	15.6	1.104	1032	917	750
51" X 66"	3 X 1	60	19.3	1.230	1372	1219	998
55" X 73"	3 X 1	66	23.2	1.343	1749	1554	1272
59" X 81"	3 X 1	72	27.4	1.454	2177	1935	1584
63" X 87"	3 X 1	78	32.1	1.573	2688	2390	1955
67" X 95"	3 X 1	84	37.0	1.683	3241	2881	2357
71" X 103"	3 X 1	90	42.4	1.800	3885	3453	2825
75" X 112"	3 X 1	96	48.0	1.911	4577	4068	3329
4'-7" X 6'-1"	6 X 2		22.1	1.298	1628	1447	1184
5'-1" X 7'-0"	6 X 2		28.4	1.463	2266	2014	1648
5'-5" X 7'-8"	6 X 2		32.9	1.565	2746	2441	1997
5'-9" X 8'-2"	6 X 2		37.7	1.670	3286	2921	2390
6'-1" X 8'-10"	6 X 2		42.9	1.776	3895	3463	2833
6'-5" X 9'-6"	6 X 2		48.5	1.881	4576	4067	3328
6'-7" X 9'-9"	6 X 2		51.2	1.930	4914	4368	3574
7'-3" X 11'-5"	6 X 2		64.0	2.145	6591	5859	4793
8'-1" X 12'-8"	6 X 2		81.0	2.390	8965	7969	6520
8'-4" X 12'-10"	6 X 2		85.5	2.465	9660	8587	7026

Source: American Concrete Pipe Association Publication

Chart 8-24

Critical Depth for Pipe Arch 2-2/3 x 1/2 in. Corrugation



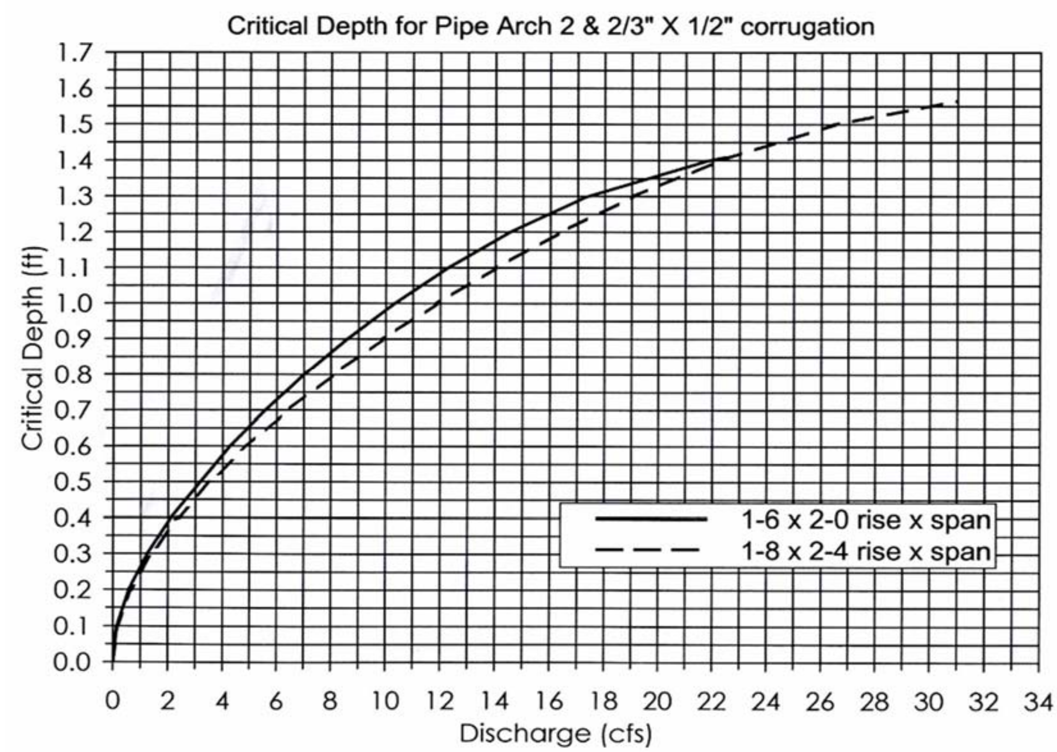


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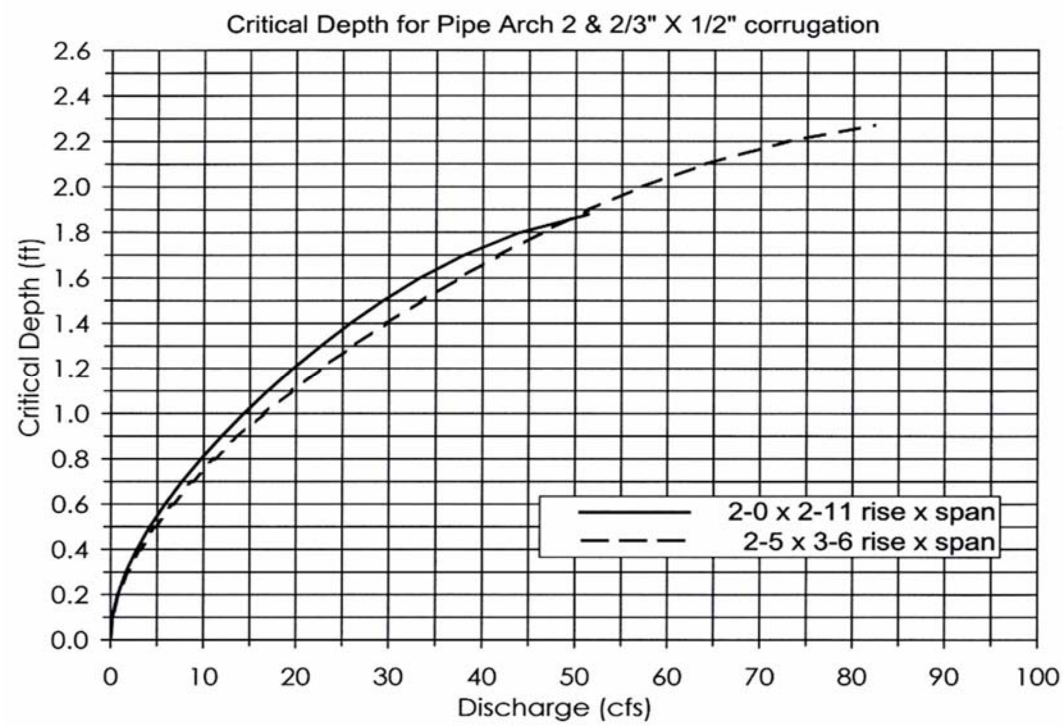
Chart 8-25

Critical Depth for Pipe Arch 2-2/3 x 1/2 in. Corrugation



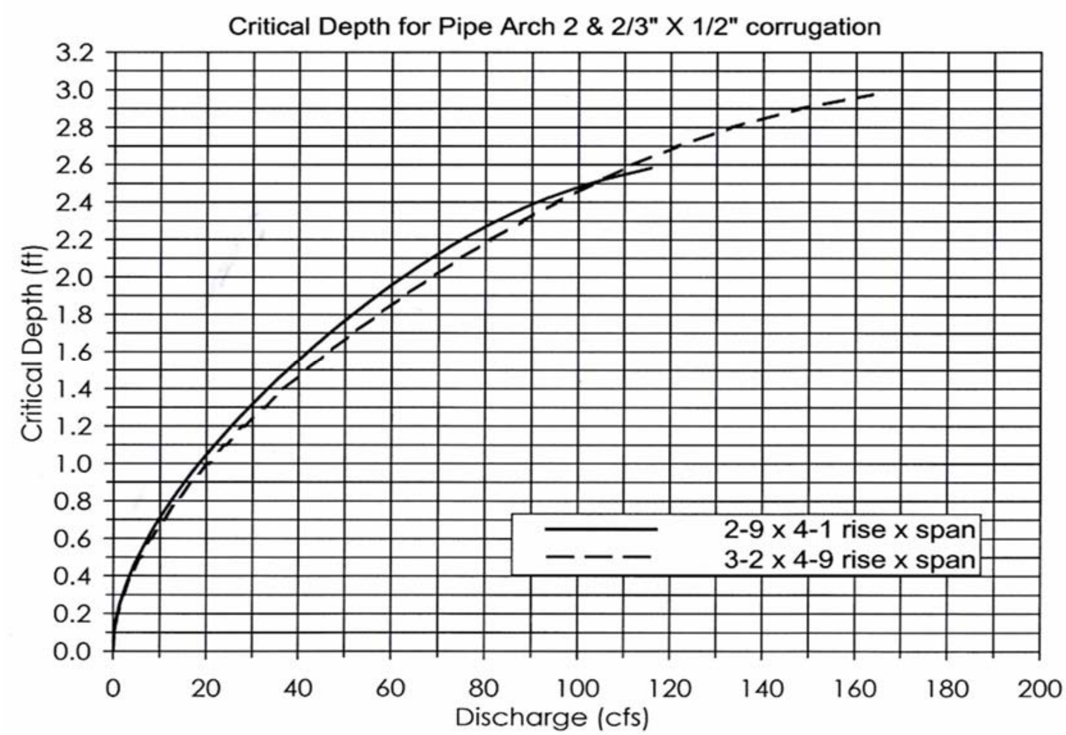


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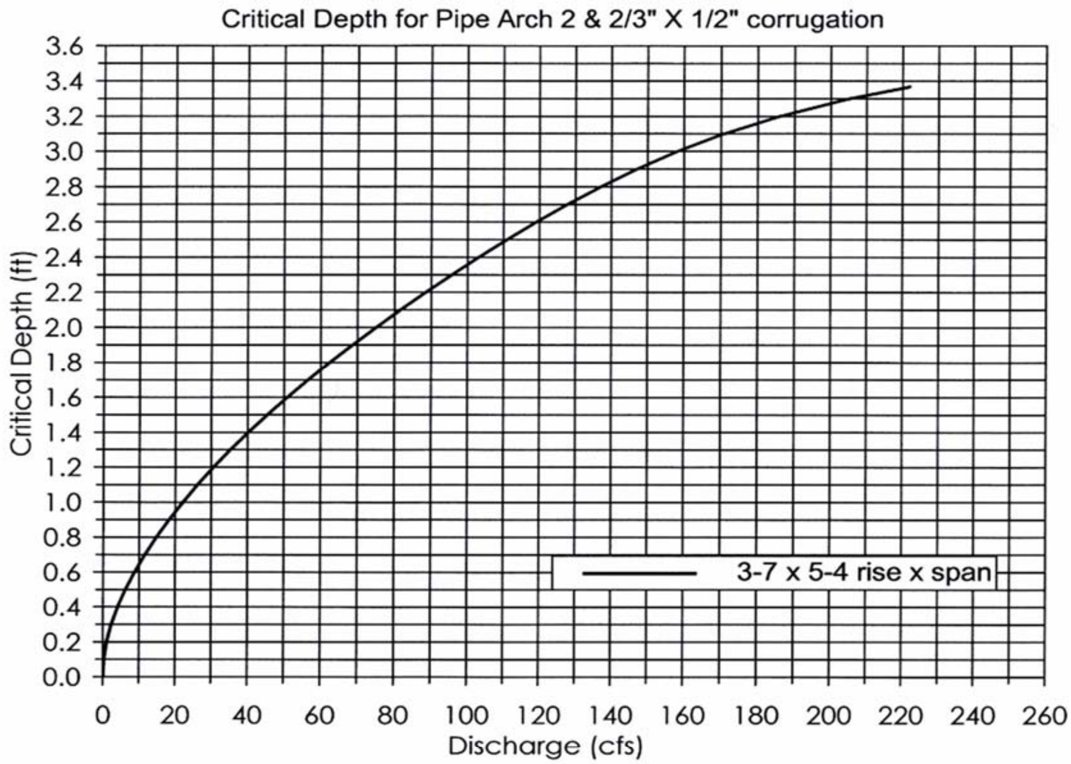
Chart 8-26

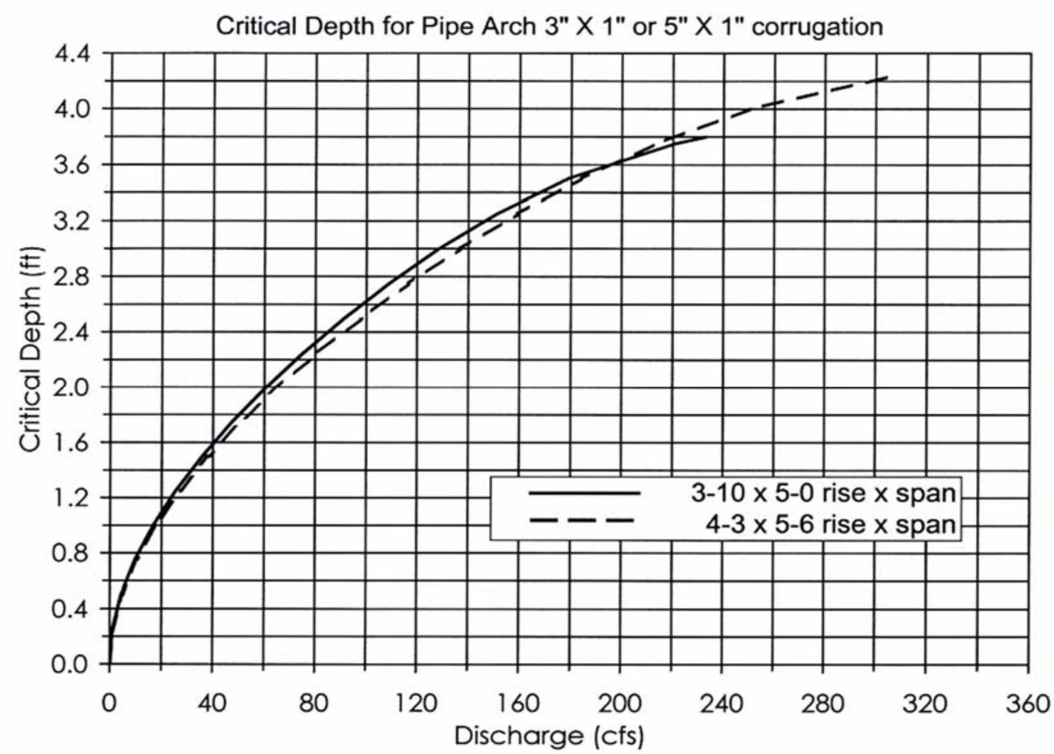
Critical Depth for Pipe Arch 2-2/3 x 1/2 in. Corrugation



Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-27

Critical Depth for Pipe Arch 3 x 1 or 5 x 1 in. Corrugation



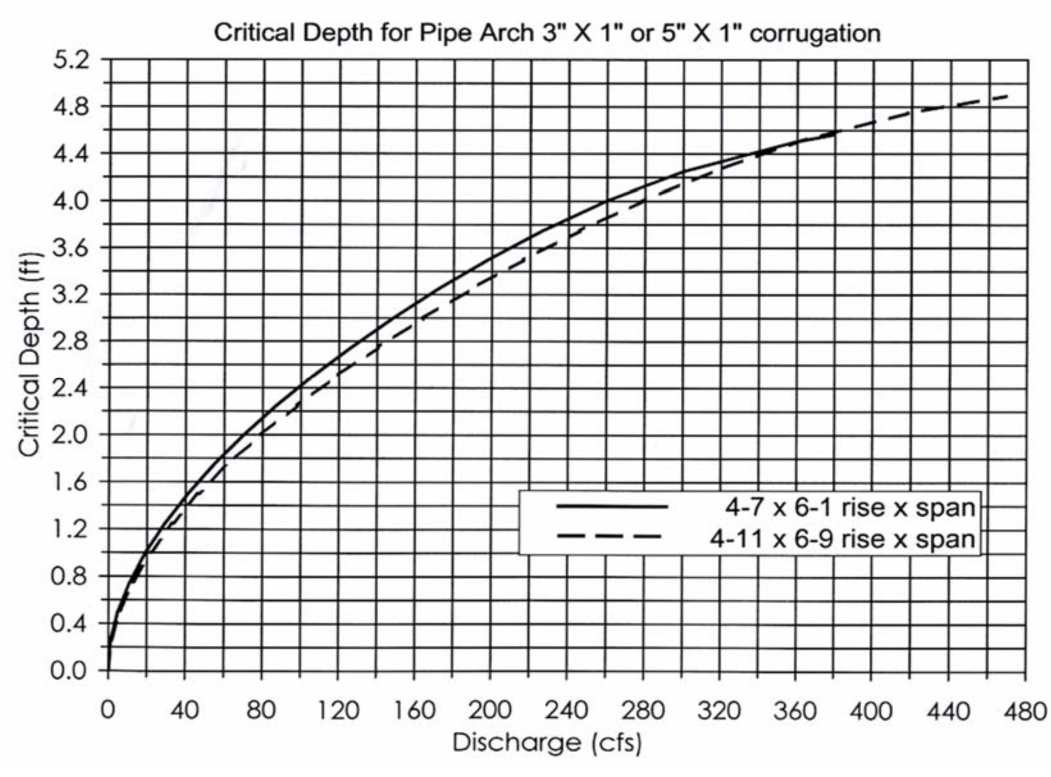


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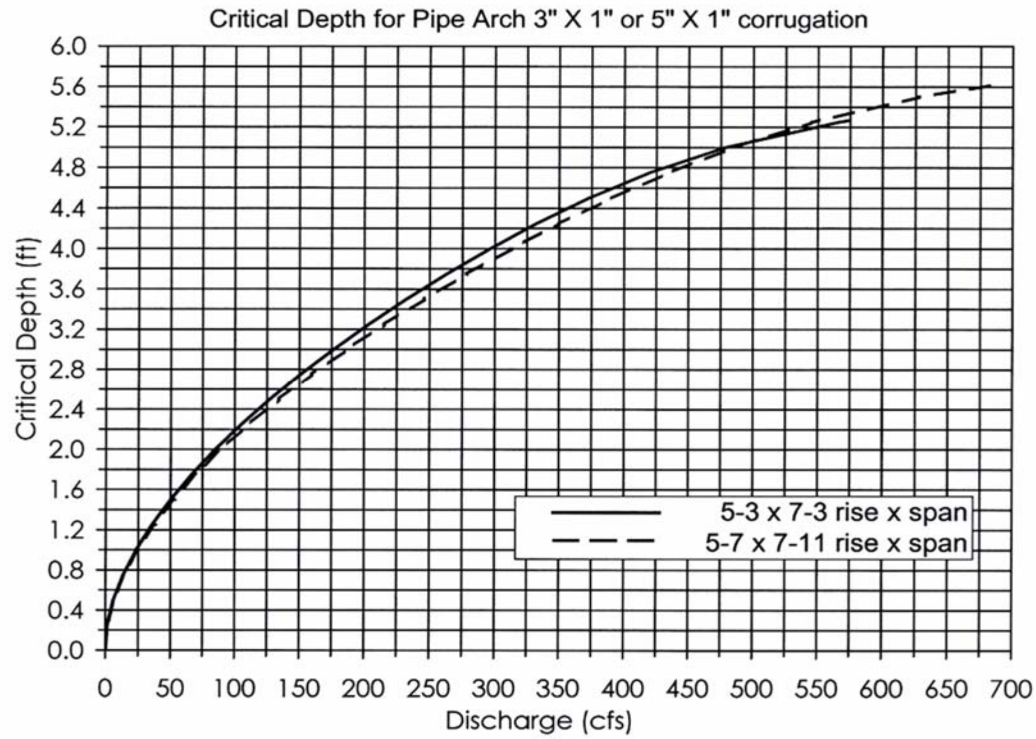
Chart 8-28

Critical Depth for Pipe Arch 3 x 1 or 5 x 1 in. Corrugation



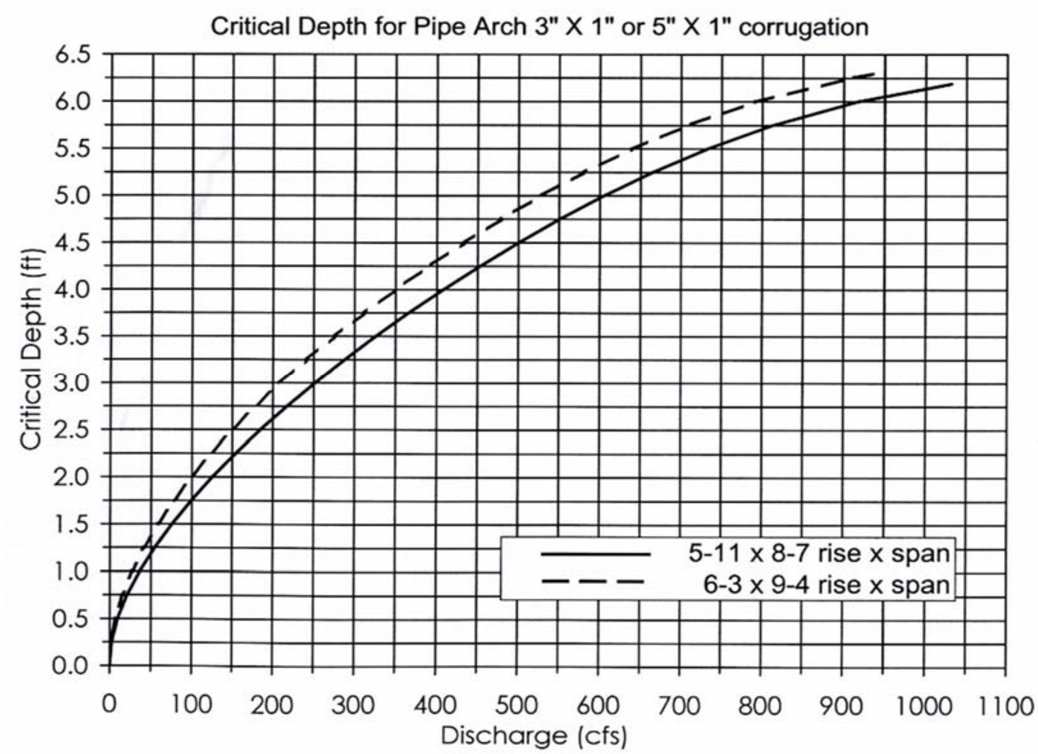


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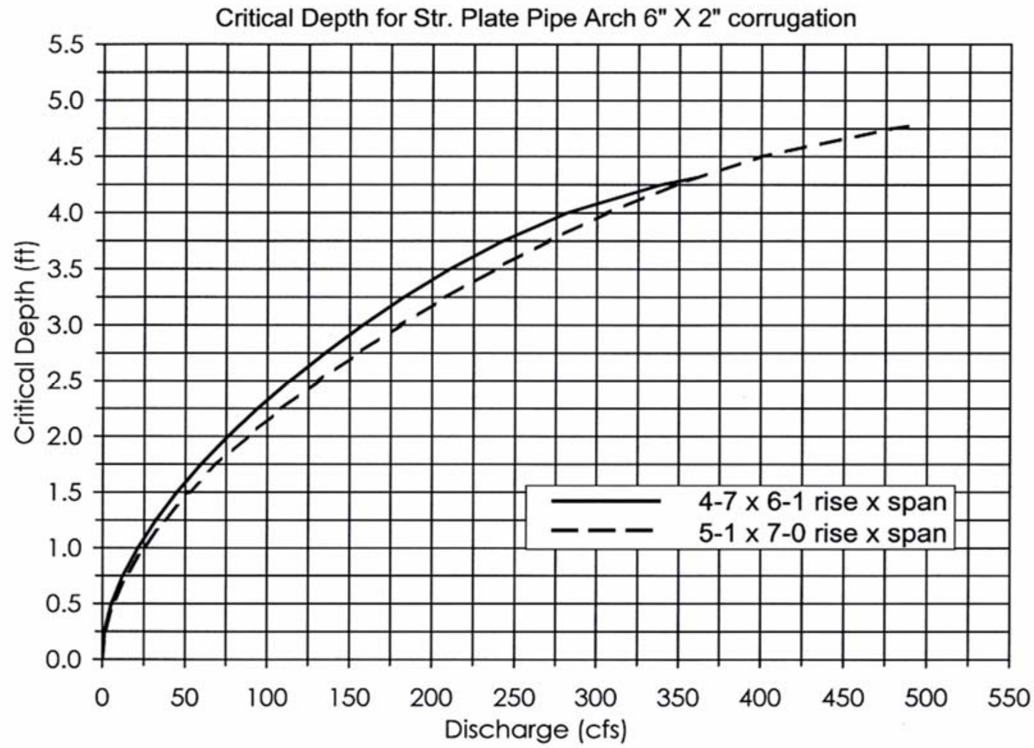
Chart 8-29

Critical Depth Str. Plate Pipe Arch 6 x 2 in. Corrugation



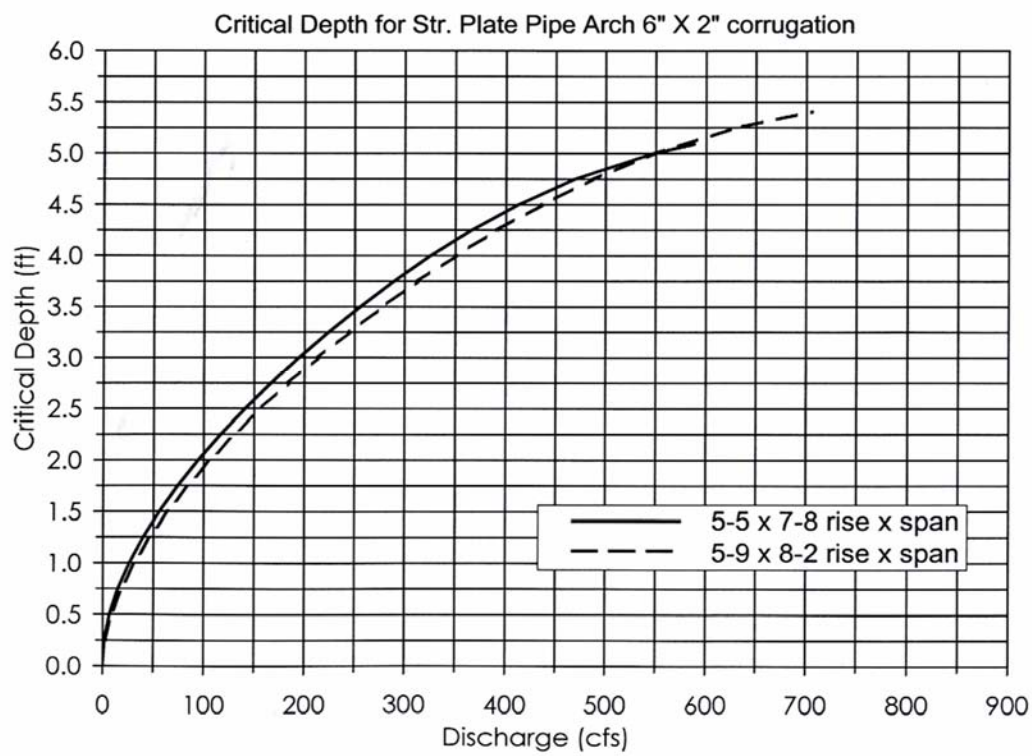


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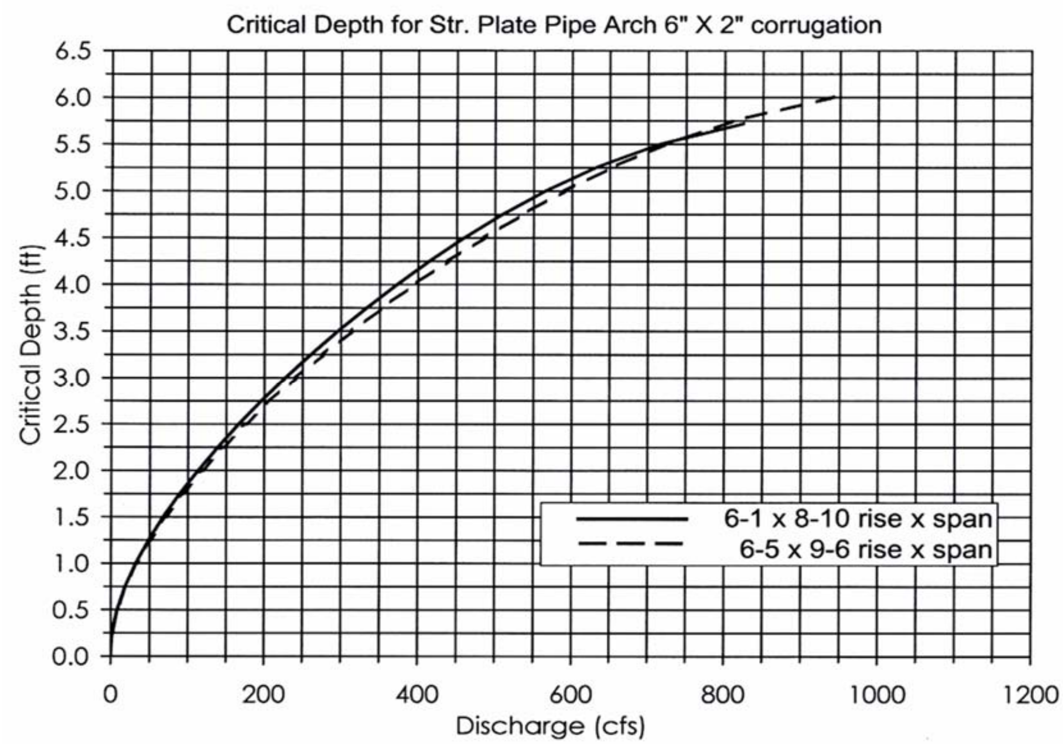
Chart 8-30

Critical Depth Str. Plate Pipe Arch 6 x 2 in. Corrugation



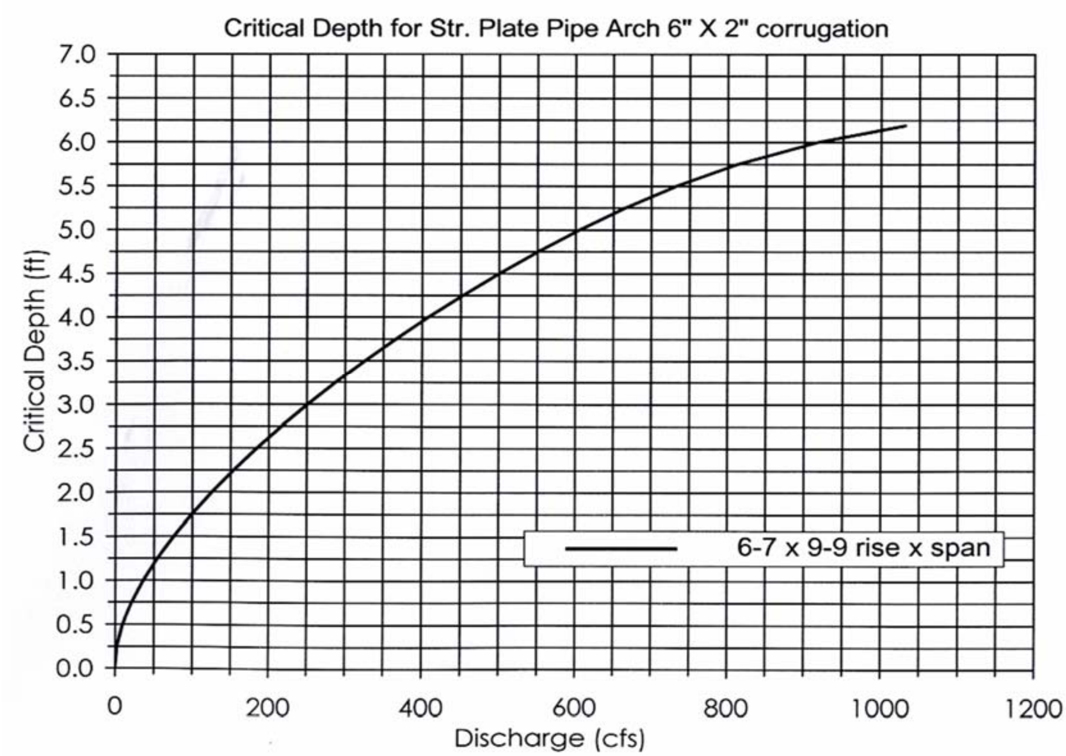


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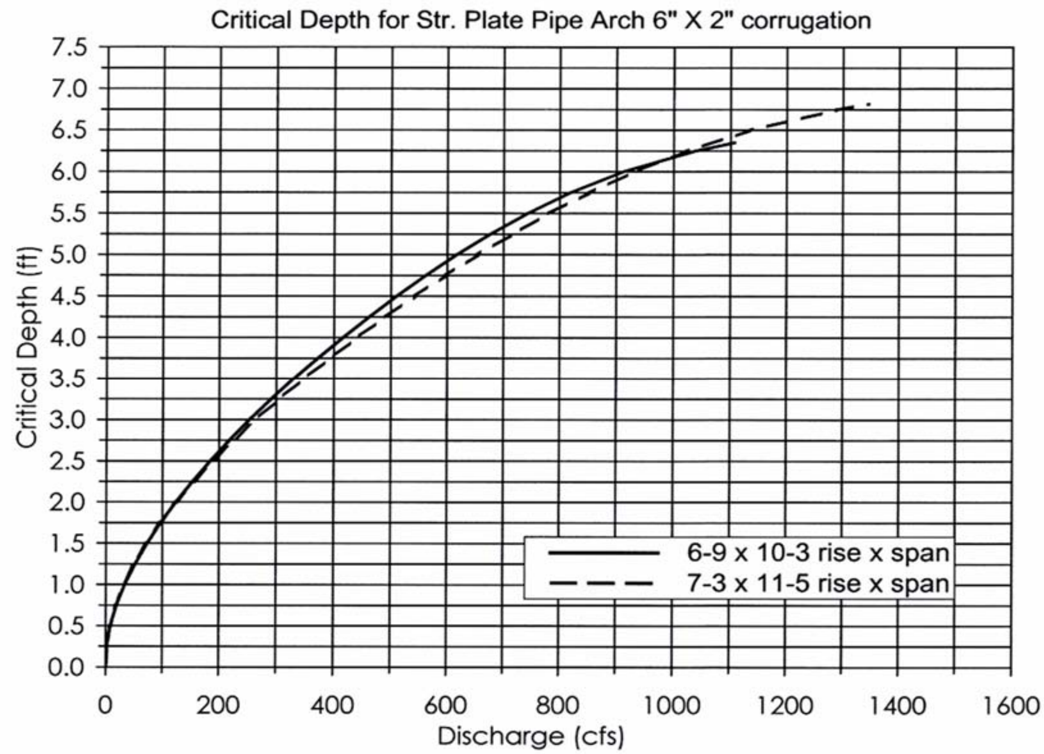
Chart 8-31

Critical Depth Str. Plate Pipe Arch 6 x 2 in. Corrugation



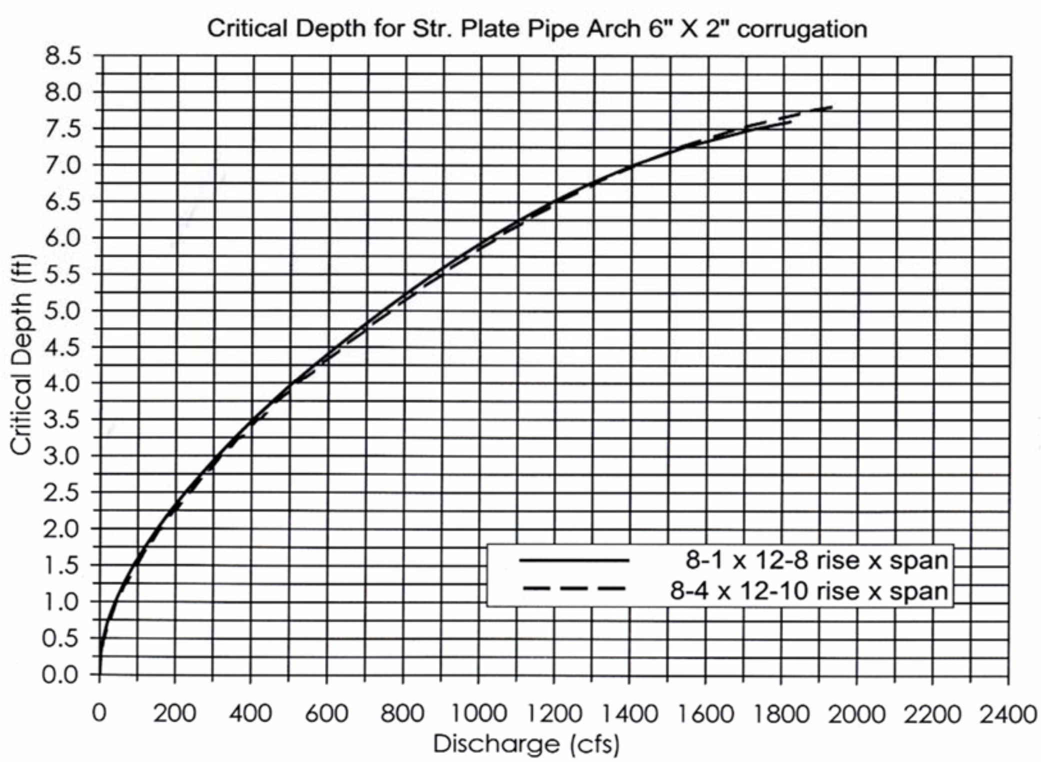


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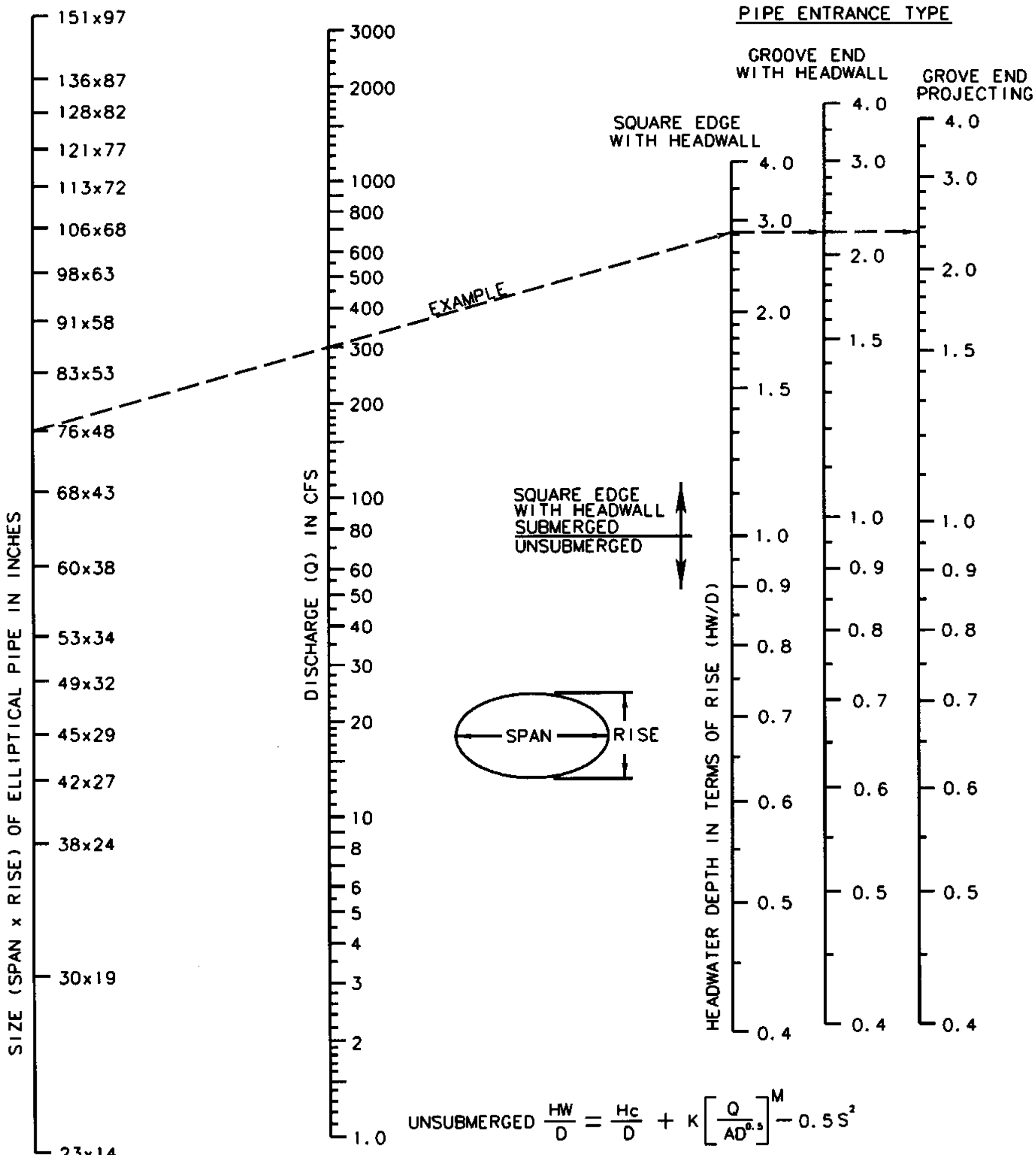
Chart 8-32

Horizontal Elliptical Concrete Pipe with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^M - 0.5S^2$

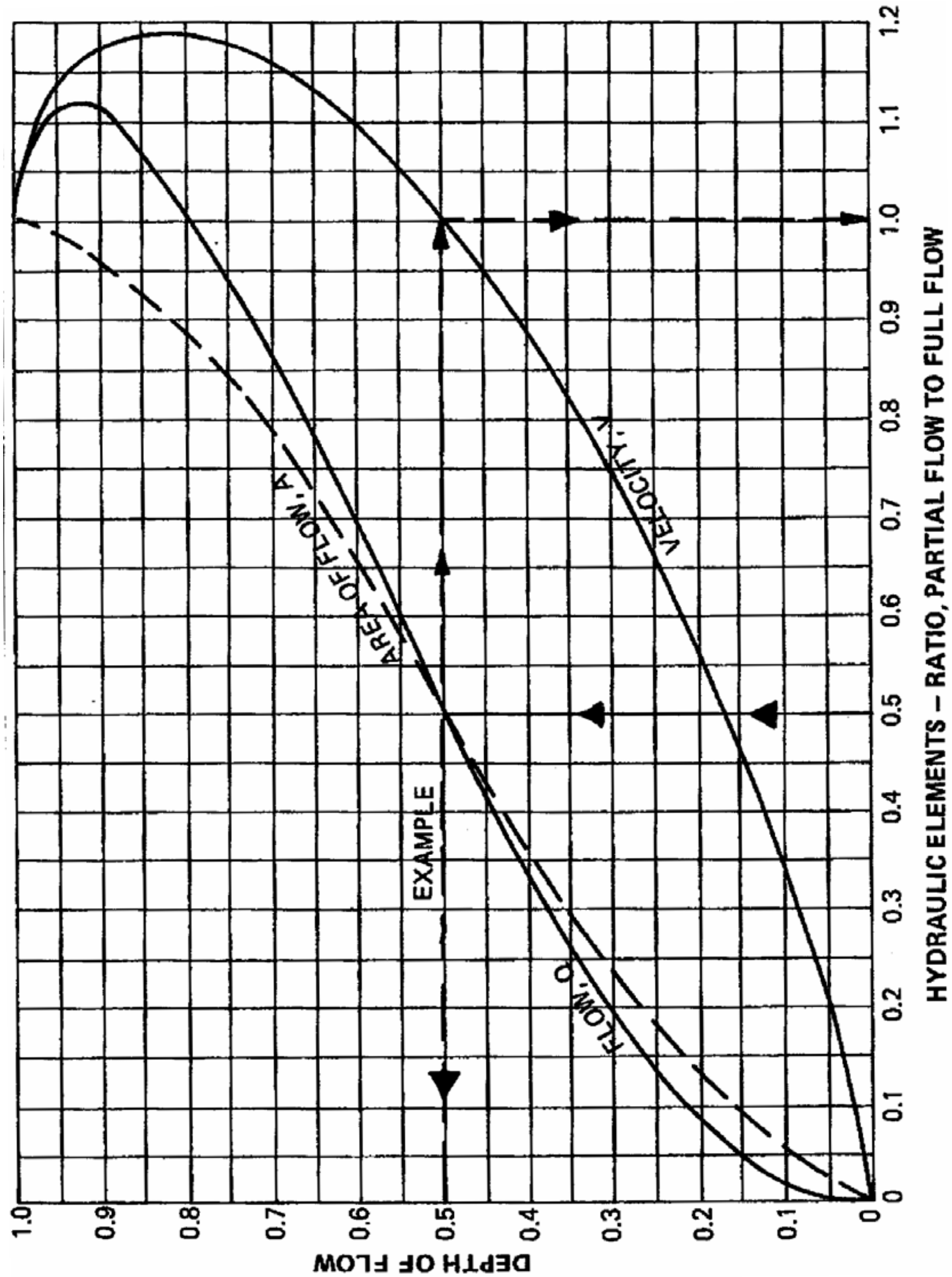
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S^2$

SEE TABLE 8-3 AND TABLE 8-4 FOR K, M, c, Y

REFER TO SECTION 8.4.5 FOR EXPLANATION

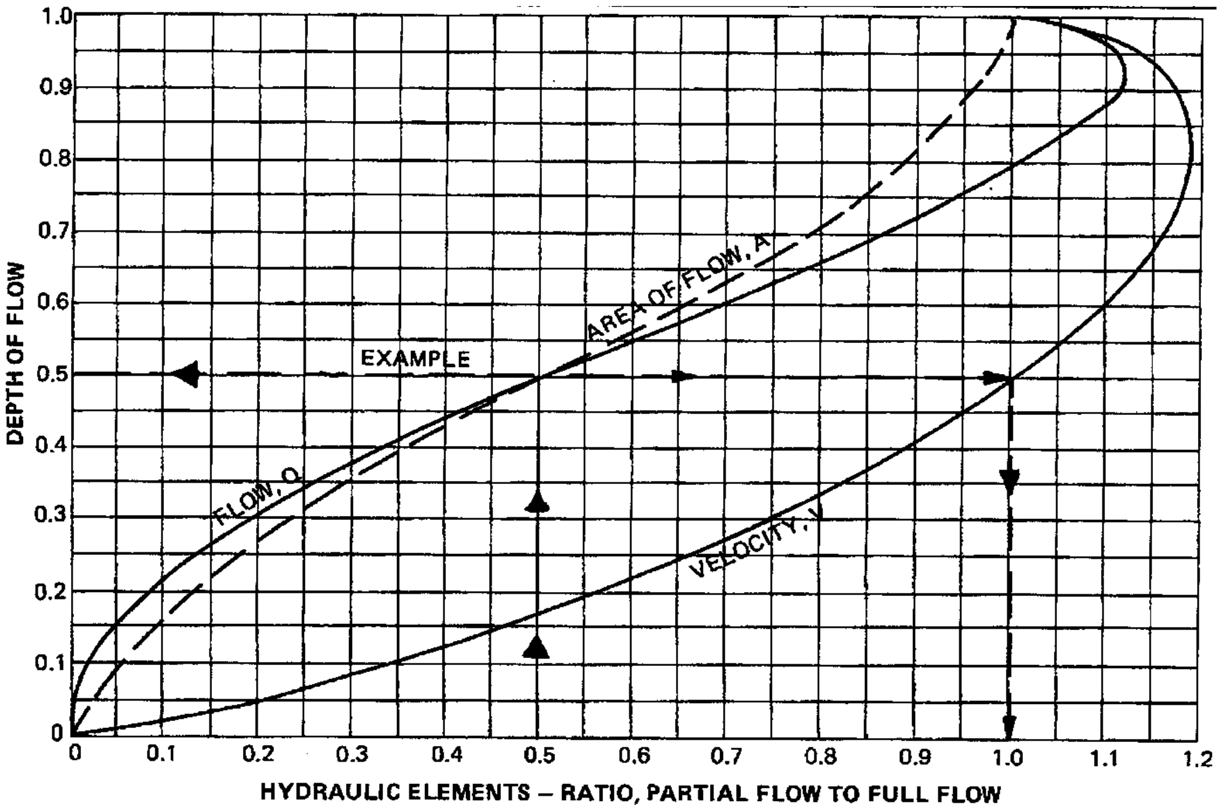
Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-33
Hydraulic Elements for Partially Full Horizontal Elliptical Concrete Pipe



Source: Handbook of Steel Drainage Products, American Iron & Steel Inst., 1994

Chart 8-34
Hydraulic Elements for Partially Full Horizontal Elliptical Concrete Pipe Example



24 X 38 pipe, n = 0.012, Slope = 0.02, Q_{full} / $S^{0.5}$ = 456 (from Table 8-6)

Given: D_{full} = 24 in, Q = 32.2 cfs

$Q_{full} = 456*(0.02)^{0.5} = 64.5$ cfs

Required: depth at Q, velocity at Q

Solution: $Q / Q_{full} = 32.2/64.5 = 0.5$

From figure for hydraulic elements, $d / d_{full} = 0.5$

$d = d_{full} \times 0.5 = 12$ in

Vfull = Q_{full} / A_{full} = 64.5 / 5.1 = 12.6 (A_{full} from Table 8-7)

From figure for hydraulic elements, $V / V_{full} = 1$

$V = V_{full} \times 1 = 12.6$ ft/s

Table 8-6

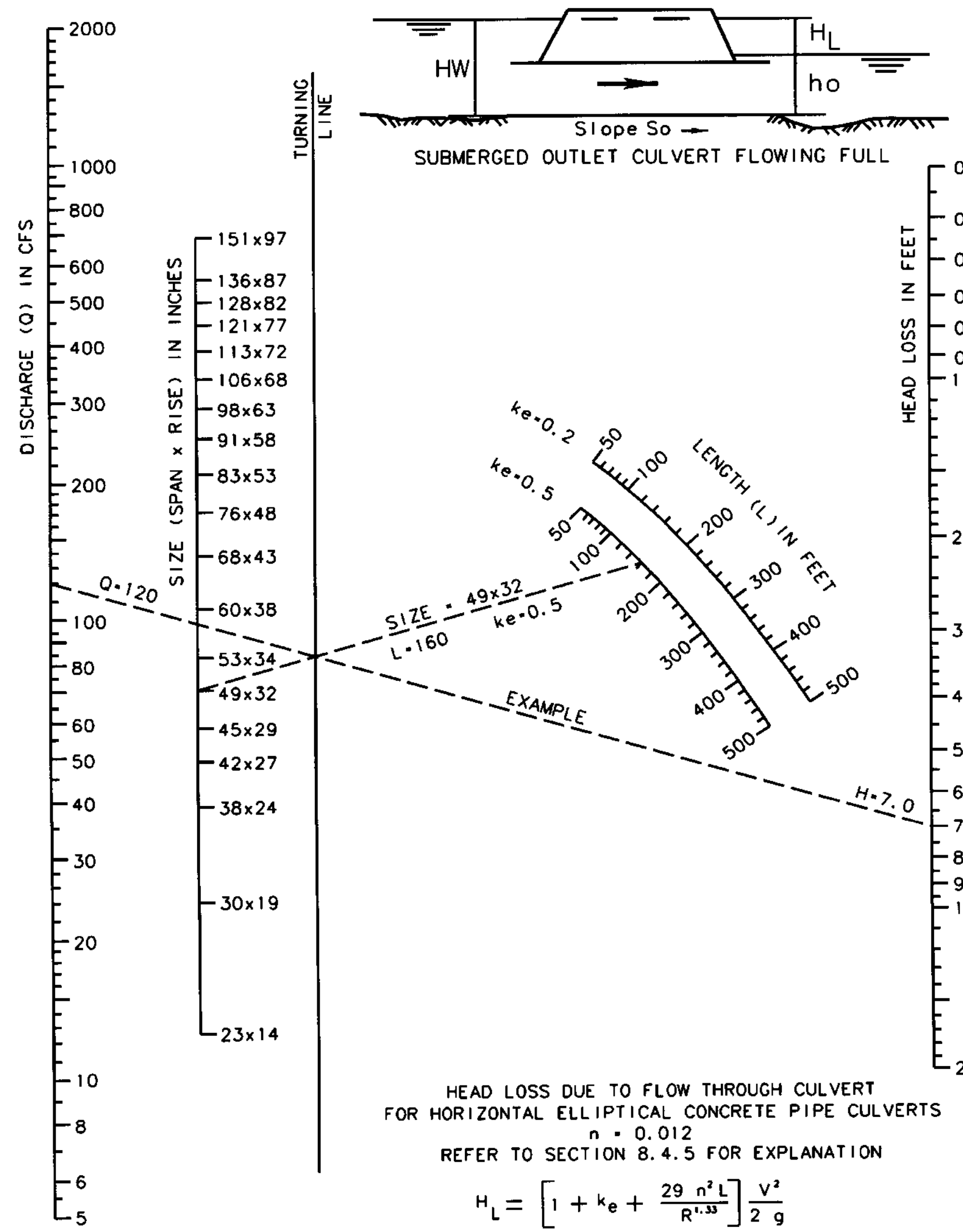
Full Flow Values for Horizontal Elliptical Concrete Pipe

Pipe Size Rise X Span inches	Equivalent Full Flow Area for Circular Diameter inches	A Area ft^2	R Hydraulic Radius ft	Value of Full Flow Conveyance K $\frac{Q}{\sqrt{S}} = \frac{1.486}{n} A R^{\frac{2}{3}}$			
				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
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
97 X 151	120	82.0	2.461	22190	20180	18490	17070

Source: Handbook of Steel Drainage Products, American Iron & Steel Inst., 1994

Chart 8-35

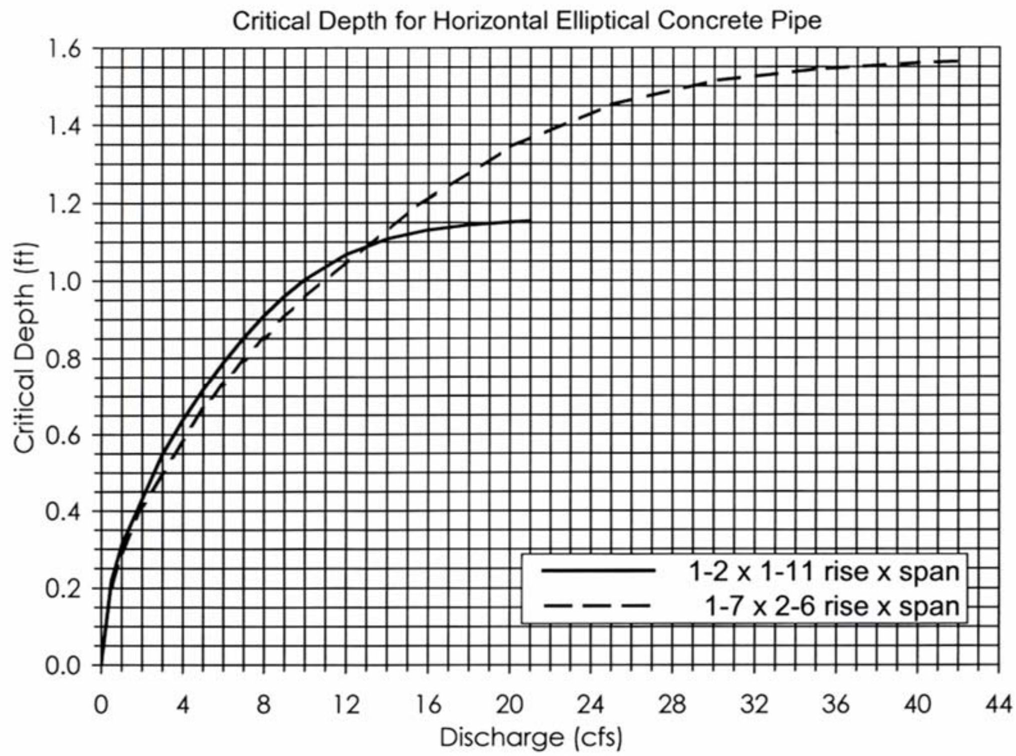
Horizontal Elliptical Concrete Pipe Flowing Full



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-36

Critical Depth for Horiz. Elliptical Concrete Pipe



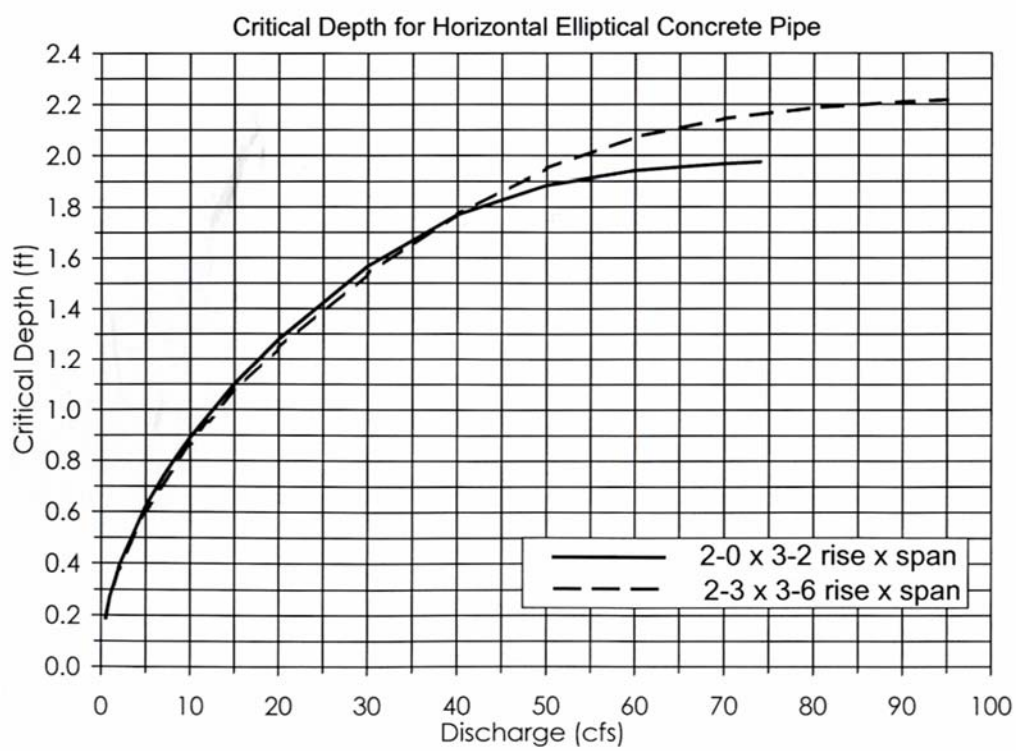


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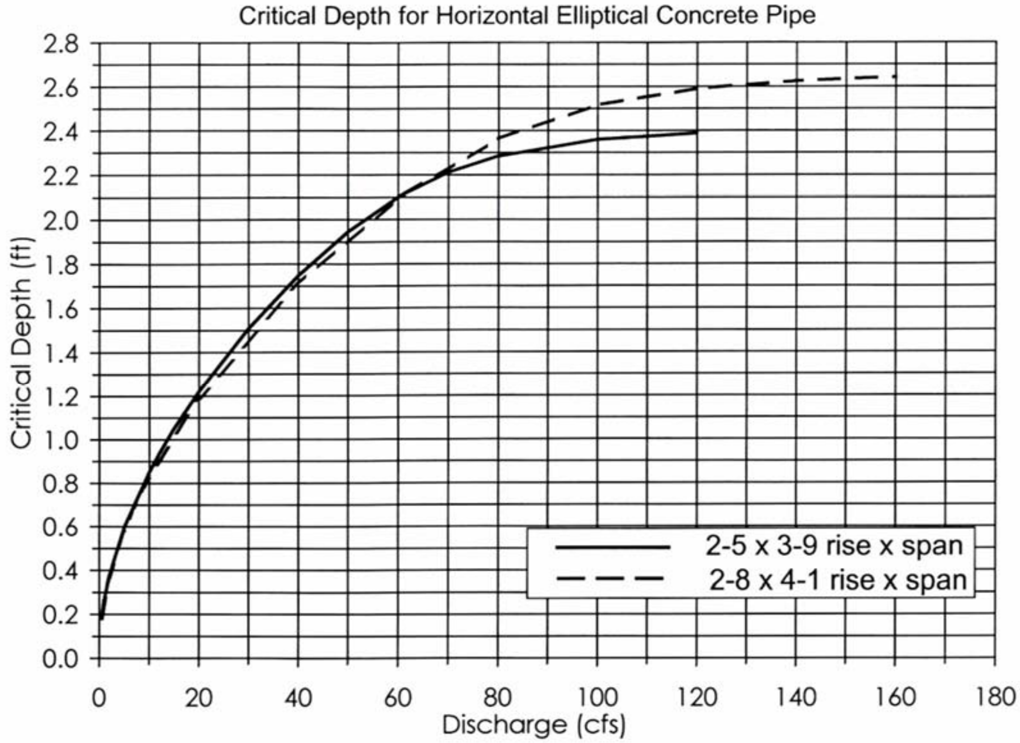
Chart 8-37

Critical Depth for Horiz. Elliptical Concrete Pipe



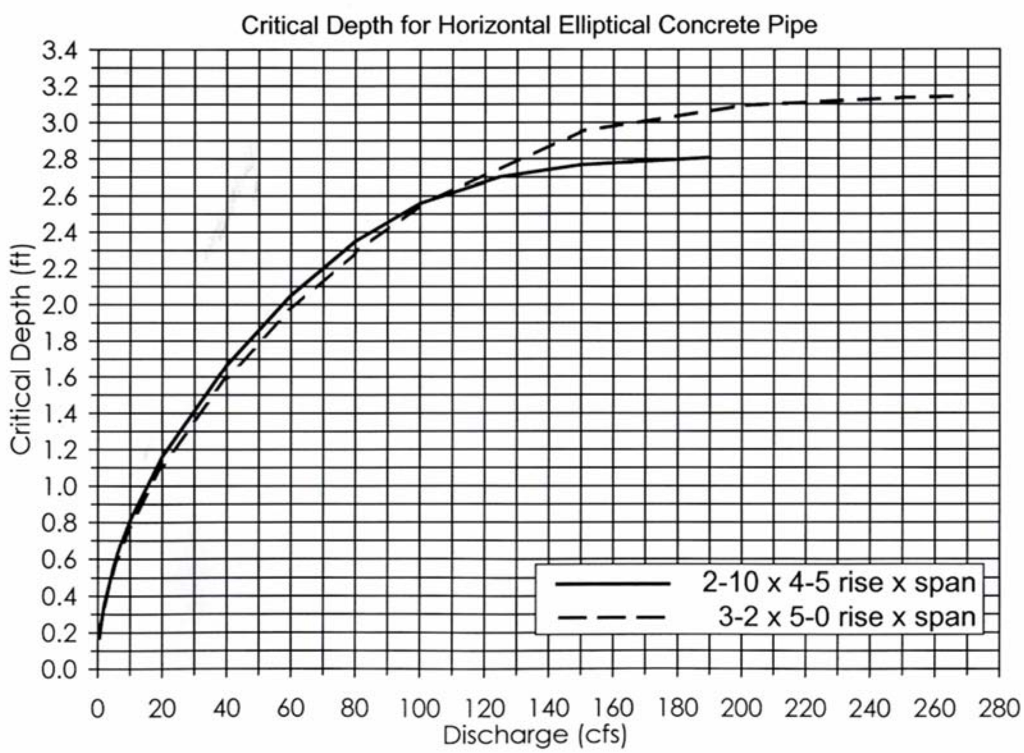


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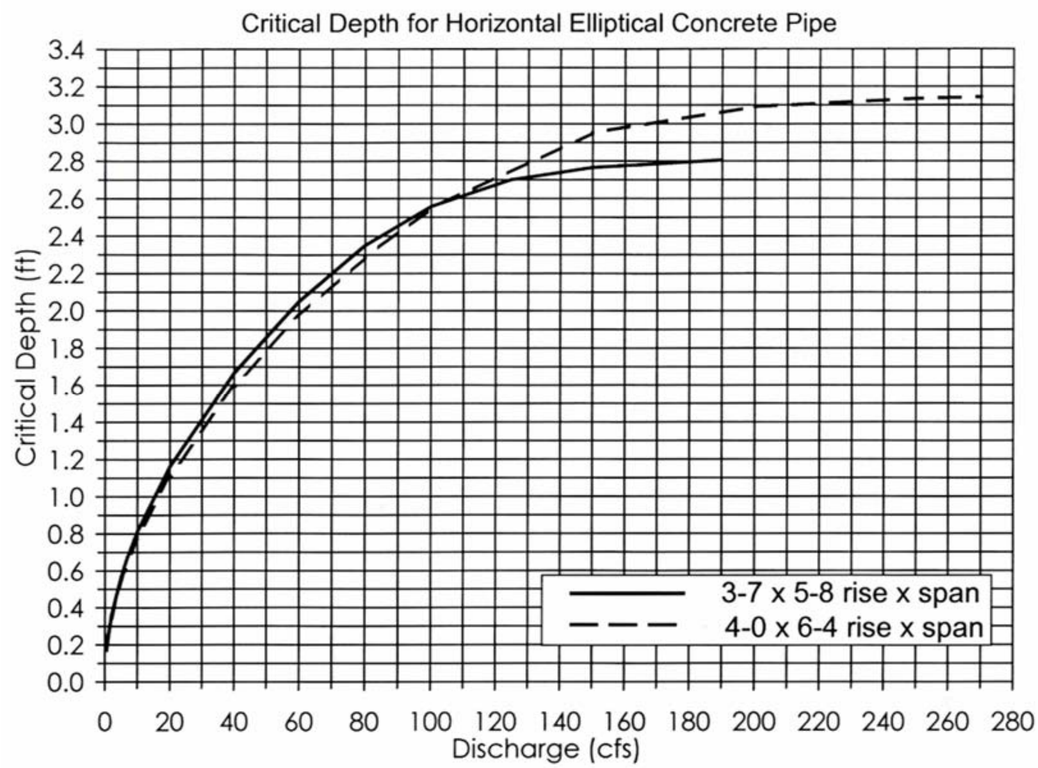
Chart 8-38

Critical Depth for Horiz. Elliptical Concrete Pipe



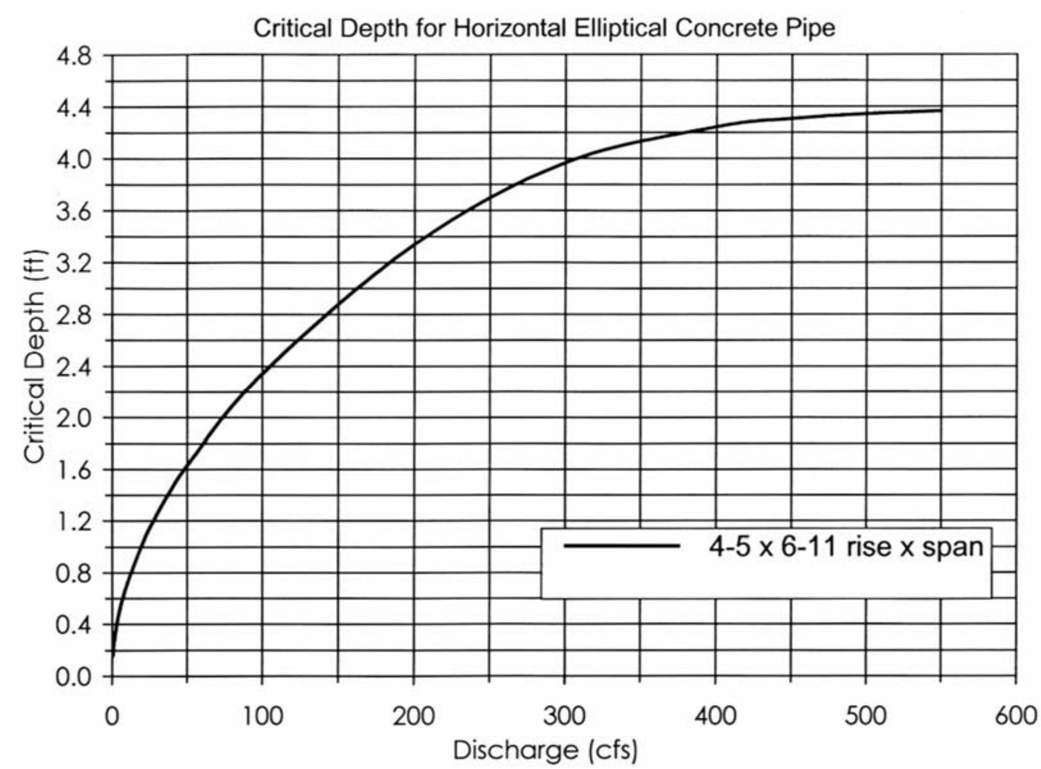


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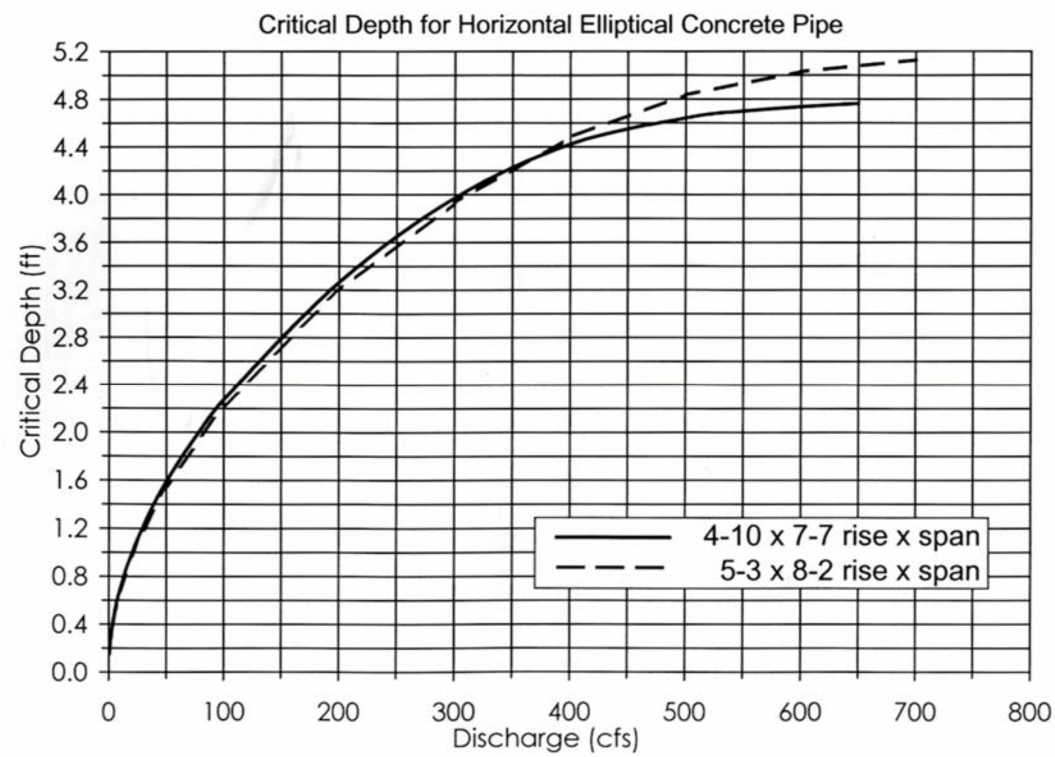
Chart 8-39

Critical Depth for Horiz. Elliptical Concrete Pipe



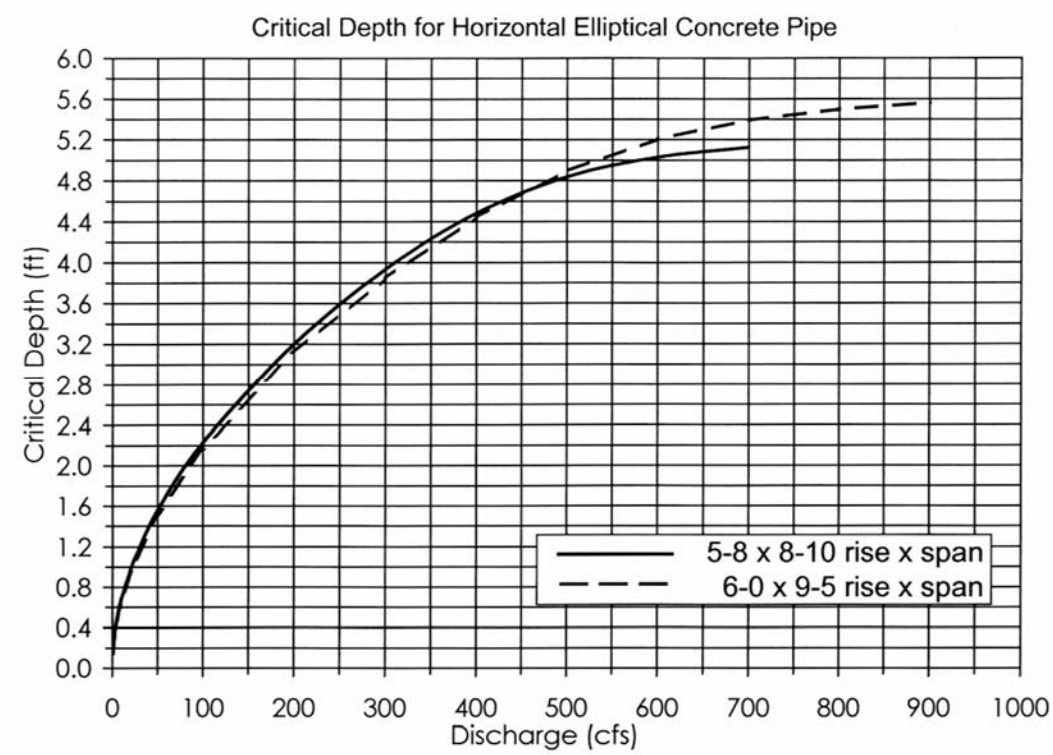


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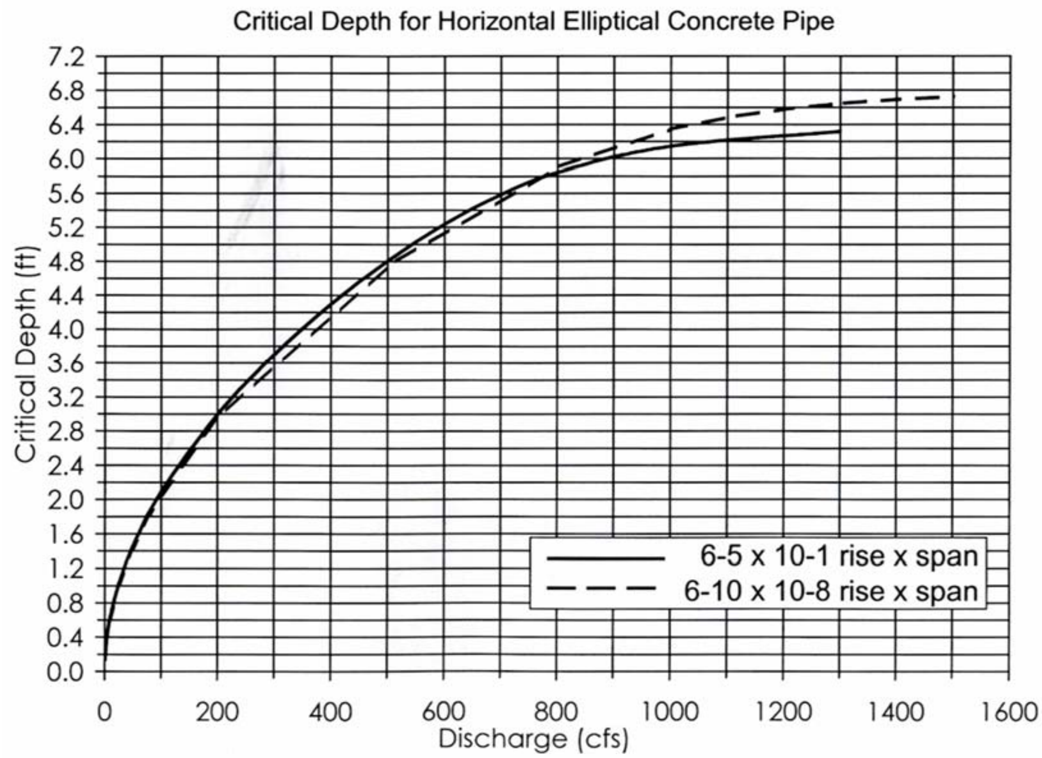
Chart 8-40

Critical Depth for Horiz. Elliptical Concrete Pipe



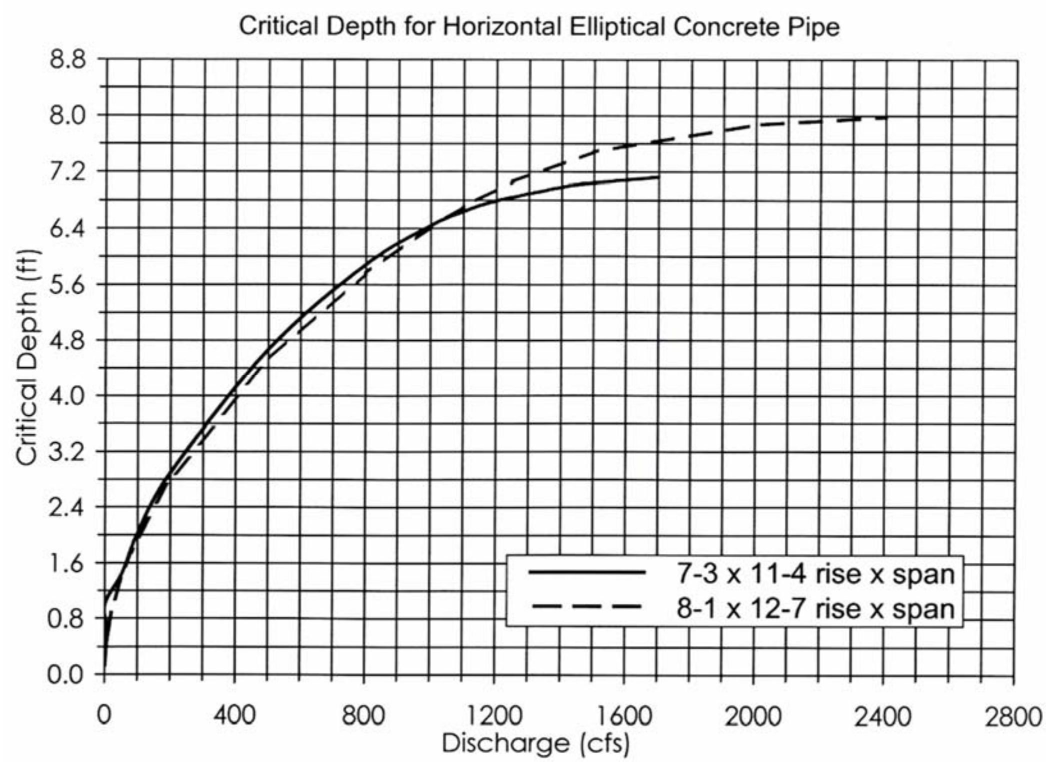


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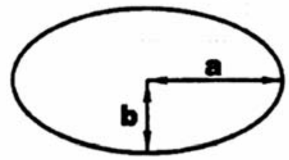
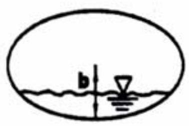
Figure 8–9

Elliptical Pipe Flow Area





$$b - depth > 0 \quad A = a\,b\,\arccos\left(\frac{b - depth}{b}\right) - a\left(b - depth\right)\sqrt{1 - \left(\frac{b - depth}{b}\right)^2}$$

$$b - depth = 0 \quad A = \frac{\pi}{2}\,a\,b$$

$$b - depth < 0 \quad A = \pi\,a\,b - \left[a\,b\,\arccos\left(\frac{depth - b}{b}\right) - a\left(depth - b\right)\sqrt{1 - \left(\frac{depth - b}{b}\right)^2}\right]$$

This chart provides equations for calculating the flow area for any flow depth within an elliptical pipe. It can be useful for calculating the critical depth of flow using the critical depth equation (see Section 8.4.5.1).

8.7 REFERENCES

(1) Design of Roadside Channels with Flexible Linings, Hydraulic Engineering Circular-15 (HEC-15), Federal Highway Administration, April 1988

(2) Design of Riprap Revetment, Hydraulic Engineering Circular-11 (HEC-11), Federal Highway Administration, March 1989

(3) Drainage Manual, West Virginia Department of Highways, Roadway Design Division, 1984

(4) Drainage Manual, West Virginia State Road Commission, Design Division, June 1963

(5) Erosion Control Technology Council, Internet Website: http://www.ectc.org/index.html

(6) Hydraulic Design of Flood Control Channels, EM 1110-2-1601, Engineer Manual, U.S. Army Corps of Engineers, July 1, 1991

(7) Model Drainage Manual, American Association of State Highway Transportation Officials (AASHTO), 2005

(8) Handbook of Steel Drainage & Highway Construction Products, American Iron and Steel Institute, Fifth Edition 1994

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 9:
STORMWATER MANAGEMENT

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 9: STORMWATER

9.1 INTRODUCTION

9.1.1 OVERVIEW

The ever-increasing development of lands has a direct impact on the watersheds that they are contained within. The result of this impact is most often seen as an increase in the peak rate of stormwater runoff. This increase often leads to erosion, siltation, and flooding issues.

This land development impacts the hydrologic cycle in several ways. The natural storage capacity of the land is reduced or even eliminated through the removal of trees and vegetation and through grading of the land. The increase of impervious area due to construction of structures and roads increases runoff volume, decreases natural storage by reducing infiltration and soil storage and also changes both the flood peak time and time of concentration due to surface changes.

The objective of stormwater management is to prevent these problems by mitigating the impacts on the hydrologic cycle to pre-development conditions, or better. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream peak flows. Detention storage facilities can range from oversized ditches, channels or other on-site facilities to large lakes and reservoirs. Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- Prevention or reduction of peak runoff rate increases caused by urban development;
- Mitigation of downstream drainage capacity problems;
- Recharge of groundwater resources;
- Reduction or elimination of the need for downstream outfall improvements; and
- Maintenance of historic low-flow rates by controlled discharge from storage

This chapter provides general information about detention storage basins and procedures for performing preliminary and final sizing and reservoir routing calculations.

9.1.2 LOCATION CONSIDERATIONS

The location of stormwater management facilities has a direct impact on the effectiveness of the facility to control downstream flooding. Small facilities will only have minimal flood control benefits that will quickly diminish as the flood wave travels downstream. Multiple facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could increase or decrease flood peaks in different downstream locations. Therefore, it is important for the engineer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin.

9.1.3 DETENTION VS RETENTION

Stormwater management facilities are often referred to as either detention or retention basins. A common misconception is that these terms are interchangeable, when in fact they have similar but not quite the same functions. Detention facilities are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are depressed areas that store runoff during wet weather and are dry the remainder of the time. They are very popular because of their comparatively low cost, few design limitations, ability to serve large and small watersheds, and their ability to be incorporated into other uses (e.g., recreational areas). Retention facilities are designed to contain a permanent pool of water and do not serve a detention function.

9.1.4 COMPUTER PROGRAMS

The calculations required for routing, although fairly easy, can be very tedious and time consuming. To speed up the process there are several programs commercially available from companies like Haestad Methods and Boss International. These can help expedite the design process with minimal data entry.

9.2 GENERAL CRITERIA

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Storage may also be developed in the following areas:

- under parking lots,
- road embankments,
- freeway interchanges,
- parks and other recreational areas, and

- small lakes, ponds and depressions within urban developments.

The utility of any storage facility depends on the amount of storage, its location within the system and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows that might be expected to pass through the storage facility should be included in the analysis. The design criteria for storage facilities should include:

- release rate
- storage volume;
- grading and depth requirements;
- outlet works;
- location;
- safety; and
- maintenance access (such as berms and access ramps).

9.2.1 STORAGE VOLUME AND DISCHARGE RATES

The storage structure shall be designed to convey significantly large storm events without embankment failure. Storage volume and discharge rates shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the chosen design storms. Routing calculations shall be used to demonstrate that the storage volume and discharge rates are adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project.

9.2.2 GRADING AND DEPTH REQUIREMENTS

Following is a discussion of the general grading and depth criteria for retention (or storage) facilities followed by criteria related to detention facilities.

9.2.2.1 RETENTION

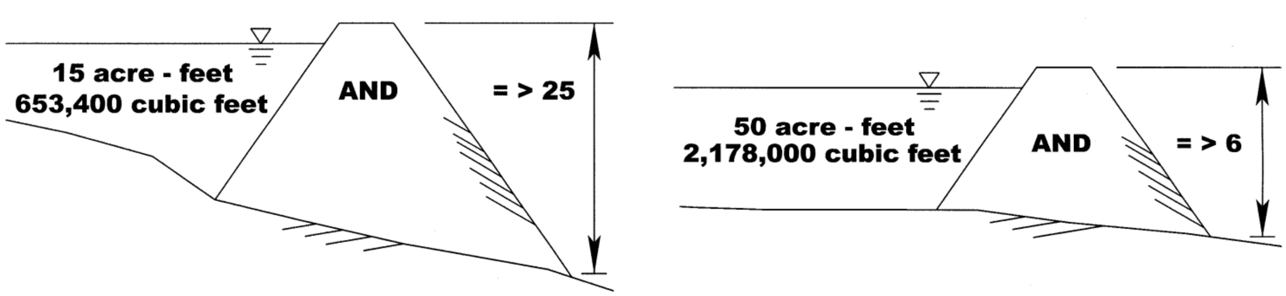
The construction of storage facilities usually requires placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments should be less than 25 feet in height and should have side slopes no steeper than 3H:1V. Side slopes shall be benched at intervals of 5 feet. Riprap-protected embankments shall be no steeper than 2H:1V. Geotechnical slope stability analyses are recommended for embankments greater than 10 feet in height and are mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks. A minimum freeboard of 1 foot above the 100-year design storm high-water elevation should be provided for impoundment depths of less than 20 ft. Other considerations when setting depths include:

- flood elevation requirements;
- public safety;
- land availability;
- land value;
- present and future land use;
- water table fluctuations;
- soil characteristics;
- maintenance requirements;
- required freeboard; and
- aesthetically pleasing features are important in urbanizing areas.

Earthen embankment structures that fall within either of the following classifications shall fall under the requirements of the WVDEP Dam Safety Laws and Rule. An embankment height equal to or greater than 25 feet retaining a volume equal to or greater than 15 acre-feet (653,400 ft^3). An embankment height equal to or greater than 6 feet retaining a volume equal to or greater than 50 acre-feet (2,178,000 ft^3). These earthen embankment heights shall be determined from the downstream toe of the structure. See West Virginia Code Chapter 22 Environmental Resources, Article 14, Dam Control Act, Section 3, Definition of terms used in article.

Figure 9-1

Embankment Rule Classifications



9.2.2.2 DETENTION

Areas above the normal high-water elevations of storage facilities should be graded to drain toward the facilities. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum four percent bottom slope is recommended. A low-flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions.

9.2.2.3 OUTLET WORKS

The sizing of the outlet works is based on the results of hydrologic routing calculations. Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow spillway. Outlet works can take the form of combinations of orifices such as drop inlets connected to pipes, and weirs. Slotted riser pipes are discouraged because of potential clogging problems. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet.

9.2.2.4 EMERGENCY SPILLWAY

The purpose of the emergency spillway is to provide a controlled overflow relief for storm flows in excess of the design discharge for the storage facility. An inlet control structure can be an adequate spillway for highway applications as well as a broad crested overflow weir cut through the original ground next to the embankment. The transverse cross-section of the overflow weir is typically trapezoidal in shape for ease of construction.

9.2.2.5 LOCATION

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin, it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations.

9.3 DESIGN CONCEPTS

9.3.1 DATA NEEDS

The following data will be needed to complete storage design and routing calculations:

- compare pre- and post-peak runoff;
- allowable release rates;

- performance curve for receiving channel;
- inflow hydrograph for selected design storms;
- stage-storage curve for proposed storage facility; and
- stage-discharge curve for all outlet control structures.

9.3.2 PROCEDURE

Step 1: Compare pre and post-construction conditions to determine impact.

Step 2: Compute synthetic inflow hydrograph for runoff from the selected design storms using the procedures outlined in this chapter.

Step 3: Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1.

Step 4: Determine the physical dimensions necessary to hold the estimated volume from Step 3, including freeboard. The maximum storage requirement calculated from Step 3 should be used.

Step 5: Size the principal spillway and emergency outlet. The estimated peak stage will occur for the estimated volume from Step 3. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 6: Perform storage routing calculations using inflow hydrographs from Step 2 to check the preliminary design. If the routed post-development peak discharges from the selected design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 3, then revise the estimated volume (step 4) and return to Step 3.

Step 7: Consider emergency overflow from runoff due to the 100-year or larger design storm and the freeboard requirement.

Step 8: Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity could cause erosion problems downstream.

This procedure can involve a number of iterations of the reservoir routing calculations to obtain the desired result.

9.3.3 STAGE-STORAGE CURVE

A stage-storage curve (Figure **9-2**) defines the relationship between the depth of water and storage volume in a basin. The data for this type of curve are usually developed using a topographic map and the average-end area formula. Storage basins are often irregular in shape in order to blend well with the surrounding terrain. The average-end area formula is expressed as:

$$V_{1,2} = ((A_1 + A_2)/2)D$$

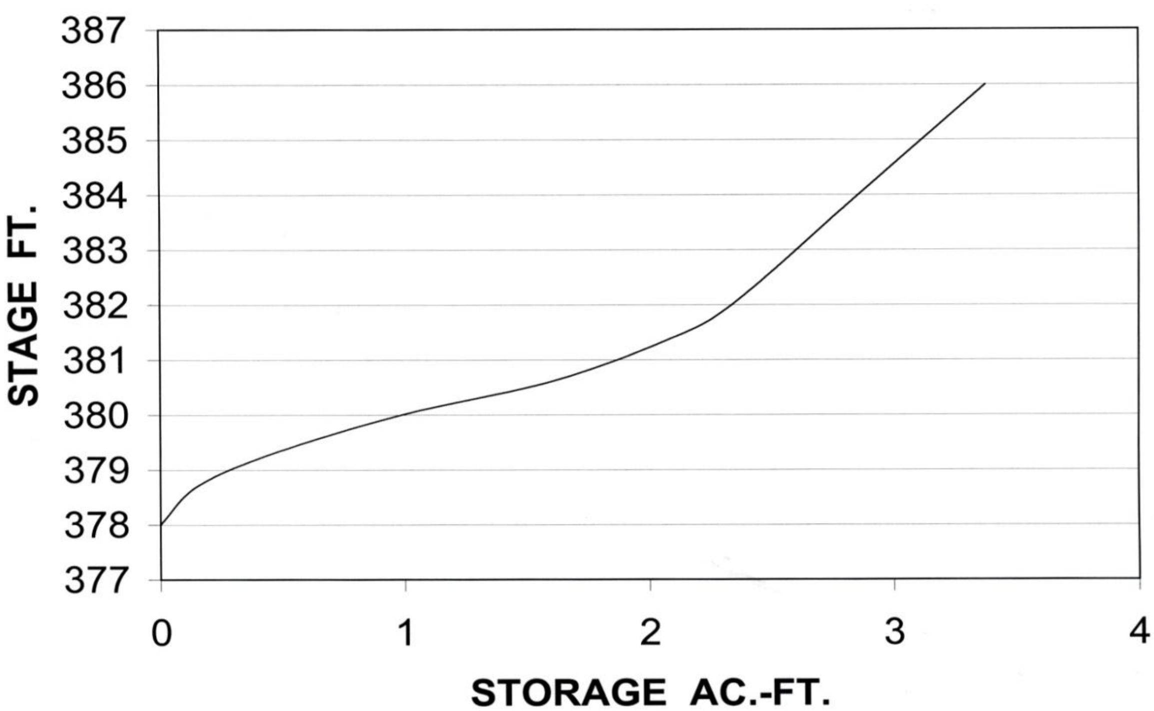
Where: $V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2

$A_{1,2}$ = surface area at elevation 1 and 2 respectively, ft^2

D = change in depth in elevation between points 1 and 2, ft.

Figure 9-2

Example Stage-Storage Curve

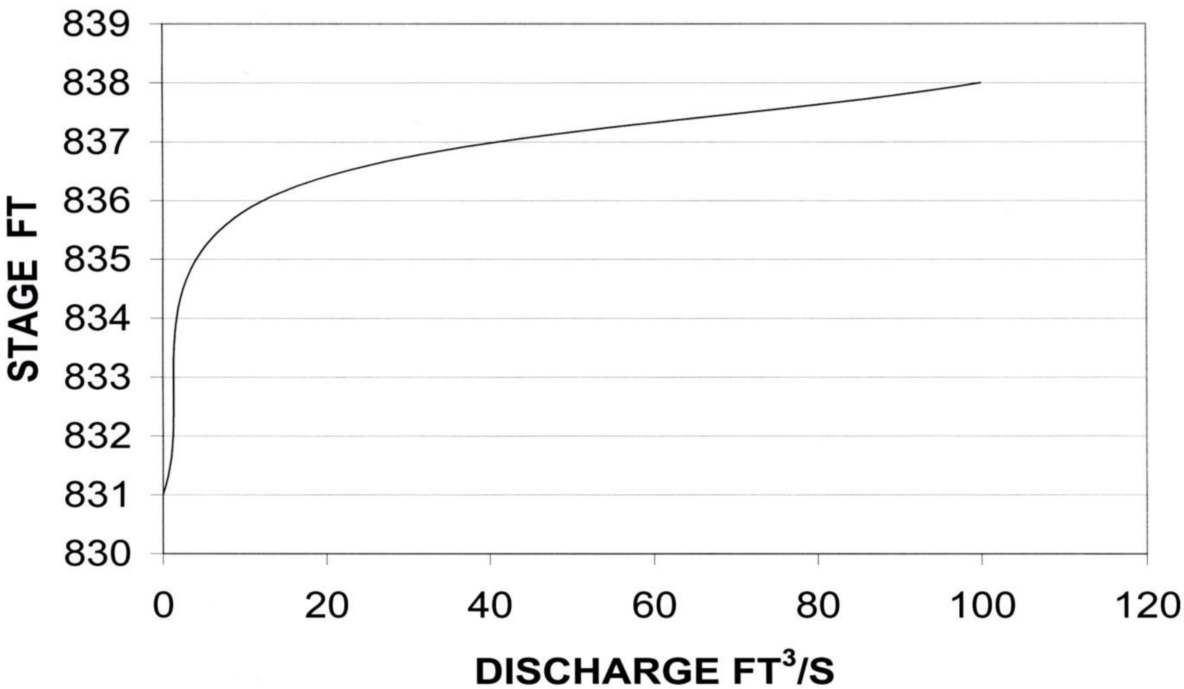


9.3.4 STAGE-DISCHARGE CURVE

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (Figure 9-3). A typical storage facility has two spillways—principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir or other appropriate outlet can be used for the principal spillway or outlet. Tailwater influences and structure losses must be considered when developing discharge curves.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. The stage-discharge curve should reflect the discharge characteristics of both the principal and emergency spillways.

Figure 9-3
Example Stage-Discharge Curve



9.4 OUTLET HYDRAULICS

9.4.1 ORIFICES

An orifice is an opening in a standpipe, riser, weir, or concrete structure. The equation for a single orifice is:

$$Q = C_o \, A_o \, \sqrt{2 \, g \, H_o}$$

Where:

Q = discharge, ft^3/s

C_o = discharge coefficient (0.2 – 0.6)

A_o = cross-sectional area of orifice, ft^2

g = acceleration due to gravity, 32.2 ft/s^2

H_o = head on orifice, ft

If the orifice discharges as a free outfall, then the effective head is measured from the centerline of the orifice to the upstream water surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces. This latter condition of a submerged discharge is shown in Figure 9-5.

For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used. For ragged edged orifices, such as those resulting from the use of a torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used.

For circular orifices with C_o set equal to 0.6, the following equation results:

$$Q = K_{or}\ D^2 \sqrt{H_o}$$

Where:

K_{or} = 3.78

D = orifice diameter, ft

Pipes smaller than 1 foot in diameter may be analyzed as a submerged orifice as long as H_o/D is greater than 1.5. Pipes greater than 1 foot in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account, not just as an orifice.

9.4.2 WEIRS

9.4.2.1 BROAD-CRESTED WEIRS

The most common type of weir associated with stormwater management is the broad-crested weir as is defined by the equation:

$$Q = C_{BCW}\ L\ H^{1.5}$$

Where:

Q = discharge, ft^3/s

C_{BCW} = broad-crested weir coefficient from Table 9-1 (Range from 2.34 to 3.32 and is generally assumed to be 3.0)

L = broad-crested weir length, ft.

H = head above weir crest, ft.

If the upstream edge of a broad-crested weir is rounded so as to prevent contraction and if the slope of the crest is as great as the head loss due to friction, flow will pass through critical depth at the weir crest. This gives the maximum entrance coefficient (C) of 3.00. For sharp corners on the broad-crested weir;

however, a minimum (C) of 2.67 should be used. The designer should also check to make certain the weir or orifice is not submerged by downstream tailwater.

Table 9-1

Broad-Crested Weir Coefficient

Measured Head, H[1] (ft)	***Breadth of the Crest of Weir (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*	*2.27*	*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*

9.4.2.2 SHARP-CRESTED WEIRS

Typical sharp crested weirs are illustrated in Figure 9-4. The following equation provides the discharge relationship for **sharp crested weirs with no end contractions** (Figure 9-4a).

$$Q = C_{SCW}\ L\ H^{1.5}$$

Where:

Q = discharge (ft3/s)

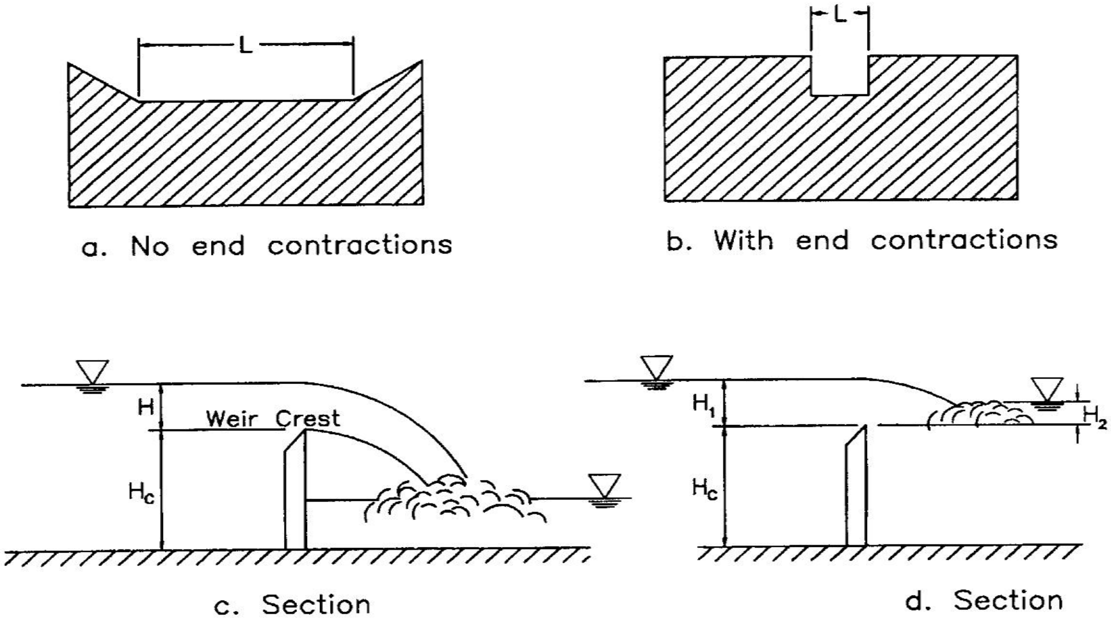
L = horizontal weir length (ft)

H = head above weir crest excluding velocity head (ft)

C_{SCW} = 3.27 + 0.4 (H/H_c)

Figure 9-4

Sharp Crested Weirs



As indicated above, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c (see Figure 9-4c for definition of terms). For values of the ratio, H/H_c less than 0.3, a constant C_{SCW} of 3.33 is often used.

The following equation provides the discharge equation for **sharp-crested weirs with end contractions** (Figure 9-4b). As indicated above, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c (see Figure 9-4c for definition of terms). For values of the ratio H/H_c less than 0.3 a constant C_{SCW} of 3.33 is often used, but not mandated.

$$Q = C_{SCW}\left(L - 0.2H\right)H^{1.5}$$

Sharp crested weirs will be affected by submergence when the tailwater rises above the weir crest elevation, as shown in Figure 9-4d. The result will be that the discharge over the weir will be reduced. The discharge equation for a **submerged sharp-crested weir** is:

$$Q_S = Q_r\left(1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right)^{0.385}$$

Where:

Q_s = submerged flow (ft^3/s)

Q_r= unsubmerged weir flow (from either of the 2 previous unsubmerged equations) (ft^3/s)

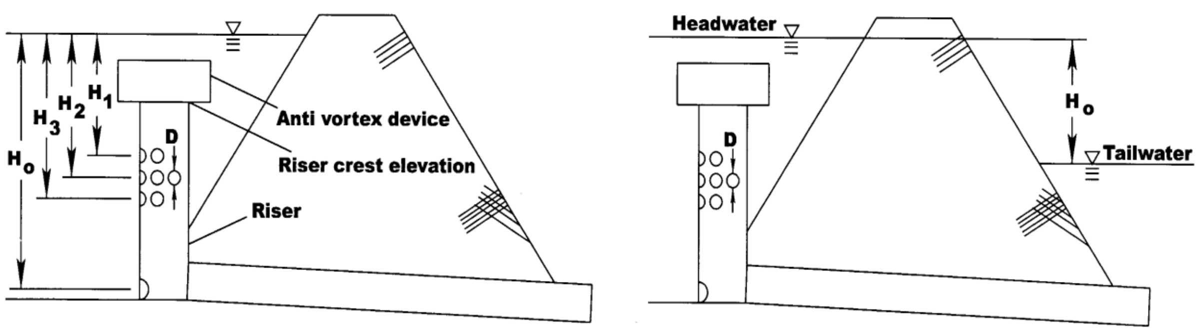
H_1 = upstream head above crest (ft)

H_2 = downstream head above crest (ft)

Flow over the top edge of a riser pipe is typically treated as flow over a sharp crested weir with no end contractions; therefore, the equation for no end contractions should be used.

Figure 9-5

Riser Pipe



9.4.3 PRELIMINARY DETENTION STORAGE VOLUME CALCULATIONS

9.4.3.1 PRELIMINARY DESIGN COMPUTATIONS

The final design of a detention facility requires three items. They are an inflow hydrograph, a stage vs. storage curve, and a stage vs. discharge curve. However, before a stage vs. storage and a stage vs. discharge curve can be developed, a preliminary estimate of the needed storage capacity and the shape of the storage facility are required. Trial computations will be made to determine if the estimated storage volume will provide the desired outflow hydrograph.

9.4.3.2 ESTIMATING REQUIRED STORAGE

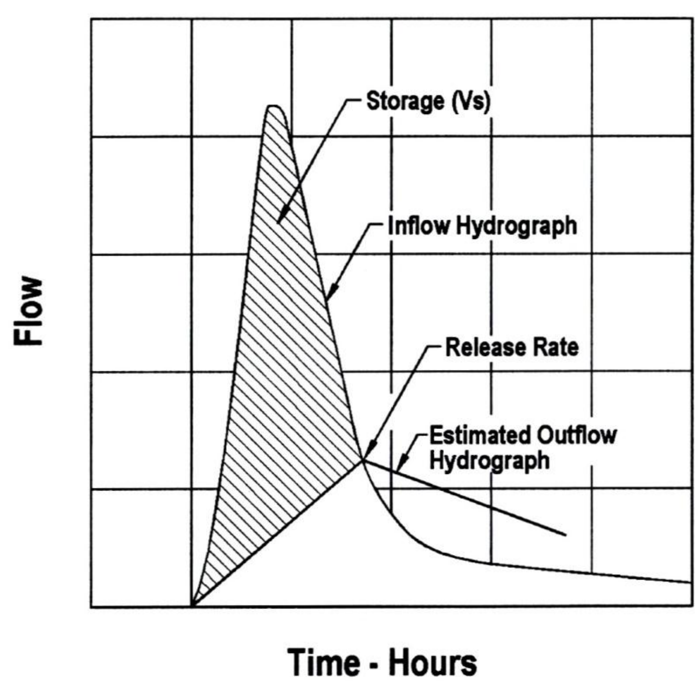
Estimating the required volume of storage to accomplish the necessary peak reduction is an important task since an accurate first estimate will reduce the number of trials involved in the routing procedure. The following sections present three methods for determining an initial estimate of the storage required to provide a specific reduction in peak discharge. All of the methods presented provide preliminary estimates only. It is recommended that the designer apply several of the methods and a degree of judgment to determine the initial storage estimate.

9.4.3.3 HYDROGRAPH METHOD

The hydrograph method of estimating the required volume of storage requires an inflow hydrograph and an outflow hydrograph. The storage required for the basin will be the volume difference between the two hydrographs. The inflow hydrograph will be the one established as the final runoff from the watershed flowing into the detention basin. The outflow hydrograph is unknown at the beginning of the process and is what the routing process will eventually establish. However, for the initial estimation of the needed storage, the outflow hydrograph must be estimated. It may be approximated by straight lines or by sketching an assumed outflow curve as shown on Figure 9-6. The peak of this estimated outflow hydrograph must not exceed the desired peak outflow from the detention basin. After this curve is established, the shaded area between the curves represents the estimated storage that must be provided. To determine the necessary storage, the shaded area can be measured by planimeter or computed mathematically by using a reasonable time period and appropriate hydrograph ordinances.

Figure 9-6

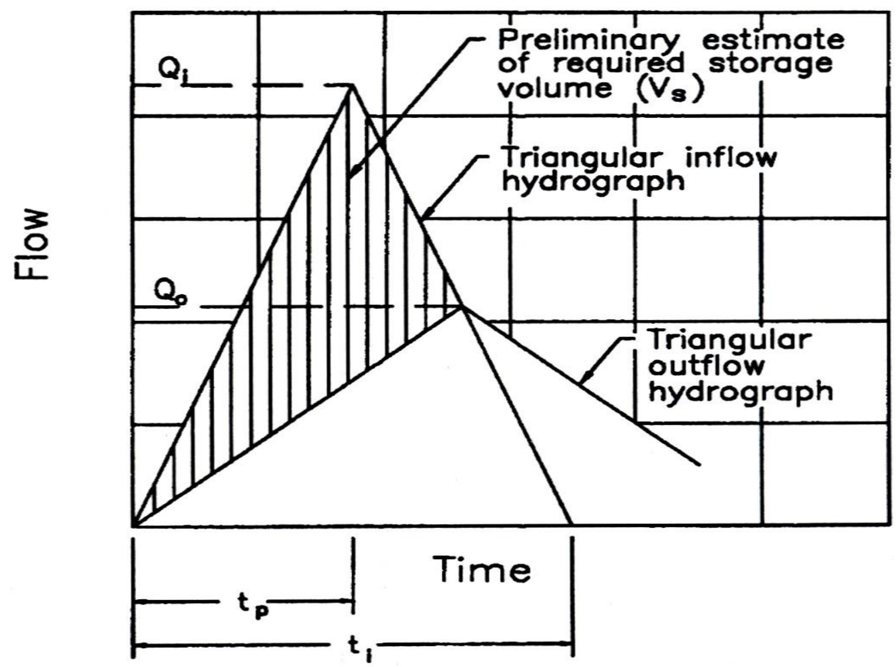
Hydrograph Method



9.4.3.4 TRIANGULAR HYDROGRAPH PROCEDURE

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the inflow and outflow hydrographs with standard triangular shapes. This method should not be applied if the hydrographs cannot be approximated by a triangular shape. This would introduce additional errors in the preliminary estimate of the required storage.

Figure 9-7
Detention Storage with Triangular Hydrographs



The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = 0.5T_i\left(Q_i - Q_o\right)$$

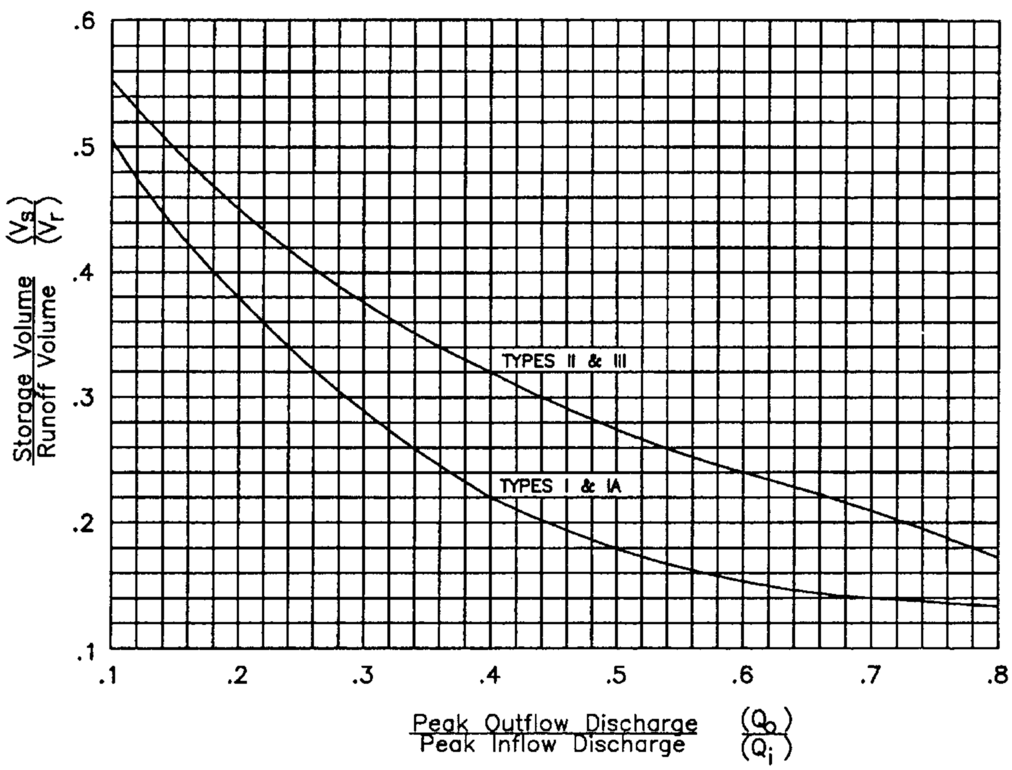
Where:	V_s	=	Storage volume estimate, ft^3/s
	Q_i	=	Peak inflow rate, ft^3/s
	Q_o	=	Peak outflow rate, ft^3/s
	T_i	=	Duration of basin inflow, s

9.4.3.5 SCS PROCEDURE

The Natural Resources Conservation Service (NRCS, formerly known as Soil Conservation Service or SCS), in its TR-55 Manual (Second Edition), describes a manual method for estimating required storage volumes based on peak inflow and outflow rates. This method is based on average storage and routing effects observed for a large number of structures. A dimensionless figure relating the ratio of basin storage volume (V_s) to the inflow runoff volume (V_r) with the ratio of peak outflow (Q_o) to peak inflow (Q_i) was developed as illustrated in Figure 9-8. This procedure for estimating storage volume may have errors up to 25% and, therefore, should only be used for preliminary estimates.

Figure 9-8

SCS Detention Basin Routing Curves



Source: Urban Hydrology for Small Watersheds, TR-55, June 1986

The procedure for using Figure 9-8 in estimating the detention storage required is described as follows:

- Determine the inflow and outflow discharges Q_i and Q_o.
- Compute the ratio Q_o/ Q_i

- Compute the inflow runoff volume, V_r, for the design storm.

$$V_r = K_r Q_D A_m$$

Where: V_r = inflow volume of runoff, (ac-ft)

K_r = 53.33

Q_D = depth of direct runoff (in)

A_m = area of watershed (mi^2)

- Using Figure 9-8, determine the ratio V_s/V_r.
- Determine the storage volume V_s, as

$$V_s = V_r\left(\frac{V_s}{V_r}\right)$$

9.4.4 ROUTING CALCULATIONS

The most commonly used method for routing inflow hydrograph through a detention pond is the Storage Indication or Modified Pulse Method. This method is based on the principle of conservation of mass which states that the inflow minus the out flow equals the change in storage (I – O = ΔΣ). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by the following equation:

$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2}$$

Where: ΔS = change in storage, (ft^3)

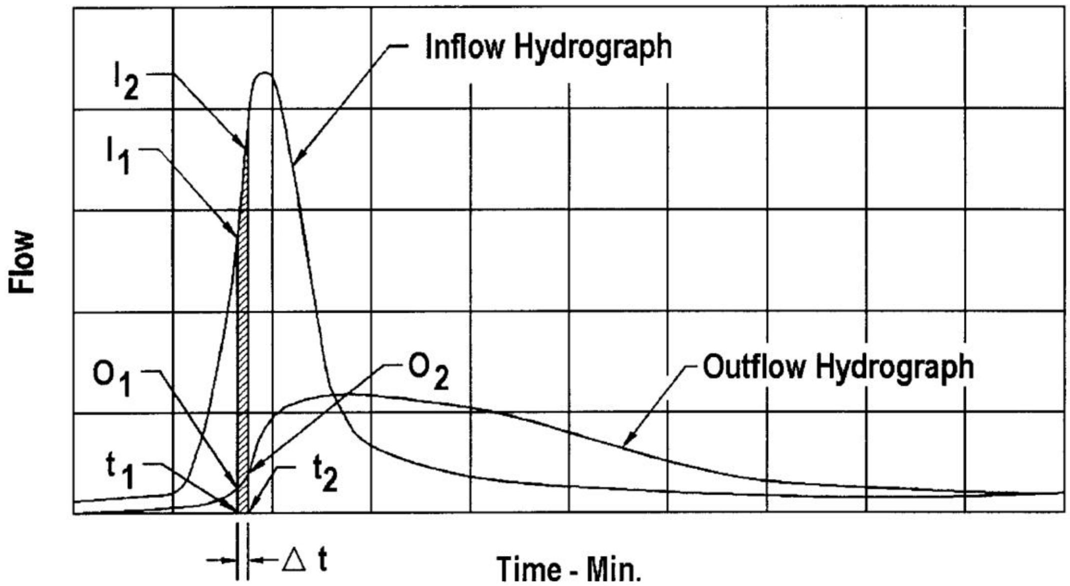
Δt = time interval, min

I = inflow, (ft^3)

O = outflow, (ft^3)

Figure 9-9

Hydrograph Routing Schematic



The equation can then be rearranged so that all the known values are on the left side of the equation and all the unknown values are located on the right side of the equation. Now the equation with two unknowns, S_2 and O_2, can be solved with one equation.

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right)$$

The following procedure can be used to perform routing through a reservoir or storage facility using this equation.

Step 1: Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility.

Step 2: Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph.

Step 3: Use the stage-storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S/(\Delta t) + O/2$ versus stage. A typical storage indicator table contains the following column headings:

Table 9-2

Stage Storage Indicator Table

(1) Stage (ft)	(2) Discharge (O) (ft^3/s)	(3) Storage (S) (ft^3)	(4) $O_2/2$ (ft^3/s)	(5) $S_2/\Delta t$ (ft^3/s)	(6) $S_2/\Delta t + O_2/2$

a. The discharge (O) and storage (S) are obtained from the stage-discharge and stage-storage curves, respectively.

b. The subscript 2 is arbitrarily assigned at this time.

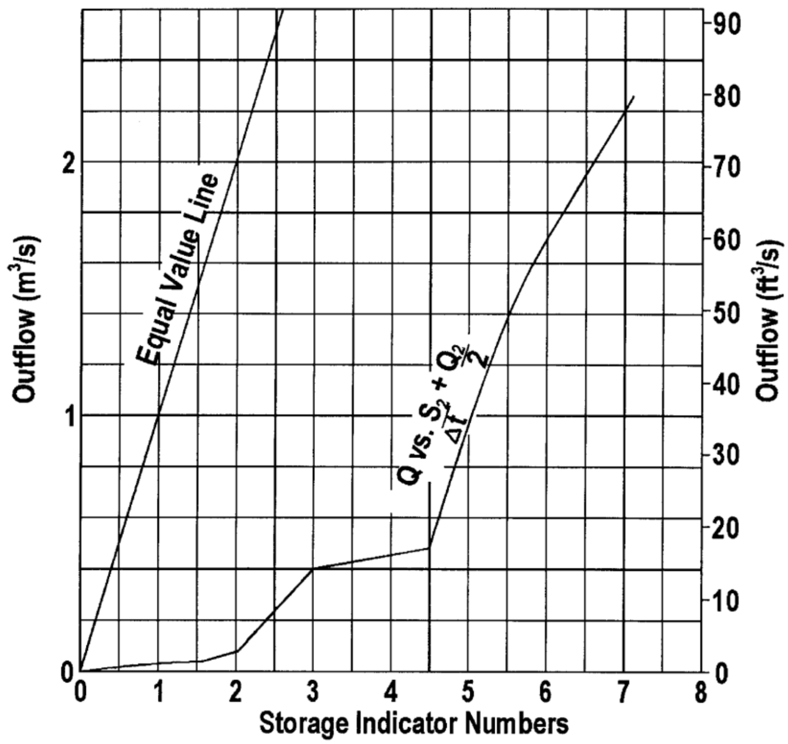
c. The time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.

Step 4: Develop a storage indicator curve by plotting the outflow (O) vertically against the storage indicator values ($S_2/\Delta t + O_2/2$). An equal value line plotted as $O_2 = S_2/\Delta t + O_2/2$ should also be plotted. If the storage indicator curve crosses the equal value line, a smaller time increment (Δt) is needed.

- The discharge (O) and storage (S) are obtained from the stage-discharge and stage-storage curves.
- The subscript 2 is arbitrarily assigned at this time.
- The time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.

Figure 9-10

Storage Indicator Curve



Step 5: A supplementary curve of storage (S) vs. $S_2/\Delta t + O_2/2$ can also be constructed. This curve is not used in the routing calculations; however, it is useful for identifying storage for any given value of $S_2/\Delta t + O_2/2$. A plot of Storage vs. Time can be developed from this curve.

Step 6: The routing can now be performed by developing a routing table for the solution of the routing equation (Table 9-3):

- Columns (1) and (2) are obtained from the inflow hydrograph.
- Column (3) is the average inflow over the time interval.
- The initial values for columns (4) and (5) are generally assumed to be zero since there is no storage or discharge at the beginning of the hydrograph when there is no inflow into the basin.
- The left side of the rearranged equation is determined algebraically as columns (3) + (4) – (5). This value equals the right side of the equation or $S_2/\Delta t + O_2/2$ and is placed in column (6).
- Enter the storage indicator curve with $S_2/\Delta t + O_2/2$ (column 6) to obtain O_2 (Column 7).

- Column (6) $S_2/\Delta t + O_2/2$ and column (7) O_2 are transported to the next line and become $S_1/\Delta t + O_1/2$ and O_1 in columns (4) and (5), respectively. Because $S_2/\Delta t + O_2/2$ and O_2 are the ending values for the first time step, they can also be said to be the beginning values for the second time step.
- Columns (3), (4), and (5) are again combined and the process is continued until the storm is routed.
- Peak storage depth and discharge (O_2 in column (7)) will occur when column (6) reaches a maximum. The storage indicator numbers table developed in step 3 is entered with the maximum value of $S_2/\Delta t + O_2/2$ to obtain the maximum amount of storage required. This table can also be used to determine the corresponding elevation of the depth of stored water.
- The designer needs to make sure that the peak value in column (7) does not exceed the allowable discharge as prescribed by the stormwater management criteria.

Step 7: Plot O_2 (column 7) vs. time (column 1) to obtain the outflow hydrograph.

Table 9-3

Example Final Routing Table

(1) Time (hr)	(2) Inflow (ft^3/s)	(3) $(I_1+I_2)/2$ (ft^3/s)	(4) $(S_1/\Delta t+O_1/2)$ (ft^3/s)	(5) O_1 (ft^3/s)	(6) $(S_2/\Delta t+O_2/2)$ (ft^3/s)	(7) O_2 (ft^3/s)

Table 9-4 shows an example of the stage-discharge tabulation for a typical detention basin routing.

Table 9-4

Example Stage-Discharge Tabulation

Stage (ft)	Low Flow Orifice (ft3/s)	Riser Orifice Flow (ft3/s)	Emergency Spillway (ft3/s)	Total Discharge (ft3/s)
32.8	0.00	0.00	0.00	0.00
33.5	0.39	0.00	0.00	0.39
34.1	0.85	0.00	0.00	0.85
34.8	1.13	0.00	0.00	1.13
35.4	1.34	0.00	0.00	1.34
36.1	1.52	9.18	0.00	10.70
36.7	1.69	13.06	0.00	14.76
37.4	1.87	15.89	0.00	17.76
38.1	2.01	18.71	0.00	20.73
38.7	2.15	20.83	39.55	62.53
39.4	2.26	22.60	55.79	80.65

9.4.4.1 *QUANTITY*

For quantity purposes, the pond should be designed to reduce the post-construction peak flow from the chosen storm event to the preconstruction level, and it should be able to pass the 100-year storm safely. To control these storms, the basin storage should be equal to the area between the pre- and post-construction hydrographs. After a storage volume has been determined for each event, a 2- and a 10-year storm should be routed through the facility to ensure that the peak flows from the post-construction watershed are not greater than the corresponding pre-construction peak flows. Finally, a 100-year storm should be routed through the facility to ensure that the embankment will not be damaged or fail during the passage of that storm. It is very common to have several outlets to control the different storms-one for a 2-year storm; one for a 10-year storm; and an emergency spillway to control larger events, including the 100-year storm. To improve the efficiency of the outlet, it may be necessary to include an anti-vortex device.

9.4.4.2 *OUTLETS*

Outlets for dry basins can be designed in a wide variety of configurations. Most outlets use riser pipes of concrete or corrugated metal. These risers can be designed to control different storms through the use of several orifices on the riser; for example, a small diameter to control the Water Quality Volume (WQV), an orifice to control a 2-year storm and a larger orifice to control a 10-year storm. This larger flow is usually controlled by stormwater flowing in through the top of the riser, using the entire riser diameter. In the latter case, an anti-vortex design may be necessary. Larger flows are usually handled by an emergency spillway. Because the WQV outlet must be small to detain the WQV long enough, it can be easily clogged; thus, a minimum size of 3 inch should be used. To prevent clogging, a trash rack may be included in the design to cover the orifices.

9.5 REFERENCES

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(2) Design of Small Dams. U.S. Department of the Interior, 1977

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(4) Technical Release No. 55 (TR-55), User's Manual, Urban Hydrology for Small Watersheds, Natural Resources Conservation Service (Soil Conservation Service), U.S. Department of Agriculture, June 1986

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WEST VIRGINIA
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DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 10:
BRIDGES

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER 10. BRIDGES

10.1 INTRODUCTION AND CONCEPTS

This chapter pertains to the hydraulic analysis and design procedures for bridges over waterways. The information presented here can be used to design waterway openings and analyze backwater conditions. The design of bridge deck drainage is addressed in Chapter 5 (Storm Drainage Systems).

10.1.1 DEFINITION

Bridges are defined as:

- Structures that transport traffic over waterways or other obstructions;
- A part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure; and
- Structures with a centerline span of 20 feet or more. However, for the purpose of this chapter, all structures designed hydraulically as bridges should be treated as bridges, regardless of their length.

10.1.2 HYDRAULIC DESIGN CONSIDERATIONS

Proper hydraulic analysis and design is as vital as the structural design. A bridge carrying a highway over a watercourse must as a minimum, pass the design storm without overtopping the roadway or causing damage to the structure. This is to assure that frequent overtopping does not interrupt roadway service. The frequency of allowable overtopping is a function of the roadway classification and the availability of alternative routes in the event the roadway is overtopped and traffic flow is interrupted.

Serviceability of the bridge and roadway must be balanced against flooding concerns. In certain situations, it may be more feasible to size replacement drainage structures to carry less than the design discharge. A replacement structure, where approach work is limited by physical or monetary restraints, should convey discharges in such a way as to not create a backwater elevation higher than that created by the existing structure. In many such cases, replacement structures or the approach roadway will be inundated to convey expected discharges.

Careful consideration must be given to the effects of increased frequency and depth of flooding on residential, commercial, industrial, agricultural, and recreational property. While it would be preferable not to cause any adverse impacts from backwater for a range of storms including the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year storms, this is not always possible or economical. Minor increases in flood levels may be acceptable for some properties. However, significant increases in the frequency or depth of flooding will not.

For example, a property that is subject to flooding once every 50 years on average before the project may be able to sustain a minor increase in the 50-year flood level without major economic hardship if the frequency of flooding has not increased significantly. However, considerable economic hardship would occur for a property that was flooding once every 25 years on average before the project is now flooded once every 2 years on average as a result of the project, regardless of the increase in the 25-year flood level.

Bridges should be designed to minimize cost subject to the design criteria for the desired level of hydraulic performance in accordance with acceptable risk.

10.1.3 Common NFIP Acronyms

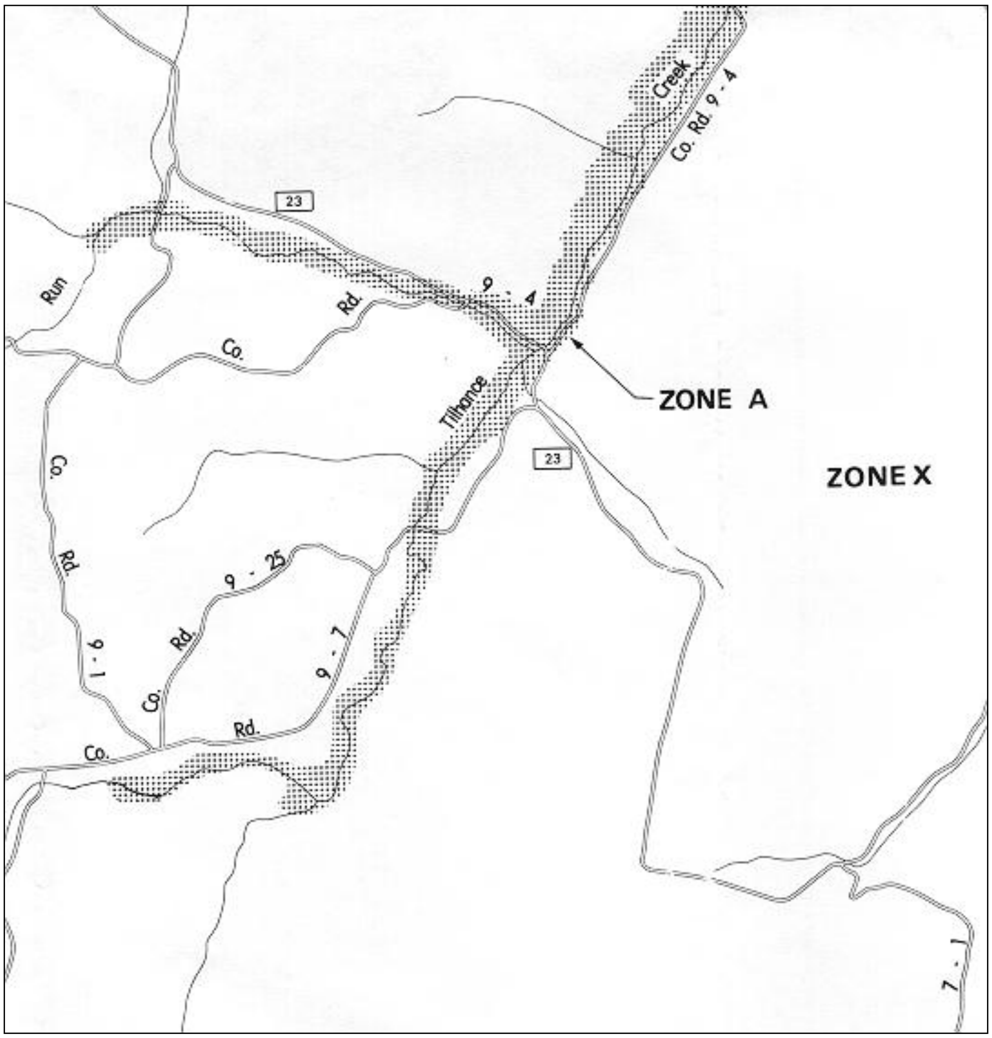
The designer should be familiar with the following common acronyms:

1. *Special Flood Hazard Area (SFHA)*: An area of land that would be inundated by the base flood shown on Flood Hazard Boundary Maps (FHBMs), Flood Insurance Rate Maps (FIRMs), and Flood Boundary and Floodway Maps (FBFMs).
2. *Flood Insurance Study (FIS) Report:* A report that describes the detailed methodologies and procedures used in developing the flood risk information shown on the FIRM and FBFM. Detailed methodologies include hydrology, hydraulics, and floodplain mapping. The FIS report usually contains the profiles for the 10-, 50-, 100-, and 500-year floods.
3. *Base Flood*: A flood having a 1-percent chance of being equaled or exceeded in any one given year. Commonly referred to as the 100-year flood.
4. *Base Flood Elevation (BFE):* The water surface elevation associated with the 100-year flood profile, usually referenced to either the National Geodetic Vertical Datum of 1929 (NGVD 29) or the North American Vertical Datum of 1988 (NAVD 88).
5. *Design Discharge:* Discharge that must be safely conveyed through a bridge or culvert without overtopping the road. The design discharge is described by a return interval (e.g., 50-year flood) and is a function of the highway classification. (See Chapter 4, Section 4.3.1)
6. *Flood Hazard Boundary Map (FHBM):* The initial flood map issued by FEMA that identifies, on the basis of approximate analysis, the areas of 100-year

flood hazard in a community. The FHBM shows floodplain boundaries, but no BFE's or floodways (Figure 10-1).

Figure 10-1

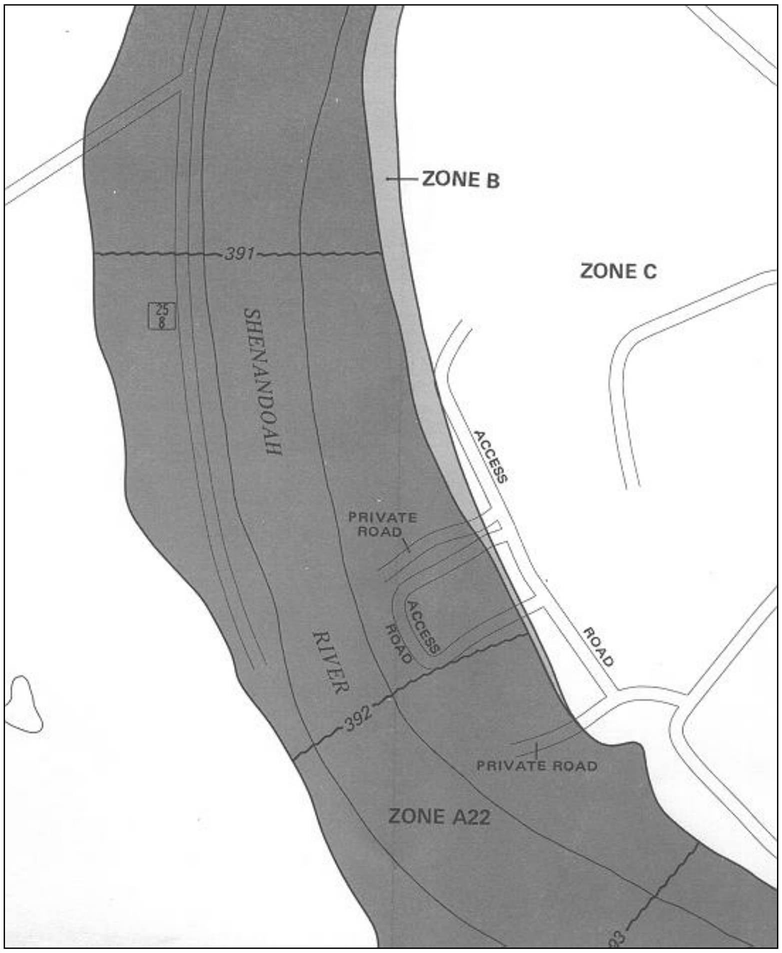
Example FHBM: Berkeley County, WV



7. *Flood Insurance Rate Map (FIRM):* A map produced by the Federal Emergency Management Agency (FEMA) which depicts flood risk information including the 100-year floodplain and other flood risk zones and base flood elevations (BFEs). Versions of this map published later than 1986, combined information from both the FIRM and FBFM. The base (100-year) floodplains on pre-1986 FIRMs were designated by dark-shaded areas (Zones A, A1-A30, A99, AO, AH, AR, V, V1-V30) as shown in Figure 10-2. The 500-year floodplains were designated by lighter-shaded areas (Zone B). Post-1986 FIRMs show simplified flood insurance zone designations. The previous Zones A1-A30 and V1-V30 were replaced by the designations AE and VE; and

Zones B and C were replaced by Zone X. The 500-year floodplain is still shown as "shaded" portions of Zone X.

Figure 10-2
Example FIRM: Jefferson County, WV



Base Flood Elevations (BFEs) are shown as wavy lines across the base floodplain.

8. *Flood Insurance Rate Map Zones:*

Zone A: The 100-year or base floodplain. There are six types of A Zones:

A — The base floodplain mapped by approximate methods; BFEs are not determined. This is often called unnumbered A Zone or an approximate A Zone.

A1-30 — These are known as numbered A Zones (e.g., A7 or A14). This is the base floodplain where the FIRM shows a BFE (old format)

AE — The base floodplain where base flood elevations are provided. AE Zones are now used on new format FIRMs instead of A1-A30 Zones.

AO The base floodplain with sheet flow, ponding, or shallow flooding. Base flood depths (feet above ground) are provided.

AH Shallow flooding base floodplain. BFEs are provided.

A99 Area to be protected from base flood by levees or Federal Flood Protection Systems under construction. BFEs are not determined.

AR The base floodplain that results from the decertification of a previously accredited flood protection system that is in the process of being restored to provide a 100-year or greater level of flood protection.

Zone B and Zone X (shaded): Area of moderate flood hazard, usually the area between the limits of the 100-year and 500-year floods. B Zones are also used to designate base floodplains of lesser hazards, such as areas protected by levees from the 100-year flood, or shallow flooding areas with average depths of less than one foot or drainage areas less than 1 square mile.

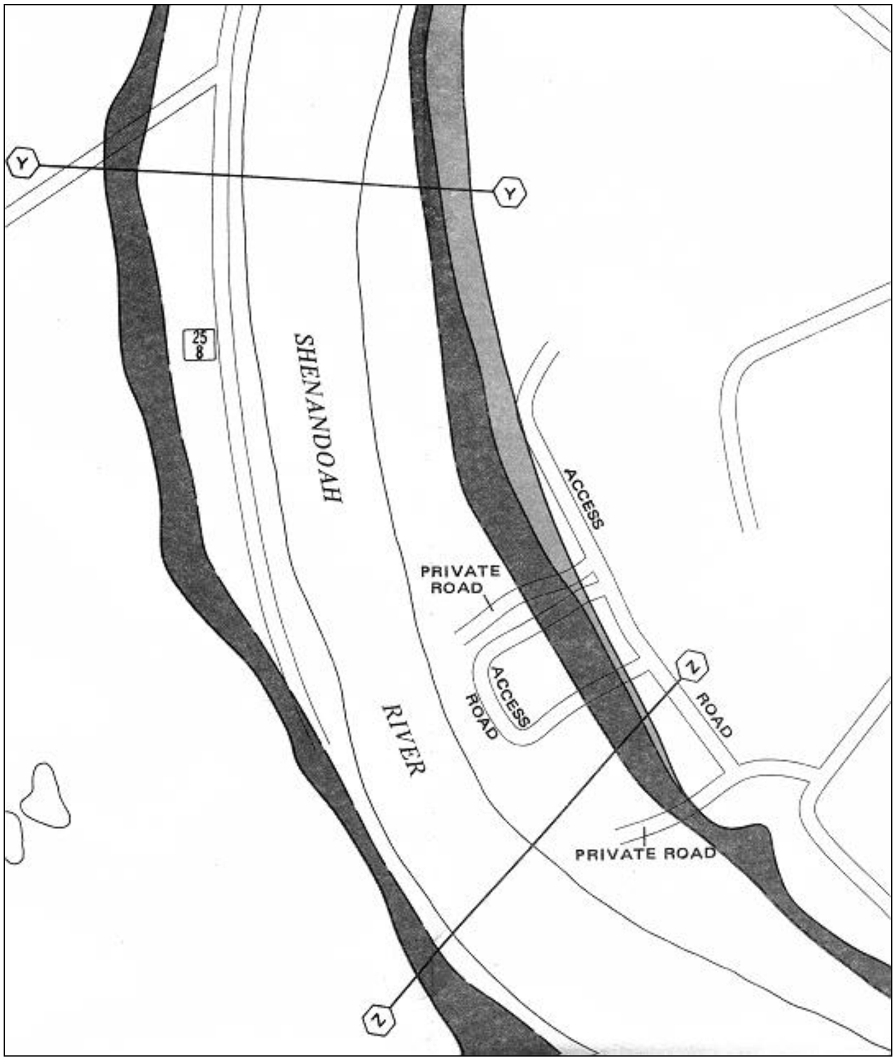
Zone C and Zone X (unshaded): Area of minimal flood hazard, usually depicted on FIRMs as above the 500-year flood level. Zone C may have ponding and local drainage problems that don't warrant a detailed study or designation as base floodplain. Zone X is the area determined to be outside the 500-year flood and protected by levee from 100-year flood.

Zone D: Area of undetermined but possible flood hazards.

9. *Flood Boundary and Floodway Map (FBFM):* A map issued by FEMA that depicts the boundaries of the 100-year floodplain and the regulatory floodway on the basis of detailed analysis (see Figure 10-3). This map also shows the location of many of the cross-sections used in the hydraulic analysis that were used to establish the BFEs. FBFMs were published prior to 1986. Since 1986, the information on the FBFM is being published on FIRMs.

Figure 10-3

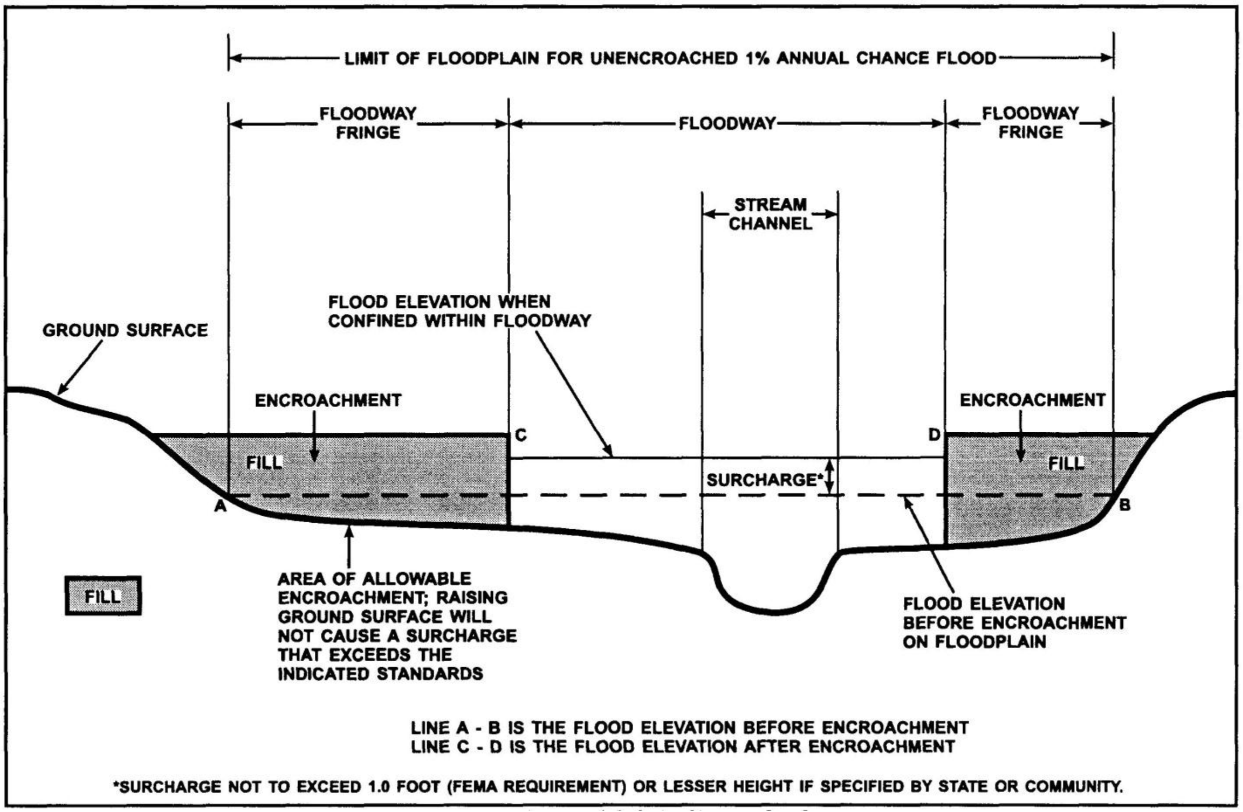
Example FBFM: Jefferson County, WV



10. *Floodway:* A regulatory floodway adopted into a community's floodplain management ordinance, is the stream channel plus that portion of the overbanks that must be kept free from encroachments in order to discharge the base flood without increasing flood levels more than 1.0-foot (see Figure 10-4).

Figure 10-4

FEMA Regulatory Floodway Concept



11. *Floodway Fringe:* The portion of the floodplain between the floodway boundary and the base flood boundary.

12. *Conditional Letter of Map Revision (CLOMR):* A CLOMR is FEMA's formal review comment whether a proposed project if constructed would comply with the minimum NFIP floodplain management criteria and warrant a revision to the FIRM.

13. *Letter of Map Revision (LOMR)*: A LOMR is an official revision to the currently effective FEMA map. It consists of a letter with an attached copy of the FIRM, annotated to show changes to the BFEs, floodplain boundary, or regulatory floodway based on revised conditions.

14. *Physical Map Revision (PMR):* An official republication of a community's NFIP map to effect changes to the BFEs, floodplain boundaries, regulatory floodways, or planimetric features. These changes typically occur as a result of structural works or improvements, annexations resulting in additional flood hazard areas, or corrections to the BFEs or SFHAs.

10.1.4 BRIDGE SCOUR CONSIDERATIONS

Reasonable and prudent hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge's vulnerability to undermining due to potential scour. Because of the extreme hazard and economic hardships posed by a rapid bridge collapse, special considerations must be given to selecting appropriate flood magnitudes for use in the analysis. The hydraulics engineer must always be aware of and use the most current scour forecasting technology.

FHWA issued a Technical Advisory (TA 5140.23) in October 1991 requiring a scour evaluation for existing and proposed bridges over waterways. Refer to Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges (HEC 18) for a thorough discussion on scour and scour prediction methodology. A companion FHWA document to HEC 18 is Hydraulic Engineering Circular No. 20 Stream Stability at Highway Structures (HEC 20).

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multi-level solution procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with a quantitative analysis using basic hydrologic, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses (e.g., evaluation of watershed sediment yield, incipient motion analysis, and scour calculations). This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis, based on detailed mathematical modeling and/or physical hydraulic models, should be considered. This multi-level approach is presented in HEC 20.

Less hazardous perhaps are problems associated with aggradation. Where freeboard is limited, problems associated with increased flood hazards to upstream property or to the traveling public due to more frequent overtopping may occur. Where aggradation is expected, it may be necessary to evaluate these consequences. Aggradation is sometimes referred to as negative scour.

10.2 DESIGN POLICY

This section presents policies that are unique to the hydraulic design of bridges. For new and replacement bridges, 0.00 backwater increase is always the goal. However measures to limit any increase in the risk of flood damage to property and structures must be balanced with other competing interests.

- Minimizing social, economic, and environmental impacts.
- Compliance with the requirements of the National Flood Insurance Program (NFIP) administered by the Federal Emergency Management Agency (FEMA).
- As far as practicable, bridges in floodplains shall be designed to avoid significant encroachments that could result in the interruption or termination of the highway facility that serves as the only evacuation route during an emergency.
- The hydraulic design of bridge waterway openings, road grade, bridge scour estimates, and abutment protection shall consider the potential risks to traffic, adjacent property, and the environment.
- The design discharge is normally one or more of the customarily documented events (i.e. the 2, 5, 10, 25, 50, 100, & 500-year floods) that will pass under the bridge superstructure at its lowest elevation with the minimum freeboard, provided that level of protection is acceptable to the bridge designer.
- The discharges obtained from the effective FIS for the community shall be utilized for hydraulic design provided they are representative of current site conditions. The selected design discharges shall reflect consideration of traffic service level, environmental impact, property damage, hazard to human life and floodplain management criteria.
- Bridges that exceed the maximum backwater increases allowed by FEMA for waterways mapped under the National Flood Insurance Program (NFIP) due to special circumstances shall require approval by the Division Director.
- Bridge pier spacing, pier orientation and the abutments shall be designed to minimize flow disruption and potential scour.
- Bridges should be designed to operate with a minimum freeboard for the passage of the design discharges and floating debris to account for the risk associated with the hydraulic computations.
- The bridge foundations shall be positioned below the total estimated scour depth to prevent bridge failure by scour. The total scour estimate shall consist of long-term degradation or aggradation of the riverbed as well as contraction and local scour.
- Evaluation of various bridge alternatives shall be accomplished by using an approved backwater program such as the Corps of Engineers' HEC-RAS.

10.3 DESIGN CRITERIA

The following design criteria shall apply to the hydraulic design of bridges:

10.3.1 BACKWATER INCREASES AND NFIP REQUIREMENTS

Due to economic reasons, bridges usually encroach upon the floodplain with the roadway embankment and span the stream channel and a portion of the over-banks. This is true for both new bridges as well as a bridge replacement projects.

Bridges shall be designed such that they do not cause increases in backwater that would have adverse impacts on the surrounding community. Small increases are sometimes unavoidable. 0.00 backwater increase is always the goal. Always evaluate increased risk of flood damage to property and structures. If the water surface elevation will be increased, consider the need for buying permanent drainage easements.

Designers shall conform to FEMA's National Flood Insurance Program (NFIP) regulations for sites located in the Special Flood Hazard Areas (SFHA) shown on the Flood Insurance Rate Maps (FIRMs) for the community. FEMA regulations and sound engineering practice require coordination with the community's floodplain administrator on all projects that may potentially affect the BFEs on a stream. FEMA requirements depend on the level of study that has been performed for a given stream.

Streams with BFEs and Regulatory Floodway

The NFIP regulations do not permit any increase in the BFEs (i.e., 0.00 foot or zero) for construction within the regulatory floodway. Exceptions to this requirement must be processed through a CLOMR request to FEMA. All impacts must be evaluated and affected property owners must be notified.

Construction in the floodway fringe may be allowed to cause up to 1-foot of cumulative increase in the BFE, also known as the floodway surcharge. The surcharge is defined as the cumulative effect of the proposed bridge combined with

all other existing and anticipated development since the original NFIP study was completed. The published floodway surcharge shown in the FIS may be less than 1-foot in many places. Therefore, some, or all, of the allowable 1-foot surcharge may already be used up by previous development in the floodway fringe. The designer is cautioned that previous development may or may not be properly documented.

Streams with BFEs but without Regulatory Floodway

A hydraulic analysis is required to demonstrate that the proposed bridge causes no more than 1-foot of cumulative increase in the BFEs. BFE increases greater than 1-foot must be approved through the CLOMR process.

Streams without BFEs: Studied by Approximate Methods

BFEs shall be developed for streams studied by approximate methods (i.e., the discharges and water surface elevations have not been published in the FIS) to ensure that the proposed bridge causes no more than 1-foot of cumulative increase in the BFEs including the effect of the proposed bridge. BFE increases greater than 1-foot must be approved through the CLOMR process.

Unstudied Streams

A hydraulic analysis shall be performed to ensure that increased water surface elevations will not adversely affect nearby property. A CLOMR is not required.

Conditional Letter of Map Revision (CLOMR)

Bridge encroachments may be allowed to cause BFE increases beyond the NFIP allowances if a CLOMR request is submitted for a BFE and/or floodway revision, and the following provisions of 44 CFR 65.12 are satisfied:

- An evaluation demonstrating why other alternatives, which would not result in allowable BFE increases are not feasible.
- Documentation of individual legal notice to all impacted property owners.
- Concurrence of the Chief Executive Officers of the affected communities.
- Certification that no structures are affected by the BFE increases.

CLOMR requests should be prepared and submitted using FEMA's MT-2 Forms, which can be downloaded from FEMA's website at http://www.fema.gov/fhm/dl_mt-2.shtm.

Coordination with the community's floodplain manager is required in all cases. It should be noted that FEMA approval does not absolve the Highway Department's responsibility for any increased flooding. Therefore, zero increase is the goal.

10.3.2 FREEBOARD

Where practical, a minimum freeboard of two feet should be provided between the design storm water surface elevation and the lowest portion of the superstructure for the design discharge. Where this is not practical, the bridge designer should establish the freeboard based on the desired level of protection.

10.3.3 FLOATING DEBRIS CONSIDERATIONS

Floating debris that becomes trapped on a bridge can cause substantial damage through increased flooding and scour. If piers or abutments will be placed in the active channel or floodplain, then the potential for floating debris should be evaluated. Streams with steep soil banks generally have a greater likelihood for receiving floating debris. Piers in the active channel should be avoided.

10.3.4 BRIDGE SCOUR

Scour potential shall be considered in all designs. Whenever necessary, adequate details shall be incorporated into the design and plans to minimize the effects of scour. A Form DS-34 shall be completed during the design phase of the project. Scour calculations are based on the 1% annual chance flood (Q100) and the 0.2% annual chance flood (Q500). Form DS-34 is available in the Bridge Design Manual, Appendix C-3. Scour depth estimates shall be based upon the guidance provided in the following FHWA publications:

- *Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18, (HEC 18)*
- *Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20 (HEC-20)*

The above publications can be downloaded from FHWA's Internet website at http://www.fhwa.dot.gov\engineering\hydraulics\library_listing.cfm.

Spread footings shall be embedded at least one foot into non-scourable rock. For pile foundations, the bottom of the pile cap shall be below the scour depth. Drilled shaft foundations shall be analyzed to ensure stability without soil support. Whether rock is considered scourable is currently determined by the rock quality designation (RQD). If the RQD is greater than 50%, then the rock is considered non-scourable. The engineer and the geologist should also consider the type of rock. Further research is ongoing.

10.3.5 HYDRAULIC ANALYSIS METHODS

A one-dimensional step-backwater computer program is usually acceptable to perform the hydraulic analysis for most situations. Any computer program currently approved by FEMA may be used but the Division prefers that the latest version of the Corps of Engineers' HEC-RAS computer program be used.

10.3.6 ROCK PROTECTION

Rock shall not to be used for scour protection at piers for new bridges. Rock may be used to protect exposed abutment slopes or as a scour countermeasure at existing bridge piers and abutments. Design guidelines for placement and sizing of rock are presented in the following publications:

- *Design of Riprap Revetment, Hydraulic Engineering Circular No. 11, (HEC 11)*
- *Bridge Scour and Stream Instability Countermeasures, Hydraulic Engineering Circular No. 23 (HEC 23)*

The above publications can be downloaded from FHWA's Internet website at http://www.fhwa.dot.gov\bridge\hydpub.htm.

10.4 HYDRAULIC DESIGN PROCEDURE

10.4.1 REQUIRED DATA

The data necessary to perform a hydraulic analysis typically includes topographic maps, aerial photographs, roadway plans, profiles, and typical sections to cover the width of the floodplain in the vicinity of the crossing. The designer should refer to Chapter 7 for detailed guidance on obtaining backup data from the FEMA Library.

Flood insurance studies (FIS) and some FIRMs for the entire State organized by District are available on the dot shared server. WVDOH project managers will obtain these files for their consultants. The flood studies are in PDF format and the maps are TIF format. One complete set of full size paper flood maps is available in the Hydraulic & Drainage Unit of the Engineering Division.

10.4.2 STREAM SURVEYS

Cross-section surveys should include shots at breaks in the grade, edge of water, thalweg (deepest part of the channel) and the top of bank. Surveyors trained to identify “Ordinary High Water” (OHW) and “bankfull” indicators and should include them in the survey.

Cross sections should be taken at the following locations:

- Immediately upstream and downstream of the existing bridge approach fills
- If the alignment is expected to change, Immediately upstream and downstream of the proposed bridge approach fills
- One bridge length or channel width (whichever is less) upstream and downstream of the bridge.

If an existing hydraulics model will be used, the cross sections adjacent to the bridge must be field verified in order to account for differences in survey datum, and aggradation or degradation of the stream. In this case, a total of six cross sections will be surveyed.

If an existing hydraulic model is not available or will not be used, then additional cross sections will be required. A total of eight to twelve cross sections should typically suffice.

- Include two additional cross sections upstream and downstream, spaced at one channel width apart.
- If the stream has pools and riffles, cut sections at each pool and at the head of each riffle to include at least two riffles upstream and downstream of the proposed and existing bridge.
- Include a section at each significant constriction or obstruction to the channel or floodplain. This may be several hundred feet from the bridge.
- Include nearby bridges.

All cross sections do not need to cross the entire flood prone area if it is fairly uniform. If the flood prone area is fairly uniform, then just three of the sections across the entire flood prone area might be sufficient. All other surveyed cross sections would extend just beyond the top of the stream bank. Flood prone area limits and elevations for intermediate cross sections can be interpolated. For very wide floodplains, the extent of the flood prone area may be determined from USGS maps.

A profile of the thalweg (the deepest part of the channel) must be surveyed from the most downstream section to the most upstream section. Include shots at each break in the profile grade and at regular intervals.

See Section 7.4.3 to determine boundary conditions and cross section locations.

10.4.3 HYDROLOGIC ANALYSIS

The Division prefers that, if available, the FEMA FIS discharges and hydraulic model be obtained and used for the analysis, if available. Otherwise, discharges should be developed using the methods outlined in Chapter 4 (Hydrology).

It is frequently necessary to use high flow (10-year, 50-year, 100-year, and 500-year discharges) as well as low flow (2-year, 5-year, normal water, and ordinary high water) discharges.

Hydrologic analyses require the determination of the contributing drainage area for the stream at the bridge crossing. The USGS has published drainage area measurements for major drainage basins in the State broken down to a scale of approximately 2 square miles. Contact the WVDOH Hydraulic & Drainage Unit for copies of the books containing these drainage areas. A limited number of copies are available.

10.4.4 HYDRAULIC ANALYSIS

A detailed hydrologic and hydraulic analysis should be performed for all new or replacement bridges and structures involving significant lateral encroachments due to placement of highway fill embankments within a floodplain. A hydraulic analysis is required for all bridged waterways regardless of whether it is within a FEMA or other officially delineated floodplain.

The bridge can be subject to either free-surface flow or pressure flow through one or more openings with possible embankment overtopping. Manual backwater calculations are impractical due to the interactive and complex nature of the computations. These hydraulic complexities are best analyzed using a one-dimensional step backwater computer program. The use of a two-dimensional numerical model should be approved by WVDOH.

The designer should refer to Chapter 7 for modeling guidance regarding study limits, boundary conditions, cross-section layout and spacing, Manning's Roughness values, and calibration.

Major culvert installations where the backwater may affect insurable structures or developable property, or that are located within a FEMA designated flood zone may need to be analyzed using a backwater program with the Division's approval.

The Division prefers a multi-step procedure for performing the hydraulic analysis as described below:

Duplicate Effective Model

The first step will be to mathematically reproduce the baseline hydraulic model (such as the effective FIS model) using a step-backwater computer program. It is not necessary to use the same computer program that was used to develop the effective model provided the differences can be explained. The designer should consider using any model or data available from FEMA. This hydraulic model will be referred to as the "DUPLICATE EFFECTIVE" model. The effective model may or may not be available from the FEMA library.

Corrected Effective Model

The second step would be to add or delete any cross sections necessary at the locations necessary to subsequently model the proposed bridge. Corrections to the modeling should also be made at this stage. Any changes made to the "CORRECTED EFFECTIVE" model are primarily for the purpose of facilitating the modeling of proposed conditions. If gaged flows or observed highwater marks are available, the corrected effective model may be calibrated by adjusting the Manning's roughness values.

Existing Conditions Model

The Duplicate Effective Model or Corrected Effective Model should be modified to produce the "EXISTING CONDITIONS" model to reflect any modifications that may have occurred within the floodplain since the date of the Effective model but prior to the construction of the bridge project. If no modification has occurred since the date of the effective model, then this model would be identical to the Corrected Effective Model or Duplicate Effective Model. The existing conditions model may be required to support conclusions about the actual impacts of the project associated with the revised or proposed model or to establish more up-to-date models on which to base the revised or proposed conditions model. This model becomes the basis for measurement of any changes that would take place as a result of the proposed construction.

Proposed Conditions Model

The proposed conditions hydraulic model(s) should include two separate models, the "PROPOSED PERMANENT CONDITIONS" and the "PROPOSED TEMPORARY CONDITIONS" models. Multiple models may be required for phased construction. The proposed conditions models are compared to the corrected effective or existing conditions models as appropriate, in order to determine the permanent and temporary effects from the project.

10.4.5 BRIDGE SCOUR

The seven specific steps are recommended in HEC-18 for estimating scour at bridges:

Step 1: Determine scour analysis variables.

Step 2: Analyze long-term bed elevation change.

Step 3: Compute the magnitude of contraction scour.

Step 4: Compute other general scour depths.

Step 5: Compute the magnitude of local scour at piers.

Step 6: Determine abutment foundation type, protection and elevation. Computation of local scour depths may be used to aid in this determination.

Step 7: Plot and evaluate the total scour depths.

The engineer should evaluate how reasonable the individual estimates of general scour (contraction or other) and local scour depths are in Steps 3, 4 and 5 and evaluate the reasonableness of the total scour in Step 7.

Perform the bridge foundation analysis on the basis that all streambed material in the scour prism above the total scour line has been removed and is not available for bearing or lateral support.

Finally repeat the procedure and calculate the scour for a superflood. It is recommended that this superflood (or check flood) be on the order of a 500-year event. However, flows greater or less than these suggested floods may be appropriate depending upon hydrologic considerations and the consequences of damage to the bridge. An overtopping flood less than the 500-year flood may produce the worst-case situation for checking the foundation design. The foundation design should be reevaluated for the superflood condition and design modifications made where required.

10.4.6 DOCUMENTATION

The designer should refer to the Division's recommended Hydrology and Hydraulics Report format in Chapter 3. Preliminary reports are generally only submitted on significant bridge projects that have a drawn Span Arrangement, Type, Size, and Location (TS&L). For most bridges, one submission is generally adequate.

10.5 HYDRAULIC DESIGN OF TEMPORARY CONSTRUCTION

10.5.1 BACKGROUND

The designer shall evaluate the risk of flood damage to property and structures for the temporary construction condition. Temporary construction features such as temporary bridge crossings (detour), causeways, and cofferdams are needed to provide construction access and to facilitate bridge construction. Impacts caused by construction features should be analyzed due to their effect on both low and high stream flow rates.

10.5.2 TEMPORARY BRIDGE CROSSINGS (DETOUR)

Flow Criteria

A temporary bridge or detour should be constructed in a manner to remain serviceable for at least the 10% (10-year) storm. In the case of a high ADT roadway, allowance for a more frequent flow criterion for serviceability is at the discretion of the designer.

Risk Criteria

A temporary bridge shall be designed such that it does not cause increases in the water surface elevation that would have adverse impacts on nearby property. Increases in water surface elevation due to the presence of a temporary bridge with the existing bridge and/or proposed bridge in place are usually unavoidable. The criterion for the amount of allowable elevation increase is to be determined by the designer through an examination of the surrounding area. This criterion shall be determined based on the type of land use and the presence and elevation of nearby structures. Thus the effects of flooding the surrounding area at the water surface elevation for the serviceable flow of the temporary condition set the risk criterion.

For example, if the presence of the temporary detour with the existing structure in place floods a nearby meadow or pasture field at the 10% (10-year) storm, the risk of adverse impacts is low. Moreover, if the same design situation floods a nearby outbuilding or garage for the 10% (10-year) storm, the risk of adverse impacts is high. In the second case the higher risk of adverse impacts shall warrant a more frequent flow criterion for serviceability of the temporary condition.

The designer should be mindful of the local floodplain managers' possible knowledge and familiarity of the surrounding area. The local floodplain manager may require consultation on the determination of this risk criterion.

10.5.3 CAUSEWAY DESIGN

The objective of a temporary causeway is to provide a design that is reasonably convenient, economical, and logistically feasible for the contractor to build and remove.

Temporary fills placed in or near watercourses shall be constructed in a manner to withstand expected high flows and shall consist of clean and course non-erodible materials with 15% or less like fines. The causeway's influence on flood flow elevations should also be checked for a range of flows from the OHW flow up to the 0.2% (500-year) storm.

The hydraulic analysis should consider in-stream obstructions such as piers or islands that could direct high velocity flows at points along the causeway.

The causeway shall be designed so that it will not significantly:

- Increase the water surface elevation for flows that could damage property
- Increase the velocity of flow through the causeway opening(s)
- Alter the usual flow distribution
- Direct flow at the piers and foundations which would subject them to forces for which they were not designed.

The top of the causeway should typically be set about 1 foot above the OHW level. If a causeway will completely cross the channel, then the largest possible pipe diameter should be placed through the causeway spaced at 0.5 times the diameter. Pipe size will vary with the depth of the stream.

10.5.4 NORMAL FLOW

USACE Nationwide 404 Permit states: "Appropriate measures must be taken to maintain near normal downstream flows and to minimize flooding." In order to ensure compliance with the normal flow requirement, an analysis of normal flow must be done when a construction project will have temporary works within the active channel.

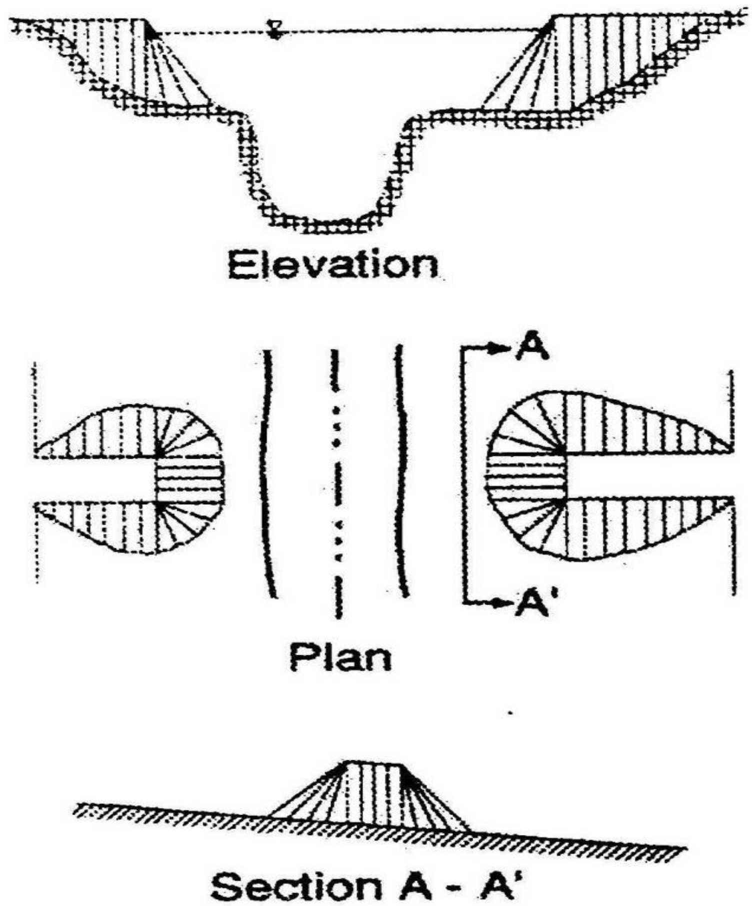
Normal flow is the water flow prevailing during the greater part of the year. Statistically, this is most closely represented by the "mode", which is the value or item occurring most frequently in a series of observations or statistical data. However, for the purpose of meeting the 404 permit requirements, statistical analysis will not be required to determine normal flow. On regulated rivers, such as the Ohio, normal flow and or stage can be obtained from the USACE. On unregulated rivers or streams, estimate the normal stage from field observations. Then determine the corresponding normal flow through calibration by using HEC-RAS.

10.6 BRIDGE ABUTMENT PROTECTION

Rock is frequently used for protection of the earthen fill slopes in spill-through abutments. A spill-through abutment is a bridge abutment having a fill slope on the stream ward side as shown in Figure 10-5. In such situations, rock protection serves the two-fold purpose of protecting the underlying abutment shelf against runoff coming from the approach roadway and bridge superstructure as well as from scouring due to impinging flow from floodwaters. Rock can also be used around tall abutments on spread footings to protect against scour.

Figure 10-5

Spill-Through Abutment



Source: HEC-18, 4TH Edition, FHWA

10.7 REFERENCES

1. Code of Federal Regulations (CFR), National Flood Insurance Program, Title 44 – Emergency Management and Assistance, Chapter I – Federal Emergency Management Agency, Part 60 - Criteria for Land Management and Use, Subpart 60.3 (Floodplain Management Criteria for Flood-prone Areas).

2. Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges (HEC 18), FHWA

3. Hydraulic Engineering Circular No. 20 Stream Stability at Highway Structures (HEC 20).

4. Model Drainage Manual, AASHTO, 2005.

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

APPENDIX A
GLOSSARY

APPENDIX A: GLOSSARY

OVERVIEW

This glossary is an abridged version of the glossary contained in the AASHTO Model Drainage Manual. It is divided into two parts:

- Overview
- Glossary

The terms are not intended to be rigorously accurate or complete. A particular term can have several meanings. The primary purpose of this glossary is to define drainage terms in a manner that makes them easier to interpret and understand. A lesser purpose is to provide a compendium of terms that will be useful for both the novice and the more experienced hydraulics engineer. It is hoped that this will help improve communication between the highway hydraulics engineer and others.

Some terms are explained and defined in several ways-sometimes with considerable detail. This was done intentionally for several reasons:

- to facilitate better understanding,
- to highlight actual or apparent contradictions in current terminology, and
- to avoid or minimize litigation problems from overly restrictive definitions.

As often happens in any science, some practitioners have different names for the same thing. An attempt has been made to sort out these colloquialisms and synonyms and assign all the definitions to one term.

GLOSSARY OF TERMS

ABSTRACTION. That portion of rainfall that does not become runoff. It includes interception, infiltration, and storage in depressions. It is affected by land use, land treatment and condition, and antecedent soil moisture.

ABUTMENT. The superstructure support at either end of a bridge or similar type structure; usually classified as spillthrough or vertical. Considered part of the bridge substructure. See Spillthrough Abutment and Vertical Abutment.

ADSORPTION. The adhesion in an extremely thin layer of molecules (such as gases, solutions, or liquids) to the surface of solid bodies or liquids with which they are in contact. Compare with Absorption.

AGGRADATION. General and progressive up-building of the longitudinal profile of a channel or within a drainage facility by the deposition of sediment. Permanent or continuous aggradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place.

ALERT. Acronym for Automated Local Evaluation in Real Time. An automated, local flood warning system consisting of automatic self-reporting river and rainfall gages, a communications system based on line-of-sight radio transmission of data, and a base station. The base station consists of radio receiving electronic equipment and a microprocessor. Data analysis software is available to collect, quality control and display data. A hydrologic model to provide simulation of streamflow is also available.

ALLOWABLE HEADWATER DEPTH. The depth or elevation of the flow impoundment for a drainage facility above which damage, some other unfavorable result, or a significant flood hazard could occur.

ALLUVIAL. Referring to deposits of silts, sands, gravels, or similar detrital material that has been transported by running water.

ALLUVIAL CHANNEL. Channel formed wholly in alluvium with no bedrock exposed in the channel at low flow or likely to be exposed by erosion. A channel whose processes are controlled by the flow and boundary interactions.

ALLUVIAL FAN. A fan-shaped deposit formed where a fast flowing stream flattens, slows, and spreads typically at the exit of a canyon onto a flatter plain

ANABRANCH. An anabranch is a section of a river or stream that diverts from the main watercourse channel (or main stem) and rejoins the main stem downstream. In the simplest case, an island or rock in the river creates a main course and an anabranch course; a more significant anabranch would diverge for a distance of several feet before rejoining. River deltas branch into large numbers of courses, though these are not normally regarded as anabranches, as the net result is usually multiple discharge points rather than a rejoined unified flow.

ANGLE OF REPOSE. The maximum angle, as measured from the horizontal, at which granular particles can stand.

ANTECEDENT MOISTURE CONDITION (AMC). The degree of wetness of a watershed's surface soils at the beginning of a storm.

APPROACH CHANNEL. The reach of channel upstream from a dam, bridge constriction, culvert, or other drainage structure.

APRON. Protective material laid on a stream bed to prevent scour commonly caused by some drainage facility. More specifically, a floor lining of such things as concrete, timber, and riprap, to protect a surface from erosion, such as the pavement below chutes, spillways, at the toes of dams, or at the outlet of culverts. Material placed on the banks is commonly termed a blanket.

ARID. Geographic areas that are dry, lacking moisture.

ARMORING. A natural process whereby an erosion-resistant layer of relatively large particles is formed on a channel bank and/or channel bed due to the removal of finer particles by streamflow; i.e., the concentration of a layer of stones on the bed of the stream that are of a size larger than the transport capability of the recently experienced flow—the winnowing out of smaller material capable of being transported while leaving the larger sizes as armor that, for discharges up to that point in time, cannot be transported. Armoring may also refer to the placement of a covering on a channel bank and/or channel bed to prevent erosion.

AVULSION. A sudden change in channel course that occurs when a stream suddenly breaks through its banks; usually associated with a large flood or a catastrophic flood or event.

BACKFILL. The material used to refill a ditch or other excavation, material placed adjacent to or around a drainage structure, or the process of doing so.

BACKWATER. The increase in water surface elevation induced upstream from such things as a bridge, culvert, dike, dam, another stream at a higher stage, or other similar structures or conditions that obstruct or constrict a channel relative to the elevation occurring under natural channel and floodplain conditions. Stated another way, water backed up or retarded in its course as compared with its normal or natural conditions of flow. Also applies to the water surface profile in a channel or conduit.

BACKWATER CURVE. A particular form (or profile) of the surface curve (water surface) of a stream of water that is concave upward. It is caused by an obstruction in the channel such as those that cause backwater. The depth is greater at all points under the curve than the critical and the normal depth, and the velocities diminish downstream. The term is sometimes used in a generic sense to denote all water surface curves or profiles. Compare with Backwater and Water Surface Profile.

BAFFLE. A structure constructed on the bed of a stream or drainage facility to deflect or disturb the flow. Vanes or guides, a grid, grating, or similar device placed in a conduit to check eddy currents below them, and effect a more uniform distribution of velocities. Also a device used in a culvert or similar structure to facilitate fish passage.

BANK. The side slopes or margins of a channel between which the stream or river is normally confined. More formally, the lateral boundaries of a channel or stream, as indicated by a scarp, or on the inside of bends, by the streamward edge of permanent vegetal growth.

The margins of a channel. Banks are called right or left as viewed facing in the downstream direction.

BAR, ALTERNATE. An alluvial deposit of sand and gravel lacking permanent vegetal cover occurring in an alternating pattern from bank to bank in a relatively straight channel reach.

BAR, MIDDLE. A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.

BAR, POINT. An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop bendway and usually somewhat downstream from the apex of the loop. The longer, tapered portion, when present, commonly points upstream.

BASE. The layers of specified material placed on the sub-base or subgrade to support the pavement, surface course, or a drainage facility.

BASE FLOODPLAIN. Surface area flooded by the base flood.

BASIN, DETENTION. A basin or reservoir incorporated into the watershed whereby runoff is temporarily stored, thus attenuating the peak of the runoff hydrograph. A stormwater management facility that impounds runoff and temporarily impounds runoff and discharges it through a hydraulic outlet structure to a downstream conveyance structure.

BED. The bottom of a channel. The part of a channel not permanently vegetated which is bounded by banks and over which water normally flows.

BED FORM. A recognizable and commonly transient relief feature on the bed of a channel, such as a ripple, dune, or bar. A relief feature caused by water flowing over a mobile material such as sand and gravel.

BED LAYER. A flow layer, several grain diameters thick (usually two) immediately above the bed.

BED, INDURATED. A channel bed that has been made hard or has hardened.

BED LOAD. Sediment that is transported in a stream by rolling, sliding, or skipping (saltating) along the bed or very close to it; considered to be within the bed layer. The quantity of silt, sand, gravel, or other detritus rolled along the bed of a stream, often expressed as weight or volume per time.

BED LOAD DISCHARGE. The quantity of bed load passing a cross section of a stream in a unit of time.

BED MATERIAL. The sediment mixture of which a stream bed, lake, pond, reservoir, or estuary bottom is composed. Sediment consisting of particle sizes large enough to be found in appreciable quantities at the surface of a channel bed. Material found on the bed of a channel (may be transported as bed load discharge or sediment discharge).

BED SEDIMENT DISCHARGE. The part of the total sediment discharge that is composed of grain sizes found in the bed and is equal to the transport capability of the flow occurring at the time.

BED SHEAR. See Tractive Force.

BED SLOPE. The longitudinal inclination of a channel bottom.

BEDROCK. The scour-resistant material underlying erodible soils and overlying the mantle rock, ranging from surface exposure to depths of several hundred kilometers [miles].

BLANKET. Material covering all or a portion of a channel bank to prevent erosion. Stream bank surface covering, usually impermeable, designed to serve as protection against erosion. Common pavements used on channel banks are concrete, compacted asphalt, and soil-cement.

BOULDER. A rounded or angular fragment of rock, the diameter of which is in the size range of 250 to 4000 mm (10 to 160 in.) according to FHWA's Highways in the River Environment Manual.

BRIDGE. A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a tract or passageway for carrying traffic or moving loads, and, for definition purposes (AASHTO), having an opening measured along the center of the roadway equal to or more than 20 ft (6.1 m) between undercopings of abutments or spring lines of arches, or extreme outside ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening. Also a structure that, while designed hydraulically using the principles of open channel flow to operate with a free water surface, may be inundated under flood conditions. The structure generally consists of a deck or superstructure supported on two or more abutments and often includes intermediate piers.

BRIDGE OPENING. The total cross section area beneath a bridge superstructure that is available for the conveyance of water.

BRIDGE, RELIEF. An opening through an embankment located on a floodplain for the purpose of permitting passage of overbank flow.

BRIDGE WATERWAY. The area of a bridge opening available for flow as measured below a specified stage and normal to the principal direction of flow.

CALIBRATION. The process of fitting a [computational] model to a set of observed data by changing unknown or uncertain model parameters systematically within their allowable ranges until a “best fit” of the model to the observed data is achieved.

CANAL. A constructed open conduit or channel for the conveyance of irrigation water that is distinguished from a ditch or lateral by its larger size. It is usually excavated in natural ground, although lined canals on berms are not uncommon. Compare with Channel and Swale.

CAPACITY. A measure of the ability of a channel or conduit to convey water.

CAPACITY CURVE. A graph of the volume of such things as a reservoir, tank, or detention pond as a function of elevation. The capacity of a reservoir is defined by reference to an elevation.

CAUSEWAY. Rock or earth embankment carrying a roadway across water.

CHANNEL. The term “channel” has been defined numerous ways: (1) the bed and banks that confine the flow of surface water in a natural stream or artificial channel; also see River and Stream; (2) the course where a stream of water runs or the closed course or conduit through which water runs, such as a pipe; (3) An open conduit either naturally or artificially created that periodically or continuously contains moving water or which forms a connecting link between two bodies of water. River, creek, run, branch, anabranch, [arroyo, draw, wash] and tributary are some of the terms used to describe natural channels. Natural channels may be single or braided (see Braiding of River Channels). Canal [ditch, lateral] and floodway are some of the terms used to describe artificial channels.

CHANNEL, COLLATERAL. Channels that are side-by-side; accompanying; from the same source; or similar but of subordinate nature.

CHANNEL CONTRACTION. The degree of contraction imposed by a bridge type constriction on the river channel for a given discharge. It is measured by the channel-contraction ratio (M), that is defined as the ratio of that portion of flow directed at the contracted opening divided by the total flow that must pass through the contracted opening. Sometimes the inverse of this ratio is used to define the channel contraction, so caution is warranted. Compare with Constriction, Contracted Section, Contraction, and Coefficient of Contraction.

CHANNEL DIVERSION. The taking of water from a stream or other body of water into a canal, pipe, or other conduit. The removal of all or a portion of the flow from a natural or artificial (canal, ditch, field ditch, or lateral) channel.

CHANNEL LINING. The material applied to the bottom and/or sides of a natural or constructed channel. Material may be such things as concrete, sod, grass, rock, or any of several other types of commercial linings. Covering of stones on a channel bed or bank (used in the AASHTO Model Drainage Manual with reference to natural covering).

CHANNEL, LOW-FLOW. Lower portion of natural or artificial watercourse often of perceptible extent with a definite bed and banks that confines and conducts continuously or periodically flowing water. A low flow channel is considered as that portion of a channel commonly lying below the plane of the ordinary highwater (OHW). A low-flow channel may be adjoined by a floodplain. The bankfull capacity is often associated with the dominate discharge or mean annual discharge.

CHANNEL MIGRATION. Change in position of a channel by lateral erosion of one bank and the simultaneous accretion of the opposite bank. Systematic channel shifting in the direction of flow.

CHANNEL, NATURAL. A surface or underground watercourse created by natural agents and conditions. The principal stream channel or channels and, if the stream is braided, its natural and customary overflow channels.

CHANNEL, NON-UNIFORM. A channel where the flow streamlines are not straight and parallel. The velocity vector varies significantly with distance along a flow streamline at a given instance.

CHANNEL, OPEN. A channel having a water surface exposed at all points to atmospheric pressure. Any conveyance in which water flows with a free surface.

CHANNEL PATTERN. The aspect of a stream channel in plan view, with particular reference to such things as the degree of sinuosity, braiding, or anabranching.

CHANNEL PROCESS. Behavior of a channel with respect to channel migration, erosion, and sedimentation.

CHANNEL RELOCATION. Physically moving the location of a stream channel by constructing a new channel and abandoning or filling the existing channel. Channel relocations of jurisdictional streams require Individual Corps of Engineers 404 permits. The new channel may be considered as mitigation for the impact of filling the old channel, but in some cases, additional mitigation is required.

CHANNEL ROUTING. The process whereby a peak flow and/or its associated streamflow hydrograph is mathematically transposed to another site downstream taking into account the effect of channel storage.

CHANNEL SLOPE. Fall per unit length along the channel centerline.

CHANNEL, STABLE. A condition that exists when a channel has a bed slope and cross section that allows it to transport the water and sediment delivered from the upstream watershed without significant aggradation, deposition or bank erosion.

CHANNEL, UNIFORM. Channel with a uniform cross section and constant roughness. A constant slope is also a requirement for uniform flow and depth. For highway drainage design, a channel where the flow streamlines are essentially straight and parallel.

CHANNEL, WANDERING. A channel exhibiting a more or less non-systematic process of channel shifting, erosion, and deposition, with no definite meanders or braided pattern.

CHANNELIZATION. Straightening and/or deepening of a channel by such things as artificial cutoffs, grading, flow-control measures, river training, or diversion of flow into an artificial channel.

CLAY. Material passing the No. 200 (0.074 mm) U.S. Standard Sieve that exhibits plasticity (putty-like properties) within a range of water contents and has considerable strength when air-dry (Unified Soil Classification System) (FHWA Highways in the River Environment Manual).

CLEARANCE. An unobstructed horizontal or vertical space.

COBBLE. A fragment of rock the diameter of which is in the size range of 64 to 250 mm (2.5 to 10 in.) (FHWA Highways in the River Environment Manual).

CODE OF FEDERAL REGULATIONS (CFR). Codifies and publishes at least annually Federal regulations currently in force. The CFR is kept up to date by individual issues of the Federal Register. The two publications must be used together to determine the latest version of any given rule.

COFFERDAM. A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within.

CONTROL. A natural constriction of the channel, a long reach of the channel, a stretch of rapids, or an artificial structure downstream from a gaging station that determines the stage-discharge relation at the gage. A control may be complete or partial. A complete control exists where the stage-discharge relation at a gaging station is entirely independent of fluctuations in stage downstream from the control. A partial control exists where downstream fluctuations have some effect upon the stage-discharge relation at a gaging station. A control, either partial or complete, may also be shifting. Most natural controls are shifting to a degree, but a shifting control exists where the stage-discharge relation experiences frequent changes owing to impermanent bed or banks.

CONTROL SECTION. A control section, such as a bridge opening, reach of channel, or dam, with a definable flow capacity, in which the discharge is related to some measurable depth(s) such as the upstream water surface elevation, tailwater elevation, and/or contracted flow depth.

CONVEYANCE. A measure of the ability of a stream, channel, or conduit to convey water.

CORROSION. The deterioration of pipe or structure by chemical action.

COST-EFFECTIVE. A measure of a drainage design strategy's acceptability is often based on a judgment where either expected first costs or, when appropriate, the economic analysis costs are weighed against the selected design criteria.

The relationship between the benefits derived from a system and the cost of purchasing, operating, and maintaining it.

COUNTERMEASURE. A measure, either incorporated into the design of a drainage facility or installed separately at or near the facility, which serves to prevent, minimize, or control hydraulic problems.

COVER. The vertical extent of soil above the crown of a pipe or culvert. Depending on the context (as in a hydrological method), it may also be ground cover, such as vegetation, or vegetational debris that exists on the soil surface.

CREEK. Term of regional geographic usage. Same as Channel.

CRITICAL DEPTH. The depth at which the specific energy (depth + velocity head) for a particular discharge is a minimum. It is the depth at which, for a given energy content of the water in a channel, maximum discharge occurs or the depth at which, in a given channel, a given quantity of water flows with minimum content of energy.

CRITICAL FLOW. That flow in open channels at which the energy content of the fluid is at a minimum. Also, that flow which has a Froude number of unity. Flow at critical depth.

CRITICAL SHEAR STRESS. The minimum amount of shear stress (tractive force) exerted by passing stream currents required to initiate soil particle motion.

CRITICAL VELOCITY. Mean velocity (V_c) of flow at critical depth (d_c); in open channels the velocity head equals one-half the mean depth.

CROSS SECTION (STREAM OR VALLEY). A diagram or drawing cut across a channel normal to the expected flow direction that illustrates the banks, bed, and water surface. The shape of a channel, stream, or valley, viewed across the axis. In watershed investigations and channel analyses, it is determined by a line approximately perpendicular to the main path of water flow along which measurements of distance and elevation are taken to define the cross-sectional area and shape. In hydraulic analyses, vegetal patterns, floodplain material, and bed material are considered part of the cross section.

CULVERT. A structure that is usually a closed conduit or waterway that is designed hydraulically to take advantage of submergence to increase hydraulic capacity. A structure used to convey surface runoff through such things as a highway or railroad embankment. Although there are borderline cases, a culvert is a structure, as distinguished from bridges, which is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the channel bed serving as the bottom of the culvert. A culvert commonly has a regular, uniform shape, where a bridge opening may not-in other words, a culvert is a relatively large pipe or conduit. A culvert usually has a large ratio of length to width. Usually in practice, and in certain localities by AASHTO's definition, a structure of less than 20 feet span as measured along the road centerline is classified as a culvert. Compare with Bridge.

CULVERT, BROKEN BACK. A culvert having two or more longitudinal structure profile slopes. Such culverts are sometimes effective in reducing outflow velocities by energy dissipation from a hydraulic jump near the outlet.

CULVERT, SAG. A culvert where the inlet and outlet flowline is above the barrel flowline. A culvert that “sags” to pass under a low highway grade line. More commonly used to convey irrigation flows; not suitable for drainage subject to freezing. In the common but incorrect highway vernacular, a “siphon” or “inverted siphon.”

A pipeline crossing a depression or under a highway, railroad, canal, etc., that makes use of pressure flow. A closed conduit, a part of which rises above the hydraulic grade line. It utilizes atmospheric pressure to affect or control the flow of water through it. The term “inverted siphon” or “siphon” is commonly and incorrectly used in highway drainage as such structures have none of the properties of a true siphon; i.e., these two terms are misnomers.

CURRENT. Water flowing through a channel. The generally downstream moving portion or vector of flowing water.

CURRENT METER. An instrument for measuring the speed of flowing water. The U.S. Geological Survey uses a rotating cup meter (33).

CUTOFF. A natural or artificial channel that shortens the length of a stream; natural cutoffs may occur either across the neck of a meander loop (neck cutoffs) or across a point bar (chute cutoffs).

CUTOFF WALL. A wall, collar, or other structure intended to reduce percolation of water along otherwise smooth surfaces, or through porous strata. May also be a wall, usually constructed of such things as sheet piling or concrete, which extends from the end of a drainage structure and/or flowline downward to below the expected scour depth, or to scour-resistant material.

D_{50}. Median size of riprap or granular material. The particle diameter at the 50 percentile point on a size versus weight distribution curve such that half of the particles (by weight) are larger and half are smaller.

DAM. A barrier to confine or raise water for storage or diversion, or to create a hydraulic head.

DAM, DEBRIS. The barrier constructed across a channel to form a debris basin.

DAM, DIVERSION. A barrier constructed for the purpose of diverting part or all the water from a channel into a different course.

DAM-BREAK ANALYSIS. The use of a computer model to calculate the effects of a flood caused by the actual or hypothetical failure of a dam.

DATUM. Plane of reference for elevations.

DEBRIS. Any material transported by the stream, either floating or submerged, such as logs, brush, suspended sediment, bed load, or trash that may lodge against a structure.

DEBRIS CONE. A fan-shaped deposit of soil, sand, gravel, and boulders at the point where a steep stream meets a valley, or where its velocity is reduced sufficient to cause such deposits. Compare with Alluvial Fan.

DEGRADATION (STREAM BED). General and progressive lowering of the longitudinal profile of the channel bed due to long-term erosion. A progressive lowering of the channel bed due to scour. Permanent or continuing degradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place.

DEPOSITION. The settling of material from the streamflow onto the bed.

DEPRESSION STORAGE. Rainfall that is temporarily stored in land surface depressions within a watershed.

DEPTH, ALTERNATE. For a given rate of flow and a given specific head, two depths of flow are possible. These two depths are alternate depths. These depths, one less than and one greater than critical depth, may be present in a channel or conduit for any given value of specific energy above the minimum.

DEPTH, CONJUGATE. The alternate depth of flow involved with the hydraulic jump; i.e., the depth d1 and d2 before and after a hydraulic jump. Unlike the alternate depths for a given specific head, the conjugate depths for a hydraulic jump reflect the energy loss from the hydraulic jump.

DEPTH, NORMAL. The depth of water in an open conduit that corresponds to uniform velocity for the given flow. It is a hypothetical depth under conditions of steady non-uniform flow; the depth for which the water surface and bed are parallel. Normal depth and velocity apply only to uniform flow with a free water surface. These conditions will be approached with a steady discharge in a length of uniform channel that is sufficient to establish uniform flow.

DEPTH OF SCOUR. The vertical distance a stream bed is lowered by scour below a reference elevation.

DEPTH-AREA CURVE. A graph showing the change in average rainfall depth as the drainage area changes.

DESIGN DISCHARGE. The maximum rate of flow (or discharge) for which a drainage facility is designed and thus expected to accommodate without exceeding the adopted design constraints.

DESIGN FLOOD FREQUENCY (OR STORM FREQUENCY). The frequency (recurrence interval) for the selected design discharges (storms) that is expected to be accommodated without contravention of the adopted design criteria. See Design Discharge.

DESIGN STORM. Selected storm of a given frequency (recurrence interval) used for the design of any hydraulic structure, such as a storm sewer system.

DEVELOPMENT. Refers to any constructed change to improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, excavation, or drilling operations or storage of equipment or materials. Taken from the NFIP regulations.

DIKE, RIPARIAN SPUR. River training structure used for bank protection. A permeable or impermeable, linear structure projecting into a channel from a bank. A dike of rock or other material constructed from the bank into the channel for protection or for channel improvement. A dike extending from a bank into a channel that is designed: (1) to reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (2) to deflect erosive currents away from the stream bank (impermeable dike). An embankment or wall constructed more or less perpendicular to a stream bank or shoreline (also termed "groin"). A structure in the form of a barrier placed oblique to the primary motion of water, designed to control movement of bed load. Riparian spur dikes (groins) are usually solid, although they may be constructed with openings to induce sedimentation and control the elevations of such sediments. An elongated, permeable or impermeable obstruction projecting into a stream to control shoaling and scour by deflection of currents and waves. A dike constructed of piles, rock, or other material extending into a stream, the sea, or at the mouths of rivers to induce scouring, bank building, or bank or protection. Compare with Dike, Embankment Spur; and Retard.

DISCHARGE. Volume of water passing a point during a given time. The rate a volume of flow passes a point per unit of time, usually expressed in cubic meters per second (m^3/s) or cubic feet per second (ft^3/s).

DISCHARGE, DOMINANT. The channel forming (morphological sense) discharge in a specific channel for a specific channel feature. The dominant discharge for hydraulic geometry relationships is sometimes taken to be the bankfull discharge. With stable banks, the bankfull discharge has a return period of approximately 1.5 years, whereas the bankfull discharge is sometimes associated with the mean annual flood ($Q_{2.33}$) or two-year flood (Q_2).

DISCHARGE, UNIT. Discharge per unit width (may be average over a cross section, or local at a point).

DITCH. An artificial channel, usually distinguished from a canal by its smaller size.

DRAIN. A conduit for carrying off surplus groundwater or surface waters. Closed drains are usually buried.

DRAINAGE. Four definitions are provided: (1) the process of removing surplus groundwater or surface waters by artificial means; (2) the manner in which the waters of an area are removed; (3) the area from which waters are drained; (4) a drainage basin.

DRAINAGE AREA. The catchment area for rainfall [and other forms of precipitation] that is delineated as the drainage area producing runoff; i.e., contributing drainage area. Usually it is assumed that base flow in a stream also comes from the same drainage area.

DRAINAGE BASIN. A part of the surface of the earth that is occupied by 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. The land area from which surface runoff drains into a stream system.

DRAINAGE DENSITY. Length of all channels above those of a specified stream order per unit of drainage area.

DRAINAGE DIVIDE. The rim of a drainage basin. The divide separating one drainage basin from another and in the past has been generally used to convey this meaning. Drainage divide, or just divide, is used to denote the boundary between one drainage area and another.

DRAW. Term of regional geographical usage. Same as Channel.

DRAWDOWN. The difference in elevation between the water surface elevation at a constriction in a stream or conduit and the elevation that would exist if the constriction were absent. Drawdown also occurs at changes from mild to steep channel slopes and at weirs or vertical spillways.

DRAWDOWN CURVE. A particular form of the surface curve of a stream of water that is convex upward, generally a conversion from tranquil flow upstream to rapid flow downstream in which the stream flow drops through critical depth, such as flow over a weir or spillway or a sharp increase in stream bed slope, proceeding downstream.

DRAWDOWN, RAPID. Lowering the water against a bank more quickly than the bank can drain which can leave the bank in an unstable condition due to excessive pore pressure.

DRIFT. Alternative term sometimes used (perhaps incorrectly) for “debris.” Debris that drifts on or near the water surface. Sometimes used as general term for all debris, that floated or otherwise, in evidence following a flood.

DRIFT LINE. An accumulation of deposited water-carried debris and/or detritus along a contour, at the base of vegetation, or within the vegetation and other topographic objects that provides direct evidence of prior inundation and often indicates the directional flow of floodwaters.

DROP. (1) A vertical or inclined structure for dropping the water in a conduit or channel to a lower level and dissipating its surplus energy. An inclined drop in a channel is often termed a chute; and (2) A fall in water-surface elevation between the upstream and downstream (as between headwater and tailwater) sides of a flow-contacting drainage facility such as at a bridge constriction or culvert or between two sections of a slope-area reach.

DYNAMIC EQUILIBRIUM. The delicate balance of many factors that must occur in a stream reach so that the channel is neither aggrading nor degrading.

EFFECTIVE DURATION. The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rain.

EFFECTIVE PARTICLE SIZE. The diameter of particles, equivalent to a spherical shape, equal in size and arranged in a given manner, of a hypothetical sample of granular material that would have the same transmission constant, shear resistance (see Tractive Force), erosion/scouring response, and fall velocity as the actual material under consideration.

ENCROACHMENT. A highway action within the limits of a base (100-year) floodplain. With a storm sewer system, encroachment is sometimes used when referring to the width of gutter flow spreads onto a traveled way as measured perpendicular from either the edge of the traveled way or from the face of the curb.

ENERGY. The capacity to perform work: kinetic energy is that due to motion, and potential energy is that due to position. In a stream the total energy at any section is represented by the sum of its potential and kinetic energies.

ENERGY DISSIPATION. The phenomenon whereby energy is dissipated or used up. In highway drainage this is the difference in specific energy upstream and downstream of a chosen point due to friction, pipe entrance and exit, bridges, etc.

ENERGY EQUATION. The work-energy relationship, reduced to the simplified form from the Bernoulli equation.

ENERGY GRADE LINE. A line joining the elevation of energy heads of a stream. A line drawn above the hydraulic grade line a distance equivalent to the velocity head of the flowing water at each cross section along a stream or channel reach or through a conduit.

ENERGY GRADIENT. The slope of the energy line with reference to any plane or, more simply, the slope of the energy grade line. The slope of this line represents the rate of loss of head, and it must always slope downward in the direction of flow.

ENERGY, KINETIC. Energy due to motion. The kinetic energy of a given discharge is generally taken as proportional to the product of its weight per unit of time and the velocity head of its mean velocity. For a constant discharge, kinetic energy may be represented by a line at a distance above a flowing water surface proportional to the velocity head of its mean velocity. The elevation of such a line above any datum represents the total energy (potential plus kinetic) of the given discharge above that datum. Strictly, the kinetic energy of a given discharge is the integral of the kinetic energies of its particles.

ENERGY LINE, MINIMUM. An energy line corresponding to conditions of critical flow.

ENERGY, POTENTIAL. Energy due to position. The potential energy of a given volume of immobile water with reference to any datum is proportional to the product of its weight and the elevation of the center of gravity above that datum. The potential energy per unit of time of a given discharge at any instant with reference to any datum is proportional to the product of its weight per unit of time and the elevation of its hydraulic grade line above that datum at that instant.

ENERGY, SPECIFIC. The energy contained in a stream of water expressed in terms of head, referred to the bed of a stream. It is equal to the mean depth of water plus the velocity head of the mean velocity. The energy of a stream referred to its bed; namely, depth plus the velocity head based on the mean velocity.

ENTRANCE HEAD. The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head; equivalent to headwater height; energy head at approach section to culvert or bridge.

ENTRANCE LOSS. The head lost in eddies and friction at the inlet to a conduit or structure, expressed as a coefficient (K_e) times velocity head = K_e (V^2/2g).

ENTRENCHED STREAM. See Stream, Incised.

EROSION. Displacement of soil particles on the land surface due to such things as water or wind action. The wearing away or eroding of material on the land surface or along channel banks by flowing water or wave action on shores.

EROSION CONTROL MATTING. Fibrous matting (e.g., jute, paper) placed or sprayed on a channel bank or land surface for the purpose of preventing erosion or providing temporary stabilization until vegetation is established.

EVAPORATION. The process by which water passes from the liquid to the vapor state.

EVENT. As used in data collection, [event] represents a point at which a gage reaches a preset value and records the occurrence or transmits it to a receiver.

EXCEEDANCE PROBABILITY. Also referred to as frequency or the probability of occurrence. It is the probability that a storm event of a specified duration and volume will be equaled or exceeded in any given year.

EXFILTRATION. The process by which stormwater leaks or flows to the surrounding soil through such things as openings in a conduit, channel banks, or lake shores.

EXTREME VALUES. The largest and the smallest value [from a sample of data] are commonly referred to as extreme or [external] values, and are often associated with floods, droughts, surplus or deficit, and similar economic and safety connotations.

FALLING LIMB. The declining portion of a hydrograph following a crest.

FEDERAL REGISTER. A daily publication of the Federal Government making federal regulations, legal notices, Presidential Proclamations, Executive Orders, etc., known to the public as they are proposed and subsequently issued.

FILL SLOPE. Side or end slope of an earth-fill embankment. Where a fill slope forms the streamward face of a spillthrough abutment, it is regarded as part of the abutment.

FILTER. Layer of synthetic fabric, sand, gravel, and/or graded rock placed (or developed naturally where suitable in-place materials exist) between the bank revetment and soil for one or more of three purposes: (1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion (exfiltrating); (2) to prevent the revetment from sinking into the soil; (3) to permit natural seepage from the stream bank, thus preventing buildup of excessive hydrostatic pressure. Also may be a device or structure for removing solid or colloidal material from stormwater and floodwater or preventing the migration of fine-grained soil particles as water passes through soil; i.e., the water is passed through a filtering medium-usually a granular material or finely woven or non-woven geotextile. Depending on context, may be used to remove material other than soils from a substance.

FILTER BLANKET. One or more layers of graded, intermediate-size gravel or a geotextile material laid between fine-grained material and riprap to prevent the migration of the finer material (exfiltration).

FISH. An aquatic animal (pl. fish or fishes) according to Webster's New Collegiate Dictionary.

FISH, DESIGN. A hypothetical fish embodying predetermined size, swimming, and migration characteristics that are used in the design of drainage facilities to minimize adverse effects to the fishery.

FISH LADDER. A structure with pools and drops to facilitate the migration and movement of fish around culverts, chutes, dams, or other obstructions in channels.

FLANKING. Erosion resulting from streamflow between the bank and the landward end of a river training or a grade-control (drop) structure.

FLOOD. In common usage, an event that overflows the normal flow banks or runoff that has escaped from a channel or other surface waters. In frequency analysis it can also mean an annual flood that may not overflow the normal flow banks. In technical usage, it refers to a given discharge based, typically, on a statistical analysis of an annual series of events.

An overflow or inundation that comes from a river or other body of water and causes or threatens damage. Any relatively high streamflow overtopping the natural or artificial banks in any reach of a channel. A relatively high flow as measured by either gage height or discharge quantity.

FLOOD, ANNUAL. The maximum momentary peak discharge in each year of record. May be maximum daily discharge or instantaneous discharge. The highest peak discharge in a water year. The maximum flow in one year.

FLOOD, BASE. A flood (or storm) or reservoir pool elevation having a one percent chance of being exceeded in a one-year period; commonly termed a 100-year event.

FLOOD ENVELOPE CURVE. A plot showing the upper and lower boundary limits of the maximum annual floods for the range of drainage areas in a hydrologic region.

FLOOD EVENT. A flow of water in a stream constituting a distinct progressive rise, culminating in a crest, together with the recession that follows the crest.

FLOOD EXCEEDANCE PROBABILITY. Probability that a random flood event will exceed a specified magnitude in a given time period, usually one year unless otherwise indicated.

FLOOD FREQUENCY. The average time interval between occurrences of a hydrologic event of a given or greater magnitude, usually expressed in years. May also be called recurrence interval.

FLOOD–FREQUENCY CURVE. A graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are exceeded. A similar graph but with recurrence intervals [frequency] of floods plotted as the abscissa. A graph indicating the probability that the annual flood discharge will exceed a given magnitude, or the recurrence interval corresponding to a given magnitude.

Note that Flood-Frequency is hyphenated when referring to a flood-frequency (flood versus frequency) curve or relationship, and not hyphenated when referring to a specific flood's frequency.

FLOOD, FLASH. A flood that occurs in a short time (minutes to hours) after the storm event.

FLOOD, FIVE-HUNDRED YEAR. The flood due to a storm and/or tide having a 0.2 percent chance of being exceeded in any given year.

FLOOD HAZARD. Potential consequences, hazards, and inconveniences encountered by the traveling public, imposed on adjacent property owners, and incurred by the environment from a flood or a highway action in areas subject to flooding; included are such things as potential property loss or damage, loss of life, temporary or long-term loss of a transportation facility, permanent or long-lasting environmental damage, circuitous or interrupted highway travel, hydroplaning and other roadway overtopping related hazards.

FLOOD, MEAN ANNUAL. Maximum annual flood peak having a 2.33 year frequency interval (recurrence interval). Flood where, if the total population of floods were known, half would be larger and the remaining half would be smaller.

FLOOD OF RECORD. The maximum estimated or measured discharge that has occurred at a site.

FLOOD, ONE-HUNDRED YEAR. Magnitude of the flood that has a 1 percent chance of being exceeded in any given year and has approximately a 63 percent chance of being exceeded during a 100 year period. It is now in vogue to call the 100-year flood the one-percent probability (chance) flood.

FLOOD, OVERTOPPING. Incipient discharge escaping via such things as over a highway, at a watershed divide, or through emergency relief facilities. Stated another way, the flood that, if exceeded, results in flow over a highway, bridge or culvert, over a watershed divide or dike, or through structures provided for emergency relief. The worse case scour condition may occur with the overtopping flood.

FLOOD POOL. Floodwater storage in a reservoir. In a floodwater retarding reservoir, the temporary storage between the crests of the principal and emergency spillways.

FLOOD, PROBABLE MAXIMUM (PMF). The currently accepted term for the most severe flood that is considered reasonably possible at a site as a result of hydrologic and meteorological conditions. This flood would result from the greatest depth of precipitation that is physically possible at a particular site. In practice many assumptions/calculations have to be made about the most severe combinations of meteorological conditions, such as barometric pressure, wind speed, temperature, etc., and such variables as antecedent moisture, average basin infiltration rates, etc. It should be noted that floods greater than the computed PMF have occurred.

The largest flood for which there is any reasonable expectancy in this climatic era. The probable maximum flood is the greatest flood that may reasonably be expected, taking into collective account the most adverse flood related conditions based on geographic location, meteorology, and terrain. A very rare flood discharge value computed by hydrometeorological methods, usually in connection with major hydraulic structures.

A catastrophic flood that, in highway design, may be defined by the upper limits of the flood envelope curve for maximum floods that have occurred in a hydrologic area and physiographic region. With highway design the PMF is sometimes arbitrarily considered to be a 10,000-year event for computational purposes.

FLOOD PROFILE. A graph of elevation of the water surface of a river in flood, plotted as ordinate, against distance, measured in the downstream direction, plotted as abscissa.

FLOOD, PROJECT. A flood discharge value adopted for the design of projects such as dams and flood control works. The term “Design Flood” is more common to highway drainage design.

FLOOD, PROJECT DESIGN. A term common to the design of major dams and flood control works, but not routine highway drainage design.

When used in connection with levees and floodwalls, the Project Design Flood (PDF) is the flood (discharge and elevation) that, when freeboard is added, establishes the top of levee or floodwall grade. (Other, larger floods are used to design for overtopping of these kinds of structures, and to establish overtopping impacts.) For rapid flow channels the PDF is generally defined in much the same way. For reservoirs, the PDF is the flood that, with controlled releases, would fill the designated flood control storage. Tranquil flow channels do not have a single PDF; instead, they have differing design objectives over a range of flood magnitudes. They are generally formulated and designed to continue to provide stage reductions in floods exceeding channel capacity.

FLOOD (OR STORM), REVIEW. A flood (or storm) used to review (check) a drainage facility designed to accommodate a lesser design flood (or storm) so as to judge whether a significant flood hazard due to a flood larger than the proposed design discharge has been overlooked. It can also be described as a flood (or storm), larger or smaller than the selected design flood (or storm), which is used to assess the performance of a drainage facility under other than design conditions.

FLOOD ROUTING. The process of determining progressively the timing and shape of a flood wave at successive points along a river.

FLOOD STAGE. The gage elevation of the lowest bank of the reach in which the gage is situated. The term “lowest bank” is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates unimportant and small areas. The elevation or stage at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured.

FLOOD WAVE. A distinct rise in stage culminating in a crest and followed by recession to lower stages. The rise and fall in streamflow during and after a storm.

FLOOD ZONE. The land bordering a stream that is subject to floods of approximately equal frequency; for example, a strip of the floodplain subject to flooding more often than once but not as frequently as twice in a century.

FLOOD-CONTROL STORAGE. Storage of water in reservoirs to abate flood damage.

FLOODPLAIN. Any plain that borders a stream and is covered by its waters in time of flood. Topographic area adjoining a channel that is covered by flood flows and those areas where the path of the next flood flow is unpredictable, such as a debris cone, alluvial fan or braided channel. A nearly flat, alluvial lowland bordering a stream and commonly formed by stream processes, that is subject to inundation by floods.

FLOODPLAIN DEVELOPMENT. A floodplain containing, zoned to contain or reasonably foreseen to contain development that may incur a significant flood hazard or cause significant conveyance changes in a stream or river reach.

FLOODWAY. A part of the floodplain that, to facilitate the passage of floodwater, is kept clear of encumbrances.

FLORA. The aggregate of plants growing in and usually peculiar to a particular biotic or physiographic region or period.

FLOW DISTRIBUTION. The estimated or measured spatial distribution of the total streamflow from the landward edge of one floodplain or stream bank to the landward edge of the other floodplain or stream bank.

FLOW, GRADUALLY VARIED. Flow in which changes in depth and velocity take place slowly over large distances, resistance to flow dominates, and acceleration forces are neglected.

FLOW, LAMINAR. That type of flow in which each particle moves in a direction parallel to every other particle and in which the head loss is approximately proportional to the first power of the velocity. It is sometimes designated “streamline flow” or “viscous flow.” Laminar flow is characterized by the steady, translatory motion of adjacent small elements of the fluid. Tendencies toward turbulence or instability in truly laminar flow are damped out by viscous shear forces. In terms of the Reynold’s number, laminar flow corresponds to low values of that number.

FLOW LINE. Bottom invert of a conduit or channel.

FLOW, NON-UNIFORM. A flow, the velocity of which is undergoing a positive or negative acceleration. If the flow is constant, it is referred to as “steady non-uniform flow.” A flow in which the velocities vary from point to point along the stream or conduit, due to variations in cross section, slope, etc.

FLOW, ORIFICE. Flow similar to that through an orifice. For highway drainage design, in culvert flow it corresponds to flow-type V; i.e., a culvert flowing part-full under high head.

FLOW, OVERBANK. Water movement over the top of a bank either due to a rising stream stage or to inland surface water runoff.

FLOW, OVERLAND. The flow of rainwater, melting hail, or snowmelt over the land surface toward stream channels. After it enters a stream, it becomes runoff.

The flow of water over a land surface due to direct precipitation. Overland flow generally occurs when the precipitation rate exceeds the infiltration capacity of the soil and depression storage is full. Also called Horton’s Overland Flow.

Runoff that makes its way over the land surface prior to concentrating in gullies and streams; sometimes termed Sheet Flow.

FLOW, PRESSURE. Where flows passing through a bridge type opening are contracted vertically by the superstructure to the extent that the flow has a pressure head and flow streamlines analogous to that occurring at a sluice gate.

FLOW, RAPIDLY VARIED. Flow in which changes in depth and velocity take place over short distances, acceleration forces dominate and energy loss due to friction is minor.

FLOW REGIME. The system or order characteristic of streamflow with respect to velocity, depth, and specific energy.

FLOW, STEADY. A flow in which the flow rate or quantity of fluid passing a given point per unit of time remains constant. A constant discharge with respect to time.

FLOW, SUBCRITICAL. In this state, gravity forces are dominant so that the flow has a relatively low velocity and is often described as tranquil or streaming. Also, that flow that has a Froude number less than unity. Flow at velocities less than critical velocity; flow at depths greater than critical depth.

FLOW, SUPERCRITICAL. In this state, inertia forces are dominant so that flow has a high velocity and is usually described as rapid or shooting. Also, that flow that has a Froude number greater than unity. Flow at velocities greater than critical velocity; flow at depths less than critical depth.

FLOW, TURBULENT. The flow condition in which inertial forces predominate over viscous forces and in which head loss is not linearly related to velocity. That type of flow in which the fluid particles move along very irregular paths. Momentum can be exchanged between one portion of the fluid and another. That type of flow in which any particle may move in any direction with respect to any other particle and in which the head loss is approximately proportional to the second power of the velocity.

FLOW TYPE, BRIDGE. As applied to flow through bridges there are four principal flow types, enumerated as follows: (1) Type I-Subcritical approach flow with subcritical flow in the most contracted section of the bridge opening (backwater due primarily to opening geometry); (2) Type IIA-Subcritical approach flow with flow passing through the normal flood depth (but not critical depth) in the most contracted section of the bridge opening. This is a transition range where the opening geometry is still influential, but the backwater starts to become influenced primarily by the amount of contraction; (3) Type IIB-Subcritical approach flow with flow theoretically passing through the bridge opening at critical depth in the most contracted section of the bridge opening. Backwater is due primarily to the amount of contraction; (4) Type III-Supercritical approach flow with supercritical flow through the bridge opening at a greater depth than the approach or tailwater flow (no backwater possible).

FLOW TYPE, CULVERT. As applied to flow through culverts, there are six principal flow types, enumerated as follows: (1) Type I flow-Part-full flow (low upstream head) with control at inlet; (2) Type II flow-Part-full flow (low upstream head) with control at outlet; (3) Type III flow-Part-full flow (low upstream head) under backwater conditions (tailwater control); (4) Type IV flow-Full-flow with both inlet and outlet of culvert submerged; (5) Type V flow-Part full, sluice-type flow under high upstream head; (6) Type VI flow-Full-flow under high upstream head.

FLOW, UNIFORM. Flow of constant cross section and average velocity through a reach of channel during an interval of time. A constant flow of discharge, the mean velocity of which is also constant. Uniform flow is also referred to as “steady uniform flow.” It is an ideal condition that can, in reality, only be approximated. If the velocity of the constant discharge varies, the flow is defined as “steady non-uniform.” When the average velocities at successive points or sections in the direction of steady flow are the same, the flow is described as uniform. Truly uniform flow, although frequently assumed for computational convenience, seldom occurs in natural open channels. Constant depth flow through a straight reach of a uniform artificial canal is an example of reasonably uniform flow.

FLOW, UNSTEADY. Flow of variable cross section and average velocity through a reach of channel during an interval of time. A changing discharge with respect to time; opposite of Steady Flow; frequently labeled “varied flow.”

FLOW-DURATION CURVE. A cumulative frequency curve that shows the percentage of time that specified discharges are equaled or exceeded.

FLUME. An open or closed channel used to convey water. An open conduit of such things as wood, concrete, or metal on a prepared grade, trestle, or bridge. A flume holds water as a complete structure. A concrete lined canal would still be a canal without the lining, but the lining supported independently would be a flume. A large flume is also termed an aqueduct.

FLUVIAL GEOMORPHOLOGY. A study of the structure and formation of the earth’s features that result from the forces of water. Sometimes river engineers abbreviate fluvial geomorphology in discussions to a simpler, but incorrect, term “morphology.” Compare with Geomorphology. See Morphology and Morphology Problems.

FREEBOARD. Vertical clearance between the lowest structural member of the bridge superstructure, the top culvert invert, or the point of escape for a ditch or a levee system to the water surface elevation of a flood.

FREQUENCY ANALYSIS. The interpretation and analysis of a past record of hydrologic events in terms of future probabilities of occurrence. The procedure involved in estimating the frequencies of occurrence of floods, droughts, storages, rainfalls, water qualities, waves, etc.

FREQUENCY CURVE. A graphical representation of the frequency of occurrence of specific events. In flood studies, frequency is expressed as the recurrence interval (RI) that is the average number of years within which a given peak discharge or rainfall intensity will be exceeded.

FRICTION LOSS (OR HEAD). The head or energy loss as the result of disturbances set up by the contact between a moving stream of water and its containing conduit. For convenience, friction losses are best distinguished from losses due to such things as bends, expansions, obstruction and impacts, but there is no recognized line of demarcation between them, and all such losses are often included in the term “friction loss.”

FRICTION SLOPE. The friction loss (or head) per unit length of conduit. For most conditions of flow, the friction slope coincides with the energy grade line, but where a distinction is made between energy losses due to such things as bends, expansions and impacts, a distinction must also be made between the friction slope and the energy grade line. Friction slope is equal to the bed or surface slope only for relatively uniform flow in nearly uniform channels.

FROUDE NUMBER. A dimensionless number (expressed as $F = V/(gy)^{1/2}$) that represents the ratio of inertial to gravitational forces; i.e., at a Froude number of unity the flow velocity and wave celerity are equal. High Froude numbers can be indicative of a high velocity associated with supercritical flow and thus the potential for scour and high momentum forces. Stated another way, a number that varies in magnitude inversely with the relative influence of gravity on the flow pattern: $F > 1.0$ indicates rapid (supercritical) flow; $F < 1.0$ indicates tranquil (subcritical) flow.

GABION. A rectangular basket made of steel wire fabric or mesh that is filled with rock or similar material of suitable size and gradation. Used to construct such things as flow-control structures, bank protection, groins, jetties, permeable dikes and riparian spur dikes. When filled with cobbles, masonry remnants, or other rock or suitable size and gradation, the gabion becomes a flexible and permeable block with which the foregoing structures and devices can be built.

GAGE (GAUGE). Two definitions are provided: (1) a staff graduated to indicate the elevation of a water surface; (2) a device for registering water levels, flow, velocity and pressure.

GAGE DATUM. The elevation level that corresponds to stage 0.0 at a stream gage; it is often set at the stream bottom or the elevation of a very low flow.

GAGE HEIGHT. Height of the water surface above the zero reference mark on a gage. The water surface elevation is referred to some arbitrary gage datum. Gage height is often used interchangeably with the more general term stage although gage height is more appropriate when used with a reading on a gage.

GAGE, STAFF. A vertical board or structure with a graduated scale for measuring the depth of a river in millimeters or inches. A graduated scale on such things as a staff, plank, metal-plate pier, or wall, by which the elevation of the water surface may be read.

GAGE, STREAM. Instruments that measure the depth of the water in a stream.

GAGING STATION. A particular site on a stream, canal, lake, or reservoir where systematic observations of gage height or discharge are obtained. Used synonymously with Gage.

GEOMORPHOLOGY. A study of the structure and formation of the earth's features. That branch of both physiography and geology that deals with the form of the earth, the general configuration of its surface and the changes that take place due to erosion of the primary elements and in the buildup of erosional debris. Compare with Fluvial Geomorphology. See Morphology, Geomorphology, and Morphology Problems.

GRADE. Three definitions are suggested: (1) the longitudinal slope of a road, channel, or natural ground; (2) the finished surface of a canal bed, road bed, top of embankment, or bottom of excavation; (3) any surface prepared for the support of such things as conduit paving, ties, or rails.

GRADE CONTROL STRUCTURE. Structure placed bank to bank across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or headcutting.

GRADED FILTER. An aggregate filter that is proportioned by particle size to allow water to pass through at a specified rate while preventing the migration of fine-grained soil particles without clogging.

GRADED STREAM. See Stream Poised.

GRADIENT. Change of elevation, velocity, pressure, or other characteristics per unit length; slope.

GRAVEL. Particles, usually of rock, whose diameter is between 2 and 64 mm (0.08 and 2.5 in.). The term gravel is also applied to a mixture of sizes (gravel with sand or gravel with cobbles) in which the dominant or modal fraction is the gravel size range:

GRAVITY DAM. A dam depending solely on its weight to resist water pressure and any momentum forces.

GROIN. See Dike, Riparian Spur.

GROUNDWATER. Subsurface water occupying the saturation zone, from which wells and springs are fed. A source of base flow in streams. In a strict sense the term applies only to water below the water table.

GROUNDWATER DISCHARGE. That part of the discharge from a drainage basin that occurs through the groundwater. The term "underflow" is often used to describe the groundwater outflow that takes place in valley alluvium (instead of the surface channel) and thus is not measured at a gaging station.

GUIDE BANK. Formerly termed spur dike. Relatively short embankments generally in the shape of a quarter of an ellipse and constructed at the upstream side (and sometimes the downstream side) of either or both bridge ends as an extension of the abutment spill slope. The purpose is to align the flow with the bridge opening so as to decrease scour at the bridge abutment by spreading the flow and any resultant scour throughout the bridge opening. May also be a training dike (usually when constructed downstream). Sometimes referred to using the outdated term "Spur Dike."

GUTTER. That portion of the roadway section adjacent to the curb that is utilized to convey stormwater runoff.

HEAD. The height of water above any point, plane, or datum of reference. Used also in various computations, such as energy head, entrance head, friction head, static head, pressure head, lost head, etc. The height of the free surface of a body of water above a given point.

HEAD, ELEVATION. The elevation of a given point in a column of liquid above a datum.

HEAD, ENERGY. The elevation of the hydraulic grade line at any section plus the velocity head of the mean velocity of the flow in that section. The energy head may be referred to any datum or to an inclined plane, such as the bed of a conduit. Also the total head above a datum at any cross section.

HEAD, PIEZOMETRIC. Elevation plus pressure head, total head at any cross section minus the velocity head at that cross section; equivalent to water surface elevation in open channel flow; equivalent to elevation of hydraulic grade line at any point.

HEAD, PRESSURE. Hydrostatic pressure expressed as the height of a column of water that pressure can support at the point of measurement. The head at any point in a conduit represented by the height of the hydraulic grade line above that point.

HEAD, STATIC. The height above a standard datum of the surface of a column of water (or other liquid) that can be supported by the static pressure at a given point. The static head is the sum of the elevation head and the pressure head.

HEAD, TOTAL. The total head of a liquid at a given point is the sum of three components: (1) the elevation head, which is equal to the elevation of the point above a datum; (2) the pressure head, which is the height of a column of static water that can be supported by the static pressure at the point; and (3) the velocity head, which is the height at which the kinetic energy of the liquid is capable of lifting the liquid.

HEAD, VELOCITY. The distance a body must fall freely under the force of gravity to acquire the velocity it possesses; the kinetic energy, in feet of head, possessed by a given velocity. In flowing water, the velocity squared divided by twice gravity ($V^2/2g$).

HEADCUT. The relatively fast drop (as compared to the average channel bed profile slope through a channel reach) in a channel bed profile that is, or has been, headcutting.

HEADCUTTING. Channel degradation associated with abrupt changes in the bed elevation (headcut) that migrates in an upstream direction. Channel bed erosion moving upstream through a basin indicating that a readjustment of the basin's profile slope, channel discharge and sediment load characteristics is taking place. Headcutting may be evidenced by the presence of waterfalls or rapidly moving water through an otherwise placid stream or river, provided there is flow present. In dry channels the presence of a relatively steep drop in the channel bed in an erodible channel is evidence of a headcut. Headcuts may range from 1 foot or less to 10 feet or more. Headcutting often leaves channel banks in an unstable condition as it progresses through a reach as evidenced by large amounts of mass wasting.

HEADLOSS. A loss of energy in a hydraulics system.

HEADWALL. The structural appurtenance usually applied to the end of a culvert inlet and outlet or storm drain outlet to retain an adjacent highway embankment and protect the culvert ends or storm drain outlet and highway embankment or storm drain outfall from bank erosion and channel bed scour.

HEADWATER DEPTH. Depth of water above the inlet flow line at the entrance of a culvert or similar structure. Depth of water upstream of a contraction such as occurs at a bridge or similar structure. Natural flow depth plus backwater caused by a drainage structure.

HEADWATERS. The uppermost reaches for the source of water flowing in a stream. The geographic regions near the divide of a watershed.

HIGHWATER MARK. A mark left as evidence of the height to which a flood reached; usually in the form of such things as deposited sediment, debris and detritus.

HISTORICAL FLOOD. A past flood event of known or estimated magnitude. A known flood event predating systematic flow measurements at a given site.

HYDRAULIC ELEMENTS. The depth, area, perimeter, mean depth, hydraulic radius, velocity, energy and other flow related quantities pertaining to a particular stage of flowing water.

HYDRAULIC FRICTION. A force-resisting flow that is exerted on contact surface between a stream and its containing channel. It usually includes the normal eddies and cross-currents attendant upon turbulent flow occasioned by the roughness characteristic of the boundary surface, moderate curvature and normal channel variations. Wherever possible, the effects of excessive curvature, eddies and impact, obstructions and pronounced channel changes are segregated from the effects of hydraulic friction.

HYDRAULIC GRADELINE. In a closed conduit, a line joining the elevation to which water could stand in risers. In an open conduit, the hydraulic grade line is the water surface; piezometric head line.

A profile of the piezometric level to which the water would rise in piezometer tubes along a pipe run. In open channel flow, it is the water surface.

HYDRAULIC GRADIENT. The slope of the hydraulic grade line; the slope of the water surface in uniform, open channel flow.

The change in total head with a change in distance in a given direction. The direction is that which yields a maximum rate of decrease in head. The slope of the hydraulic grade line through a channel reach or drainage structure.

HYDRAULIC HEAD. See Head.

HYDRAULIC JUMP. The sudden and usually turbulent passage of water from a stage below critical depth (supercritical flow) to a stage above critical depth (subcritical flow) during which the velocity passes from supercritical to subcritical. It represents the limiting conditions of the water surface curve (or profile) wherein it tends to become perpendicular to the stream bed.

A hydraulic phenomenon, in open channel flow, whereby supercritical flow is converted to subcritical flow. This can result in a relatively abrupt and turbulent rise in the water surface.

HYDRAULIC MODEL. A small-scale physical representation of a flow situation.

HYDRAULIC RADIUS. In simplest terms, the cross section area of a stream divided by its wetted perimeter. The cross section area of a stream of water (normal to flow) divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter. A measure of the boundary resistance to flow, computed as the quotient of cross section area of flow divided by the wetted perimeter. For wide shallow flow, the hydraulic radius can be approximated by the average flow depth.

HYDRAULIC ROUGHNESS. A composite of the physical characteristics that influence the flow (or conveyance) of water across the earth's surface, whether natural, channelized, or in a conduit. It affects both the time response of a watershed and drainage channel, or conduit and the channel or conduit storage characteristics.

HYDRAULIC STRUCTURE. A facility used for such things as to impound, accommodate, convey, or control the flow of water, such as a dam, weir, intake, culvert, channel, or bridge.

HYDRAULICS. The applied science concerned with the behavior and flow of liquids, especially in pipes, channels, structures and the ground. In highway drainage, the science addressing the characteristics of fluid mechanics involved with the flow of water in or through drainage facilities.

HYDRAULICS DESIGNER. A hydraulics engineer or in some cases a technician who designs hydraulics structures under the supervision of a more experienced hydraulics engineer.

HYDRAULICS ENGINEER. An engineer whose practice is limited primarily to hydraulics and river mechanics.

HYDROGRAPH. The graph of stage or discharge versus time. A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity or other property of water with respect to time.

HYDROGRAPH, SYNTHETIC. A hydrograph determined from empirical rules. Usually a hydrograph based on the physical characteristics of the basin. A graph developed for an ungaged drainage area, based on known physical characteristics of the watershed basin.

HYDROGRAPH, UNIT. The hydrograph of direct runoff from a storm uniformly distributed over the drainage basin during a specified unit of time; the hydrograph is reduced in vertical scale to correspond to a volume of runoff of 1 inch from the drainage basin.

HYDROLOGIC BUDGET. An accounting of the inflow to, outflow from and storage in, a hydrologic unit, such as a drainage basin, aquifer, soil zone, lake, reservoir, or irrigation project.

HYDROLOGIC CYCLE. A convenient term to denote the circulation of water from the sea, through the atmosphere, to the land, and thence, with many delays, back to the sea by overland and subterranean routes and in part by way of the atmosphere; also, the many short circuits of the water that are returned to the atmosphere without reaching the sea.

HYDROLOGIC SOIL GROUP. A group of soils having the same runoff potential under similar storm and cover conditions.

HYDROLOGIC SOIL-COVER COMPLEX. A combination of a hydrologic soil group and a type of cover.

HYDROLOGIC STUDIES. Studies to determine the runoff and flood characteristics to be expected at a highway drainage site. A most important step prior to the hydraulic design of a highway drainage structure. Such studies are necessary for determining the rate of flow, runoff, or discharge that the drainage facility will be required to accommodate.

HYDROLOGIC UNIT. A geographic area representing part or all of a surface drainage basin or distinct hydrologic feature as delineated by the Office of Water Data Coordination on the State Hydrologic Unit Maps; each hydrologic unit is identified by an eight digit number.

HYDROLOGY. The science and study concerned with the occurrence, circulation, distribution and properties of the waters of the earth and its atmosphere, including precipitation, runoff, and groundwater. The science dealing with the waters of the earth in their various forms: precipitation, evaporation, runoff and groundwater. In highway drainage, the science dealing with the runoff and flood-producing process. In practice the study of the water of the oceans and the atmosphere is, in some cases, considered part of the sciences of oceanography and meteorology.

HYETOGRAPH. Graphical representation of rainfall intensity against time. A graph plotting rainfall amounts or intensities during various time increments versus time.

IMPERVIOUS. Impermeable to the movement or infiltration of water.

INCISED CHANNEL. Those channels that have been cut relatively deep into underlying formations by natural processes. Characteristics include relatively straight alignment and high, steep banks such that overflow rarely occurs, if ever.

INCISED REACH. The stretch of river with an incised channel that only rarely overflows its banks. See Incised Channel.

INFILTRATION. The flow of a fluid into a substance through pores or small openings. It connotes flow into a substance in contradistinction to the word percolation, which connotes flow through a porous substance. The downward entry of water into the soil or rock. That part of rainfall that enters the soil. The passage of water through the soil surface into the ground. Compare with Percolation.

INFILTRATION CAPACITY. The maximum rate at which a soil or rock is capable of absorbing water or limiting infiltration.

INFILTRATION RATE. The rate at which water enters the soil under a given condition. The rate is usually expressed in inches per hour, or feet per day, or feet per second.

INFLOW. The rate of discharge arriving at a point (in a stream, structure, or reservoir).

INFLUENT STREAM. See Stream, Losing.

INITIAL ABSTRACTION. When considering Surface Runoff, the initial abstraction I_a is all the rainfall before runoff begins. When considering direct runoff, I_a consists of interception, evaporation and the soil-water storage that must be exhausted before direct runoff may begin.

INITIAL LOSS. See Initial Abstraction.

INLET. Consider four definitions: (1) a surface connection to a closed drain; (2) a structure at the diversion end of a conduit; (3) the upstream end of any structure through which water may flow; (4) an inlet structure for capturing concentrated surface flow. Inlets may be located in such places as along the roadway, a gutter, the highway median, or a field.

INLET, COMBINATION. Drainage inlet usually composed of two or more inlet types, e.g., such combinations as curb opening and grate inlet, grate, and slotted drain inlet.

INLET CONTROL. A condition where the relation between headwater elevation and discharge is controlled by the upstream end of any structure through which water may flow. For example, a culvert on steep slope and flowing part full is in inlet control.

INLET, CURB OPENING. Drainage inlet consisting of an opening in a curb.

INLET, DROP. Drainage inlet with a horizontal or nearly horizontal opening that is generally flush with the street or land surface.

INLET EFFICIENCY. The ratio of flow intercepted by an inlet to the total flow.

INLET, FLANKING. Inlets placed upstream and on either side of a storm drain inlet that is located at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the longitudinal gutter slope decreases and to act as an emergency relief for the sump inlet at the low point of the vertical curve.

INLET, FLARED. A specially fabricated culvert end appurtenance at the inlet and outlet, or a special end feature of box culverts where the walls flare outward from the culvert sides at the culvert inlet and outlet. This type of inlet is effective in reducing the calculated headwater caused by less efficient inlet types where inlet control prevails. It also serves to retain the roadway embankment. The walls form an angle to the centerline of the culvert. A type of culvert design having an inlet or outlet larger than the main barrel.

INLET GRATE. Typically a steel cover over a drainage structure in the roadway section or at the roadside (in a low point, swale, or ditch) to catch debris and allow only water to enter into the structure.

INLET, IMPROVED. Flared, depressed, or tapered culvert inlets that decrease the amount of energy needed to pass the flow through the inlet and thus increase the capacity of culverts with inlet control or supercritical flow.

INLET, MITERED. A flush-entrance culvert where the barrel is mitered to the slope of the embankment.

INLET, PROJECTING. Culvert barrel projects beyond the plane of the slope or headwall; sometimes referred to as a “re-entrant” entrance.

INLET, SLOTTED DRAIN. Drainage inlet composed of a continuous slot built into the top of a pipe that serves to intercept, collect and transport the flow. Often used in conjunction with a single grate inlet for clean out access.

INLET, TAPERED. A type of culvert design having an entrance face area larger than the main barrel.

INLET TIME. The time required for stormwater to flow from the most distant point in a drainage area to the point at which it enters a storm drain.

INTENSITY. The rate of rainfall upon a watershed, usually expressed in millimeters (or inches) per hour.

INVERT. The flow line in a channel cross section, pipe, or culvert. The lowest point in the channel cross section or at flow control devices such as weirs or dams. The floor, bottom, or lowest part of the internal cross section of a conduit.

ISLAND. A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Some islands originate by establishment of vegetation on a bar and other originate by channel avulsion or at the junction of minor tributaries with a stream.

JURISDICTIONAL SURFACE WATERS. Waters that fall under the control of the US Army Corps of Engineers and require a permit for work within their boundaries. These generally include traditional navigable waters, their tributaries that maintain a baseflow for at least part of the year and wetlands that are hydraulically connected. Jurisdictional Determination shall be made in accordance with US Army Corps of Engineers protocol provided at http://www.usace.army.mil/cw/cecwo/reg/cwa_guide/cwa_guide.htm.

KARST TOPOGRAPHY. Irregular topography characterized by sinkholes, streamless valleys and streams that disappear into the underground, all developed by the action of surface and underground water in soluble rock such as limestone.

LACUSTRINE. Of or pertaining to a lake.

LAG TIME. Variously defined as time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff.

The difference in time between the centroid of the excess rainfall (that rainfall producing runoff) and the peak of the runoff hydrograph. Often estimated as 60 percent of the time of concentration ($TL = 0.6T_c$).

LAKE. An area of open, relatively deep water sufficiently large to produce somewhere on its periphery a barren, wave-swept shore.

LAND USE. A term that relates to both the physical characteristics of the land surface and the human activities associated with the land surface. A land classification. Cover, such as row crops or pasture, indicates a kind of land use.

LAUNCHING. Release of undercut material (stone riprap, rubble, slag, etc.) downslope; if sufficient material accumulates on the stream bank face, the slope can become effectively armored.

LEVEE. An embankment, generally landward of a top bank, that confines flow during highwater periods, thus preventing overflow into lowlands. A linear embankment outside a channel for containment of flow. Longer than a dike.

LOAD (or SEDIMENT LOAD). Amount of sediment being moved by a stream.

LOG PEARSON DISTRIBUTION. Widely accepted statistical method to determine flood frequency using annual maximum stream flow series data. A published document known as Bulletin 17B describes this method, which requires at least 10 years of stream gage data. See PROBABILITY DISTRIBUTION.

LONGITUDINAL PROFILE. The profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

MAIN STEM. Main branch of the watershed (drainage area) stream system.

MANHOLE. Considered to be a gender-neutral term for a structure by which one may access a drainage system. Also referred to as access hole.

MANNING'S n. A coefficient of roughness, used in a Manning's equation for estimating the capacity of a channel to convey water.

MASS CURVE. A graph of the cumulative values of a hydrologic quantity (such as precipitation or runoff), generally as ordinate, plotted against time or date as abscissa.

MASTER DRAINAGE PLAN. Planning model for drainage in a particular regional or local geographic area.

MEAN DAILY DISCHARGE. The average of mean discharge of a stream for one day. Usually given in ft^3/s.

MEAN DEPTH. Cross section area of a channel divided by its surface width.

MEAN DISCHARGE. The arithmetic mean of individual daily mean discharges during a specified period.

MEAN VELOCITY. The velocity at a given section of a stream obtained by dividing the discharge of the stream by the cross section area at that section.

MEAN-SQUARE ERROR. Sum of the squared differences between the true and estimated values of a quantity divided by the number of observations. It can also be defined as the bias squared plus the variance of the quantity.

MEANDER. The winding of a stream channel. The changes in direction and winding of flow, usually in an alluvial channel that is sinuous in character.

MEANDER AMPLITUDE. Distance between points of maximum curvature of successive meanders of opposite phase in a direction normal to the general course of the meander belt, measured between center lines of channels.

MEANDER BELT. Area between lines drawn tangential to the extreme limits of fully developed meanders.

MEANDER BREADTH. The distance between the lines used to define the meander belt.

MEANDER LENGTH. Twice the distance between successive points of inflection of the meander wave. Distance, following the general, sinuous course of the meanders, between corresponding points of successive meanders of the same amplitude.

MEANDER LOOP. An individual loop of a meandering or sinuous channel lying between inflection points with adjoining loops.

MEANDER PLUGS. Deposits of cohesive materials in old channel bendways due to a cutoff. These plugs, sometimes termed “clay plugs,” are sufficiently resistant to erosion to serve as essentially semi-permanent geological controls to advancing channel migrations.

MEANDER SCROLL. Topographical markings on old floodplains resembling a cross section of the edge pattern of a partly unrolled sheet of paper or having a spiral or coiled form, which have been left on a floodplain as a result of the historic migratory movement of the channel. Stated another way, low concentric ridges and swales on a floodplain, marking the successive positions of former meander loops.

MEDIAN DIAMETER. See D_{50}.

MITIGATE. The act of lessening, offsetting, or compensating an impact on surface waters. To moderate (a qualifying or condition) in force or intensity. To decrease or rectify an adverse condition or action.

MITIGATION ALTERNATIVES. Environmental mitigation alternatives for surface waters in order of priority are currently (1992) defined as: (1) avoidance; (2) on-site mitigation; (3) off-site mitigation within the same drainage area; (4) off-site mitigation within the same drainage and biotic region; (5) no mitigation.

MITIGATION MEASURES. Mitigation measures for surface waters are defined as the site-specific action or construction necessary to accomplish the mitigation to the extent practicable.

MITIGATION METHODS. Mitigation methods for surface waters are defined as either the on-site or offsite: (1) construction of new surface waters; (2) enhancement of existing surface waters; (3) acquisition in perpetuity and enhancement of existing surface waters; (4) combinations thereof.

MODEL. Two definitions: (1) a conceptual, mathematical, or physical system obeying certain specified conditions, whose behavior is used to understand the physical system to which it is analogous in some way; (2) a conceptual description and the associated mathematical representation of a system, subsystem, components, or condition that is used to predict changes from a baseline state as a function of internal and/or external stimuli and as a function of time and space.

MODEL, COMPUTER. The representation of a drainage system with computer software.

MODEL, PHYSICAL. The representation of a drainage system with a hydraulically scaled laboratory model.

MORPHOLOGY. The biological study of the form and structure of living organisms. May also be a shortened term used (or misused) when referring to fluvial geomorphology (technically this is incorrect but commonly used).

MORPHOLOGY PROBLEMS. These are fluvial geomorphology problems related to such things as channel aggradation or degradation, bendway migration, bank erosion, bed scour and bendway cutoffs. See Fluvial Geomorphology, Geomorphology, and Morphology.

MUD. A soft, saturated mixture mainly of silt and clay.

MUDFLOW. A well-mixed mass of water and alluvium that, because of its high viscosity and low fluidity as compared with water, moves at a much slower rate, usually piling up and spreading over the [alluvial] fan like a sheet of wet mortar or concrete.

NATIONAL GEODETIC VERTICAL. Datum of 1929. A geodetic datum derived from a general adjustment of the first order level nets of both the United States and Canada. It was formerly called “Sea level Datum of 1929” or “mean sea level” [in the annual WRD data reports]. Although the datum was derived from the average sea level over a period of many years at 26 tide stations along the Atlantic, Gulf of Mexico and Pacific Coasts, it does not necessarily represent local mean sea level at any particular place.

NFIP. Acronym for National Flood Insurance Program.

NORMAL DISTRIBUTION. See Probability Distribution.

NORMAL FLOW. Average flow prevailing during the greater part of the year.

NORMAL STAGE. The average water stage prevailing during the greater part of the year. The water surface elevation corresponding to the Normal Flow.

NORMAL VELOCITY. Mean velocity of flow at normal depth.

OPEN-CHANNEL FLOW. Flow in any open or closed conduit where the water surface is free; that is, where the water surface is at atmospheric pressure.

ORDINARY HIGHWATER (OHW). A term for defining a regulatory-related water surface for a natural channel or the shore of standing waters. This intersection reflects the highest level water reaches in an average runoff year as indicated by such things as erosion, shelving, change in the character of soil, destruction of terrestrial vegetation or its inability to grow, the presence of litter and debris; or in the absence of such evidence, an arbitrarily estimated water surface might be used such as that associated with the mean annual flood. For the purposes of this glossary, in no instance will the ordinary highwater (OHW) be considered as exceeding the estimated water surface level of the mean annual flood unless so mandated by the cognizant regulatory agency. The sum of the water right, flood right, and mean annual flood may be used to arbitrarily determine the maximum OHW for irrigation channels intercepting runoff.

ORGANIC MIXTURES AND MULCHES. Any of a number of agents (e.g., petrochemicals or vegetative matter) used to stabilize a stream bank against erosion by providing protection and nutrients while vegetation becomes established. These agents, which may be in the form of liquids, emulsions or slurries, are normally applied by mechanical means.

OUTFALL. The point where: (1) water flows from a conduit; (2) the mouth (outlet) of a drain or sewer; (3) drainage discharges from a channel or storm drain.

OUTLET CONTROL. A condition where the relation between headwater elevation and discharge is controlled by the conduit, outlet, or downstream conditions of any structure through which water may flow. In culvert flow, outlet control exists for flow-types II, III, IV and VI.

OUTLIER. Outliers (extreme events) are data points that depart from the trend of the rest of data.

PEAK DISCHARGE. The highest value of the stage or discharge attained by a flood; thus, peak stage or peak discharge. Maximum discharge rate on a runoff hydrograph for a given flood event. The instantaneous, maximum discharge of a particular flood at a given point along a stream.

PERMEABILITY. The property of a material that permits appreciable movement of water through it when it is saturated and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water.

PHYSIOGRAPHIC REGION. In highway hydrology considerations, a geographic area whose pattern of landforms and other runoff-producing features for its contiguous subregions are homogeneous but differ significantly from such feature(s) of an adjacent physiographic region(s).

PILE. An elongated member, usually made of such things as timber, concrete, or steel, that serves as a structural component of a river-training structure or bridge foundation.

PIPING. The action of water passing through or under an embankment and carrying some of the finer material with it to the surface at the downstream face. Removal of soil material through subsurface flow or seepage water that develops channels or “pipes” within the soil bank.

PMF. Acronym for Probable Maximum Flood.

PMP. Acronym for Probable Maximum Precipitation.

POND. Very small, very shallow bodies of standing water in which quiescent water and extensive occupancy by higher aquatic plants are common characteristics. Regional usage may refer to a lake as a pond.

POOL. A small, rather deep body of quiescent water, as a pool in a stream.

A deep reach of a stream. The reach of a stream between two riffles. Natural streams often consist of a succession of pools and riffles.

PRACTICABLE. Capable of being accomplished within prudent natural, social, or economic constraints using readily available resources and reasonably reliable technology and practices: available and can be economically applied.

PRECIPITATION. The process by which water in liquid or solid state falls from the atmosphere. The total measurable supply of water received directly from clouds, as rain, snow and hail; usually expressed as depth in a day, month, or year and designated as daily, monthly, or annual precipitation. Not synonymous with Rainfall.

As used in hydrology, precipitation is the discharge of water, in liquid or solid state, out of the atmosphere, generally upon a land or water surface. It is the common process by which atmospheric water becomes surface or subsurface water. The term “precipitation” is also commonly used to designate the quantity of water that is precipitated. Precipitation includes rainfall, snow, hail and sleet and is therefore a more general term than rainfall.

PRECIPITATION, EFFECTIVE. Two definitions (1) that part of the precipitation that produces runoff; (2) a weighted average of current and antecedent precipitation that is “effective” in correlating with runoff.

PRECIPITATION, PROBABLE MAXIMUM. Probable Maximum Precipitation (PMP) is an estimate that approaches the theoretically largest storm physically possible. Development of the PMP considers all storms of record and the observed precipitation is increased by maximizing the moisture inflows to the storm system. Generalized depth, area, duration and season relationships for the continental U.S. are published by the National Weather Service in a series of hydrometeorological reports (HMR).

PRESSURE, HYDROSTATIC. The pressure exerted by the weight of water at any given point in a body of water at rest.

PRESSURE AND MOMENTUM FORCE. The force due to the sum of the pressure and momentum forces caused by moving (flowing) water.

PROBABILITY. The science that deals with the measure of chance or likelihood based on the sampled data.

PROBABILITY DISTRIBUTION. Function describing the relative frequency with which events of various magnitudes occur.

PROBABILITY PAPER. Any graph paper prepared especially for plotting magnitudes of events versus their frequencies or probabilities.

The ordinate and abscissa scales are so designed the distribution plots as a straight line and the data to be fitted appear close to a straight line. The objective of using the probability paper is to linearize the distribution so that the plotted data can be easily analyzed for extrapolation or comparison purposes. In the case of extrapolation, however, the effect of sampling errors is often magnified.

PROFILE. A graphical representation of elevation plotted against distance. In open channel hydraulics a water surface profile is a plot of water surface elevation against channel distance.

PROFILE GRADE. The trace of a vertical plane intersecting any given roadway surface as shown on the plans. Profile grade means either elevation or gradient of such trace according to the context.

PULS METHOD. See Storage-Indication Method.

QUARRY-RUN-STONE. Natural material, often used for stream bank protection, as received from a quarry without regard to gradation requirements.

RACK. A screen composed of parallel bars to catch floating debris.

RAINFALL, EFFECTIVE. Sometimes used as another term for direct runoff. Usually not the same quantity on upland streams as on downstream rivers because of variability of seepage flows.

RAINFALL EXCESS. The volume of rainfall available for direct runoff. It is equal to the total rainfall minus interception, depression storage and absorption.

RAINFALL INTENSITY. Amount of rainfall occurring in a unit of time, converted to its equivalent in millimeters [inches] per hour at the same rate.

RAINFALL, POINT. Rainfall at a single rain gage.

RATING CURVE. A graph of the discharge of a river at a particular point as a function of the elevation of the water surface. A graphic (or tabular) representation of rating; a calibration; a curve (table) relating stage to discharge.

REACH. A segment of stream or valley, selected with arbitrary bounds for purposes of study. A comparatively short length of a stream or channel.

RECEIVING WATER. Body of water that one or more catchments enter into (i.e., stream, tributary, river, estuary, bay, lake, etc.).

RECHARGE. Addition of water to the zone of saturation from precipitation or infiltration.

RECHARGE AREA. An area in which water reaches the zone of saturation by surface infiltration.

RECURRENCE INTERVAL. See Flood Frequency.

REGIME. The condition of a stream and its channel as regards to their stability. A river or canal is “in regime” if its channel has reached a stable form as a result of its flow characteristics.

REGIME CHANGE. A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads or slope.

REGIME CHANNEL. Alluvial channel that has attained more or less a state of equilibrium with respect to erosion and deposition.

REGIME FORMULA. A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.

REGIME OF A STREAM. The system or order characteristic of a stream; in other words, its habits with respect to velocity and volume, form of and changes in channel, capacity to transport sediment and amount of material supplied for transportation. The term is also applied to a stream that has reached an equilibrium between erosion and deposition.

REGIME THEORY. “Regime theory” is a theory of the forming of channels in material carried by the streams. As used in this sense, the word “regime” applies only to streams that make at least part of their boundaries from their transported load and part of their transported load from their boundaries, carrying out the process at different places and times in any one stream in a balanced or alternating manner that prevents unlimited growth or removal of boundaries. A stream, river, or canal of this type is called a “regime stream, river, or canal.” A regime channel is said to be “in regime” when it has achieved average equilibrium; that is, the average values of the quantities that constitute regime do not show a definite trend over a considerable period (generally of the order of a decade).

REGIONAL ANALYSIS. A statistically based regional study of gaged stream data from a homogeneous physiographic region that produces regression equations relating various watershed and climatological parameters to such things as discharge frequency for application on ungaged streams. Used to formulate methods of predicting flood-frequency relationships for the hydraulic design of drainage facilities in hydrologically similar ungaged watersheds having characteristics similar to those used in the regression analysis.

REGIONAL FLOOD OF RECORD. Maximum flood known or recorded in a drainage area.

REGIONAL REGRESSION EQUATION. Emperical equations based on Regional Analysis. Commonly used to estimate steam flow and channel geometry.

REGULATORY FLOOD. The 100-year flood, which was adopted by the FEMA, as the base flood for most floodplain management purposes.

REGULATORY FLOODWAY. The floodplain area that is reserved in an open manner by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount.

RESERVOIR. A pond, lake, or basin, either natural or artificial, for the storage, regulation and control of water. Reservoirs regulate floods downstream from the dam by temporarily storing some part of the flood volume and releasing it later. The impact downstream is to lower flood stages, increase the duration of flooding and shift the flood to a later time.

RESERVOIR, RETARDING. Ungated reservoir for temporary storage of floodwaters. Sometimes called detention reservoir.

RESERVOIR ROUTING. Flood routing through a reservoir. Flood routing of a hydrograph through a reservoir taking into account reservoir storage, spillway, and outlet works discharge relationships.

RESTORE. To re-establish to the extent practicable a geometry, setting, or environment in which the essential elements of the natural surface waters such as floodplains, shores, riparian areas, and channel and any previously existing constructed features can again function as they did prior to a highway action. See Practicable.

RETAINING WALL. With drainage design, a structure used to maintain an elevation differential between the water surface and top bank while at the same time preventing bank erosion and instability.

REVETMENT. Rigid or flexible armor placed on a bank or embankment as protection against scour and lateral erosion.

REYNOLDS NUMBER. The effect of viscosity relative to inertia or R = (VL)/v where V is the velocity of flow, L is a characteristic length, and v (nu) is the kinematic viscosity of the liquid.

RIFFLE. A rapid in a stream. Shallow rapids in an open channel, where the water surface is broken into waves by obstructions wholly or partly submerged. Typically, riffles alternate with pools along the length of a channel.

RIFFLE-POOL RATIO. The sum of the riffle lengths divided by the sum of the pool lengths expressed in percent for a given reach. These lengths are usually measured at a relatively low stage.

RIPRAP. Stones, masonry, or similar constructed material such as broken concrete placed in a loose assemblage along such things as the banks and bed of a channel or the shore of a lake, pond, gulf, bay, or ocean to inhibit erosion and scour.

RIVER. Natural stream of water of considerable volume. Depending on local usage, a larger form of a stream.

RIVER, CONTROLLED. A river and attendant floodplain that is regulated by such constructed devices as dams, flood control measures, navigational locks and diversions.

RIVER MECHANICS. Term for the practices that relate the physical laws governing channels, streams, and rivers to practicable engineering applications.

RIVER, REGULATED. A river and attendant floodplain that is subject to such things as governmental regulations and/or interstate compacts. May also be a river that is controlled by constructed or natural measures.

RIVER TRAINING. The practice of employing structural measures to try and force a stream or river channel to perform in a specified manner.

RIVER TRAINING STRUCTURE. Any configuration of structural measures constructed in a channel or placed on, adjacent to, or in the vicinity of a channel bank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of a stream, or river channel.

ROADBED. The graded portion of a highway within top and side slopes, prepared as a foundation for the pavement structure and shoulder.

ROADWAY CROSS-SLOPE. Transverse slope and/or superelevation described by the roadway section geometry. Usually provided to facilitate drainage and/or resist the centrifugal force of a moving vehicle.

ROADSIDE. A general term denoting the area adjoining the outer edge of the roadway. Extensive area between the roadways of a divided highway (not necessarily the median) may also be considered roadside.

ROUGHNESS COEFFICIENT. The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the “n-value” coefficient used in Manning’s channel flow equation.

RUN. Term of regional geographic use. See Channel.

RUNOFF. That part of the precipitation that runs off the surface of a drainage area after accounting for all abstractions. The portion of precipitation that appears as flow in streams; total volume of flow of a stream during a specified time.

RUNOFF, ANNUAL. The total natural discharge of a stream for a year, usually expressed in inches of depth.

RUNOFF COEFFICIENT. A factor representing that portion of runoff that results from a unit of rainfall. Dependent on terrain and topography. The rate of runoff to precipitation.

RUNOFF, DIRECT. The runoff entering stream channels promptly after rainfall or snowmelt. Superimposed on base runoff, it forms the bulk of the hydrograph of a flood.

RUNOFF, "FIRST FLUSH." The condition, often occurring, in which a disproportionately high pollution load is carried in the first portion of urban runoff.

RUNOFF, SUBSURFACE. Water that infiltrates the soil and reappears as seepage or spring flow and forms part of the flood hydrograph for that storm. Difficult to determine in practice and seldom worked with separately.

RUNOFF, SURFACE. That part of the runoff that travels over the soil surface to the nearest stream channel. It is also defined as that part of the runoff of a drainage basin that has not passed beneath the surface since precipitation.

RUNOFF, URBAN. Storm-generated surface runoff from an urban drainage area. The term may relate to either the quantity or quality of the runoff or both, depending upon its application.

SAND. Soil material that can pass the No. 4 (4.76 mm) U.S. Standard Sieve and be retained on the No. 200 (0.074 mm) sieve. FHWA, HIRE, 1987. Granular material that is smaller than 2.0 mm (0.08 in.) and coarser than 0.062 mm (0.0024 in.). FHWA, HIRE, 1990.

SCOUR. The displacement and removal of channel bed material due to flowing water; usually considered as being localized as opposed to general bed degradation or headcutting. The result of the erosive action of running water that excavates and carries away material from a channel bed.

SCOUR, BENDWAY. That component of natural scour consisting of the removal of material from the channel bed or banks that occurs along the concave (outside) bank in a channel bendway located, generally, across the channel from any point bar.

SCOUR, CONTRACTION. Scour (see above) resulting from a contraction of the flow area at a bridge which causes an increase in velocity and shear stress on the streambed at the bridge.

SCOUR, GENERAL. Scour in a channel or on a floodplain that is not localized at a pier, abutment, bendway, or other obstruction to flow. In a channel, general scour usually affects all or most of the channel width.

SCOUR, LOCAL. Removal of material from the channel bed or banks that is restricted to a relatively minor part of the width of a channel. Scour in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow. Local scour is caused by the acceleration of the flow and the development of a vortex system induced by the obstruction to the flow. Does not include the additional scour caused by any contraction, natural channel degradation, or bendway.

SCUPPER. A device and/or vertical hole through such things as bridge decks or roofs for the purpose of deck or roof drainage. Sometimes a horizontal opening in the curb or barrier is called a scupper.

SEDIMENT. Fragmental material that originates from weathering of rocks and is transported by, suspended in, or deposited by water or air or is accumulated in beds by other natural agencies.

SEDIMENT CONCENTRATION. Weight or volume of sediment relative to quantity of transporting or suspending fluid or fluid-sediment mixture.

SEDIMENT DISCHARGE, SUSPENDED. The rate at which the dry weight of sediment passes a section of a stream or is the quantity of sediment (as measured by dry weight, or by volume) that is discharged in a given time. The quantity of sediment that is carried past any cross section of a stream or river above the bed layer in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.

SEDIMENT DISCHARGE, TOTAL. The sum of suspended sediment discharge and bed load discharge or the sum of bed material discharge and wash load discharge of a stream or river.

SEDIMENT LOAD, SUSPENDED. Sediment that is supported by the upward components of turbulent currents in a stream and that stays in suspension for an appreciable length of time.

SEDIMENT POOL. That portion of the total reservoir storage provided for sediment, thus prolonging the usefulness of floodwater or irrigation pools.

SEDIMENT YIELD. The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.

SEDIMENTATION. The process involving the deposition of soil particles that have been carried by floodwaters. Sometimes erroneously termed “silting” by those unfamiliar with sediment transport and deposition as the deposited material does not necessarily contain much, if any material classified as silt.

SEEPAGE. The slow movement of water through small cracks and pores of the bank material.

SEMI-ARID. Geographic areas characterized by light rainfall. More specifically, having from approximately 10 inches to approximately 20 inches of annual precipitation.

SEWER. A conduit for conveying sanitary waste flows.

SEWER, COMBINED. A sewer that conveys stormwater and, at times, sanitary sewage.

SEWER, STORM. Principally a drain for conveying stormwater, but at least part of the time, a drain that also conveys raw sewage is termed a storm sewer.

SHALLOW CONCENTRATED FLOW. In TR-55 and other methods, this term is used to describe the stage of runoff that has progressed beyond overland or sheet flow, but has not yet reached a clearly defined channel.

SHEET FLOW. See Flow, Overland.

SHOULDER. The portion of the roadway contiguous with the traveled way for accommodating stopped vehicles, for emergency use, and for lateral support of the road's base and surface courses.

SIDE SLOPES. The slope of the sides of a canal, dam, or embankment typically expressed as a ratio of horizontal distance to vertical distance (for example, 2:1)..

SILT. Material passing the No. 200 (0.074 mm) U.S. Standard Sieve that is nonplastic or very slightly plastic and exhibits little or no strength when air-dried (Unified Soil Classification System) according to the FHWA's "Highways in the River Environment Manual," January 1987. According to the FHWA's "Highways in the River Environment Manual" (February 1990), material finer than 0.062 mm (0.0025 in.) and coarser than 0.004 mm (0.00015 in.)

SKEW. The measure of the angle of intersection between a line normal to the roadway centerline and the direction of the flow in a channel at flood stage in the lineal direction of the main channel.

SLOPE, ADVERSE. A conduit is on an adverse slope when its slope is positive (slopes upward) in the downstream direction.

SLOPE, MILD (STEEP). A conduit is on mild slope when its slope is less than the critical slope for a particular value of discharge. Whether a conduit slope is steep or mild depends on the discharge under consideration and the conduit slope; the conduit may be steep for one discharge and mild for another. A conduit is on mild slope if, for a given discharge, the normal depth is greater than critical depth.

SLOPE-AREA METHOD. A method of estimating unmeasured flood discharges in a uniform channel reach using observed highwater levels. More explicitly a flow measurement method based on an indirect measurement of peak discharge by field survey of a reach of channel and highwater marks, usually after a flood has passed. Discharge is computed by the Manning equation, modified to account for non-uniform flow.

SLUMPING. When the stream bank moves a short distance downslope.

SNOW. A form or precipitation composed of ice crystals.

SOIL. Finely divided material composed of disintegrated rock mixed with organic matter; the loose surface material in which plants grow. Compare with Dirt. Dirt, an incorrect term, is not soil as it contains humus and other foreign material.

SOIL GROUP. See Hydrologic Soil Group.

SOIL MOISTURE. Water diffused in the soil; the upper part of the zone of aeration from which water is discharged by the transpiration of plants or by soil evaporation. The water contained in the unsaturated zone.

SPAN. Terminology used with culverts (and similar type openings) as the horizontal width dimension of such things as a box, pipe-arch, or arch structure, as in Span X Rise. May also be the horizontal distance between bridge piers or abutments.

SPILLWAY. A passage for spilling surplus water.

SPILLWAY, CONTROLLED. A reservoir outlet works wherein the outflow is controlled by gates, valves, or similar flow control devices.

SPILLWAY, EMERGENCY. A rock or vegetated earth waterway around a dam, built with its crest above the normally used principal spillway. Used to assist or supplement the principal spillway in conveying extreme amounts of runoff safely past the dam so as to minimize damage and flood hazards.

SPILLWAY, PRINCIPAL. Conveys all ordinary discharges coming into a reservoir and that portion of an extreme discharge that does not pass through the emergency spillway or outlet works.

A concrete or metal pipe or conduit used with a drop inlet dam or floodwater retarding structure. It conveys, in a safe and non-erosive manner, all ordinary discharges coming into a reservoir and all of an extreme amount that does not pass through the emergency spillway.

SPILLWAY, UNCONTROLLED. A spillway for a reservoir at which floodwater discharge is governed only by the inflow and resulting head in the reservoir. Usually the emergency spillway is uncontrolled.

SPREAD. The accumulated flow in and next to the roadway gutter. The transverse encroachment of stormwater onto a street. This water often represents an interruption to traffic flow, splash-related problems and a source of hydroplaning during rainstorms. The lateral distance, in meters (or feet), of roadway ponding extending out from the curb or edge of the traveled way.

SPREAD FOOTING. A pier or abutment footing that transfers load directly to the earth.

SPRING. A discrete place where groundwater flows naturally from a rock or the soil onto the land surface or into a body of surface waters.

STAGE. Height of water surface above a specified datum. Water surface elevation of a channel with respect to a reference elevation.

STAGE-CAPACITY CURVE. A graph showing the relation between the surface elevation of the water in a reservoir, usually plotted as ordinate, against: (1) the volume below that elevation, plotted as abscissa; or (2) amount of water flowing in a channel, expressed as volume per unit of time, plotted as abscissa.

STAGE-DAMAGE GRAPHS. Charts or graphs that show the relation between flood depth or elevation and damages in the area.

STAGE-DISCHARGE CURVE. The relation between stage and the discharge of a stream or river at a particular site, usually presented in the form of a graph or table. Sometimes referred to as the rating curve of a channel cross section.

STANDARD ERROR. An estimate of the standard deviation of a statistic. Often calculated from a single set of observations. Calculated like the standard deviation but differing from it in meaning.

STILLING BASIN. A device or structure placed at or near the outlet of a structure for the purpose of inducing energy dissipation where flow velocities are expected to cause unacceptable channel bed scour and bank erosion.

STILLING WELL. A pipe, chamber, or some other type of compartment with closed sides and bottom except for a comparatively small inlet or inlets communicating with a main body of water. Its purpose is to dampen waves or surges while permitting the water level within the well to rise and fall with the major fluctuations of the main body.

STORAGE. Two definitions: (1) water artificially impounded in surface or underground reservoirs, for future use (the term regulation refers to the action of this storage in modifying streamflow; (2) water naturally detained in a drainage basin, such as groundwater, channel storage and depression storage where the term “drainage basin storage” or simply “basin storage” is sometimes used to refer collectively to the amount of water in natural storage in a drainage basin.

STORAGE, BANK. The water absorbed into the banks of a stream channel when the stages rise above the water table in the bank formations; then the water returns to the channel as effluent seepage when the stages fall below the water table.

STORAGE, BASIN. Term sometimes used to refer collectively to the amount of water in natural storage in a drainage basin. See Storage. Compare with Surface Storage.

STORAGE, CHANNEL. The volume of water at a given time in the channel or over the floodplain of the streams in a drainage basin or river reach. Channel storage is [occurs] during the progress of a flood event and thus is transient in nature.

STORAGE-INDICATION METHOD. Name often given to a flood-routing method. Also often called the Puls method (after Lows G. Puls), though it is actually a variation of the method devised by Puls.

STORM. A disturbance of the ordinary average conditions of the atmosphere that, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder.

STORM DRAIN. A conduit that carries runoff, typically from developed sites such as roads, parking lots or buildings.

STORM SURGE. Oceanic tide-like phenomenon resulting from wind and barometric pressure changes.

STORMWATER. Storm-generated surface runoff from a developed drainage area.

STREAM. A general term for a body of flowing water. In hydrology the term is generally applied to the water flowing in a natural channel as distinct from a canal.

Streams in natural channels may be classified as follows:

Intermittent. One that ceases flow during dry periods, or

Perennial. One that flows continuously, or

Ephemeral. One that flows only in direct response to precipitation and whose channel is at all times above the water table;

Relation of a stream (or river) to space:

Continuous. One that does not have interruptions in space, or

Interrupted. One that contains alternating reaches, that are either perennial, intermittent, or ephemeral; and

Gaining. A reach that receives water from the zone of saturation,

Losing. A reach that contributes water to the zone of saturation,

Insulated. A reach that neither contributes water to the zone of saturation nor receives water from it; it is separated from the zones of saturation by an impermeable bed, or

Perched. A reach that is either losing or insulated and is separated from the underlying groundwater by a zone of aeration.

STREAM BANK EROSION. Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions and land use changes may also directly or indirectly lead to stream bank erosion.

STREAM BANK FAILURE. Collapse or slippage of a large mass of bank material into the channel. Sometimes called Mass Wasting.

STREAM BANK PROTECTION. Any technique used to prevent erosion or failure of a channel bank.

STREAM FLOW DEPLETION. The amount of water that flows into a valley or onto a particular land area, minus the water that flows out the valley or off from the particular land area.

STREAM GAGING. The process and art of measuring the depths, areas, velocities and rates of flow in natural or artificial channels.

STREAM-GAGING STATION. A gaging station where a record of discharge of a stream is obtained. Generally, this term is used only for those gaging stations where a continuous record of discharge is obtained:

STREAM, INCISED. A stream or river that flows in an incised channel with high banks; say, banks that stand above the 50- to 100-year water surface stage are arbitrarily regarded as high. Sometimes termed an entrenched stream or river. See Incised Channel and Incised Reach.

STREAM, INSULATED. A stream or reach of a stream that neither contributes water to the zone of saturation nor receives water from it. It is separated from the zones of saturation by an impermeable bed.

STREAM, LOSING. A stream or reach of a stream that contributes water to the zone of saturation. A stream or reach of a stream in which water flows from the channel bed into the ground. A stream or reach of a stream that is losing water by seepage into the ground; also known as an Influent Stream.

STREAM ORDER. A method of numbering (ordering) streams as part of a drainage basin network. The smallest, unbranched, mapped tributary is called first order, the stream receiving the tributary is called second order and so on.

STREAM, POISED. A term used by river engineers as applying to a stream that over a period of time is neither degrading nor aggrading its channel. A stream nearly in equilibrium as to sediment transport and supply.

STREAM POWER. An expression used in predicting bed forms and hence bed load transport in alluvial channels; a parameter comprised of the mean velocity, the specific weight of the water-sediment mixture, the normal depth of flow and the channel slope. A parameter that reflects the ability of a stream to distort its bed to produce such things as bed forms, scour, or deposition.

STREAM REACH. A length of stream channel selected for use in hydraulic or other computations.

STREAM RESPONSE. Changes in the dynamic equilibrium of a stream by any one, or combination of various causes.

STREAMFLOW. The discharge that occurs in a natural channel. Although the term discharge can be applied to the flow of a canal, the word streamflow uniquely describes the discharge in a surface stream course.

STREAMFLOW RECORD. A tabulation of the flow of a stream. Streamflow records are published annually by the United States Geological Survey in their Water-Supply Papers. Such things as daily, monthly, annual and instantaneous extremes of discharge are shown therein, along with information about the stream gage.

STRUCTURES. Such things as bridges, culverts, catch basins, drop inlets, retaining walls, cribbing, access holes, endwalls, buildings, storm drains, service pipes, under drains, foundation drains and other appurtenant features.

STRUCTURAL MEASURES. Methods of reducing damage from floods such as dams and reservoirs, levees, dikes, floodwalls, diversion channels, bridge modifications, channel alterations, pumping stations and land treatment.

SUB-BASE. The layer or layers of specified or selected material of designed thickness placed on a subgrade to support a base course.

SUBGRADE. The top surface of a roadbed upon which the pavement structure and shoulders are constructed. Compare with Sub-Bed Material and Sub-Base.

SUB-BED MATERIAL. Material underlying that portion of the channel bed that is subject to the direct action of the flow. Compare with Subgrade and Sub-Base.

SUB-SURFACE FLOAT. A submerged body that is attached by a line to and the movement of which is indicated by, a surface float; used for the purpose of observing velocities or the direction of flow.

SUBMEANDER. Small meander contained within the low flow banks of a main channel, associated with relatively low discharges and rills.

SUBMERGED INLET. Inlets of culvert like structures having a headwater greater than approximately 1.2 D, where D is the culvert rise.

SUBMERGED ORIFICE. An orifice that in use is drowned by having the tailwater higher than all parts of the opening.

SUBMERGED OUTLET. Submerged outlets are those culvert-like outlets having a tailwater elevation greater than the soffit of the culvert.

SUBMERGENCE. In culvert terminology, the condition where tailwater or headwater elevation are greater than elevation of the conduit top (soffit).

SUPERELEVATION. The increase in water surface elevation at the outside of open channel bendways. May also be a transverse tilting of the channel bed (in lined channels with predominately supercritical flow) or the increase in the elevation at the outside edge of a road or traveled way located in a horizontal curve.

SUPERSTRUCTURE. The portion of a structure above the substructure.

SURFACE WATERS. Water on the surface of the earth. Any stream, river, lake, pond, or reservoir. Some include wetlands in surface waters.

SURFACE WATER ENHANCEMENT. Improving existing or new surface water functions and values with practicable measures.

SURFACE WATER MITIGATION. The on-site and/or off-site construction of new surface waters, enhancement of existing surface waters, acquisition and enhancement of existing surface waters, or combinations thereof.

SURFACE WATER QUALITY. The findings from an evaluation of the importance and degree of excellence of surface water functions, values and features.

SURFACE WATER VALUE. The various essential and nonessential aesthetics, products and services of sometimes definable value that surface waters provide to society, including such things as fish and wildlife habitat, water supply, improvement of water quality, flood control, bank erosion and shoreline protection, outdoor recreation opportunities, education and research and beauty.

The value, economic or environmental, of a surface water function.

SURFACE WATERS. Water on the surface of the earth (62a). Any stream, river, lake, pond, or reservoir. Some include wetlands in surface waters.

For regulatory purposes, navigable waters of the U.S. as currently defined by the USACE. See Navigable Waters.

A more legally based description might be, depending on the context, water appearing on the land surface in a diffused state for a considerable time, with no permanent source of supply or regular course; as distinguished from water appearing in watercourses, lakes, or ponds. Sometimes considered as overland flow or surface flow. See Overland Flow and Surface Flow. Compare with Streamflow, Discharge, and Runoff.

SURFACE WATERS, RARE. Surface waters or wetlands having features, functions, values, or quality that are uncommon, unique, or seldom occur in the ecoregion. Compare with Surface Waters, Sensitive.

SURFACE WATERS, SENSITIVE. Those surface waters or wetlands that, by their nature and setting, are inherently important, unique, or rare due to such things as their environment, public use and flood control function. Waters that, without mitigation measures, would be threatened by a highway action. Compare with Surface Waters, Rare.

SURFACE WATERS THREAT. The likelihood that surface waters or wetlands, or a portion thereof, will be destroyed, degraded, or otherwise adversely impacted, directly or indirectly, through a highway action.

SWALE. A wide, shallow ditch usually grassed or paved and without well-defined bed and banks. A slight depression in the ground surface where water collects and that may be transported as a stream. Often vegetated and shaped so as not to provide a visual signature of a bank or shore.

SYNTHETIC FILTER. Fabric of synthetic material that serves the same purpose as a granular filter blanket.

TAILWATER. Tailwater (TW) is the depth of flow in the channel directly downstream of a drainage facility. Often calculated for the discharge flowing in the natural stream without the highway effect (but may include other local effects from development), unless there is a significant amount of temporary storage that will be (or is) caused by the highway facility; in which case, a flood routing analysis may be required. The tailwater is usually used in such things as culvert and storm drain design and is the depth measured from the downstream flow line of the culvert or storm drain to the water surface.

THALWEG. The line or path connecting the lowest flow points along the bed of a channel. The line does not include local depressions. The path very low flows would follow in proceeding down a stream, river, swale, or channel. The line extending along a channel profile that follows the lowest elevation of the bed.

TIMBER OR BRUSH MATTRESS. Such things as a revetment, blanket, or armor made of such things as brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream or river and weighted with ballast.

TIME OF CONCENTRATION. The estimated time required for runoff to flow from the most remote section of the drainage area (hydraulically most distant point) to the point at which the discharge is to be determined. Not synonymous with Travel Time.

TOE. That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.

TOPSOIL. Surface soil that is suitable for the germination of seeds and the support of vegetative growth.

TRACTIVE FORCE. The drag on a stream bank caused by passing water that tends to pull soil particles along with the streamflow. The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually expressed in units of stress; i.e., force per unit area. The force per unit area on a stationary boundary exerted by a fluid flowing past that boundary. Compare with Critical Shear Stress.

TRANQUIL FLOW. See Flow, Subcritical.

TRASH RACK. A device used to capture debris, either floating, suspended, or rolling and saltating along the bed, before it enters a drainage facility. A grid or screen across a stream or entrance to a drainage facility designed to catch debris.

TRAVEL TIME. The average time for water to flow through a reach or other stream or valley length that is less than the total (stream or valley) length. A travel time is part of a T_c (Time of Concentration) but never the whole T_c. Not synonymous with Time of Concentration.

TRAVELED WAY. That portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes (such as turning lanes and parking lanes).

TURBULENCE. Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate streamlines. A state of flow wherein the water is agitated by cross-currents and eddies; opposed to a condition of flow that is quiet or quiescent.

UNDERDRAIN. Drainage system located under pavement to drain the pavement structure and prevent weakening and degradation of the pavement structure. See WVDOH Standard Details Volume I, DR-8, Sheet 3 of 4 & 4 of 4.

UNGAGED STREAM SITES. Locations at which no systematic records are available regarding actual stream flows or water quality information.

UPLIFT. The upward water pressure force on the base of a structure.

VELOCITY. The rate of motion of a stream or river or of the objects or particles transported therein, usually expressed in distance per time.

VELOCITY, AVERAGE. Velocity at a given cross section determined by dividing the total discharge at that point by the total cross section area.

VELOCITY OF APPROACH. The mean velocity in the conduit or channel immediately upstream from a weir, dam, Venturi throat orifice, or other structure.

VELOCITY, PERMISSIBLE. The highest velocity at which water may be carried safely in a canal or other conduit without channel bed scour or bank erosion.

VERTICAL ABUTMENT. An abutment, usually with wingwalls, that has no fill slope on its channel side.

WASH LOAD. That part of the total sediment discharge that is composed of particle sizes finer than those found in appreciable quantities in the bed material. Large quantities of fine materials that could be carried easily by stream flow. That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally the fine sediment load is finer than 0.062 mm (0.0024 in.) for a sand bed channel. Silts, clays, and sand could be considered as wash load in coarse gravel and cobble bed channels. Wash load sediments commonly originate on uplands and are carried into a channel by overland flows. Compare with Bed Load; Sediment Load, Suspended; Sediment Discharge, Total; and Sediment Discharge, Suspended.

WATER BALANCE. See Hydrologic Budget.

WATER BUDGET. An evaluation of all the sources of supply and the corresponding discharges with respect to an aquifer or a drainage basin.

WATER YEAR. In the Federal agency reports dealing with surface-water supply, the 12-month period, October 1 through September 30. The water year is designated by the calendar year in which it ends and that includes 9 of the 12 months. Thus, the year ended September 30, 1959, is called the “1959 water year”.

WATER SURFACE PROFILE. A graph of water levels plotted against stream distance at a particular time or for a particular condition, such as for a flood peak or for a low-flow period. Water-surface curves or profiles are generally catalogued into twelve classifications, three of which are designated strictly as backwater curves. The classifications are accounted for by the different bottom slopes and relative values of normal and critical depth. The curves are classified by the nomenclature; M1, M2, and M3 for mild slope (backwater curves); C1 and C3 for critical slope; H2 and H3 for horizontal (zero slope); S1, S2, and S3 for steep slope; and A2 and A3 for adverse slope.

WATERCOURSE. A stream, river, or channel in which a flow of water occurs, either continuously or intermittently, with some degree of regularity.

WATERSHED. See Drainage Area.

WATERWAY. Any stream, river, lake, pond, or ocean that can be traversed for purposes of commerce or recreation. May also refer to a channel.

WEIGHTED MEAN. Statistical value obtained by multiplying each of a series of values by its assigned weight and dividing the sum of those products by the sum of the weights.

WEIR. A dam across a channel for diverting flows or for measuring the flow.

WEIR, BROAD-CRESTED. An overflow structure on which the nappe (the profile of a body of water flowing over an obstruction in a vertical drop) is supported for an appreciable length; a weir with a significant dimension in the direction of the stream. Highways generally function as broad-crested weirs when overtopped by floodwaters.

WEIR FLOW. Free surface flow over a control surface that has a defined discharge versus depth relationship.

WEIR, SHARP-CRESTED. A contracted measuring weir with its crest at the upstream edge or corner of a relatively thin plate, generally of metal.

WEIR, SUBMERGED. A weir that in use has the tailwater level equal to, or higher than the weir crest.

WETLANDS. Those lands having: wetland hydrology; hydric soils; and hydrophyte type vegetation as delineated by current editions of the Federal Manual for Identifying and Delineating Jurisdictional Wetlands. These include wetlands subject to Federal law regardless of whether they involve Federal, State, or private lands.

WETLANDS, JURISDICTIONAL. See Jurisdictional Surface Waters.

WETTED PERIMETER. The boundary over which water flows in a channel, stream, river, swale, or drainage facility such as a culvert or storm drain. The boundary is taken normal to the flow direction of the discharge in question. The length of the wetted contact between a stream of water and its containing conduit, measured along a plane at right angles to the flow in question; that part of the periphery of the cross section area of a stream in contact with its container.