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

Documentation Page

Title: WEST VIRGINIA DIVISION OF HIGHWAYS DRAINAGE MANUAL 3rd Edition December 2007	
Sponsoring Agency: WVDOH, Engineering Division, Hydraulic & Drainage Unit 1900 Kanawha Boulevard, East Building 5, Room A-430 Charleston, WV 25305-0430	Consultant: Michael Baker Jr., Inc. 5088 West Washington Street Charleston, WV 25313
Principal Authors:	
West Virginia Division of Highways Darrin Holmes, PE	Michael Baker Jr. Inc. Ramesh Chintala, PE, CFM, D.WRE
Technical Assistance provided by:	
West Virginia Division of Highways Jason Ritter, EI Kyle Stollings, PE James Sloan, PE Jackie Fields, PE Walt Simmons, EI	Michael Baker Jr. Inc 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 Feburary 2012.
Content included within the addendum is marked within the footer.

Order printed copies or CD's from:
WVDOH, Engineering Division,
Initial Design/Technical Standards 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

CONTACT INFORMATION

Please send comments regarding this edition or suggestions for a future addendum to the following e-mail address:

DOT.DrainageManual@wv.gov

Questions about the content or use of this edition can be addressed to the Hydraulics and Drainage Unit directly.

We are always available for assistance.

Doug Kirk, Unit leader – Douglas.W.Kirk@wv.gov
Darrin Holmes – Darrin.A.Holmes@wv.gov
Andrew Branard – Andrew.R.Branard@wv.gov

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 1:
INTRODUCTION

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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			
	Recommended	Value Range	
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		
Broken soil, Clean, No Residual Vegetation	0.050		
Broken Soil, Vegetation Cover ≤ 20%	0.060		
Broken Soil, Vegetation Cover > 20%	0.170		
Open Field, Undisturbed Bare Soil, No Vegetation	0.025	0.020 - 0.030	
Open Field, Short Grass, No Brush	0.030	0.025 - 0.035	
Open Field, High Grass, No Brush	0.035	0.030 - 0.050	
Scattered Brush Covered Area, Heavy Weeds	0.050	0.035 - 0.070	
Light Brush Covered Area, With Small trees	0.055	0.037 - 0.070	
Medium to Dense Brush Covered Area	0.085	0.057 - 0.135	
Forest Area, Cleared Land With Stumps	0.050	0.040 - 0.065	
Forest, Heavy Stand, Few Downed Trees	0.100	0.080 - 0.120	
Natural Undisturbed Range Land Thick Mix Of Grasses	0.130		
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

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.

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CHAPTER 4:
HYDROLOGY

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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

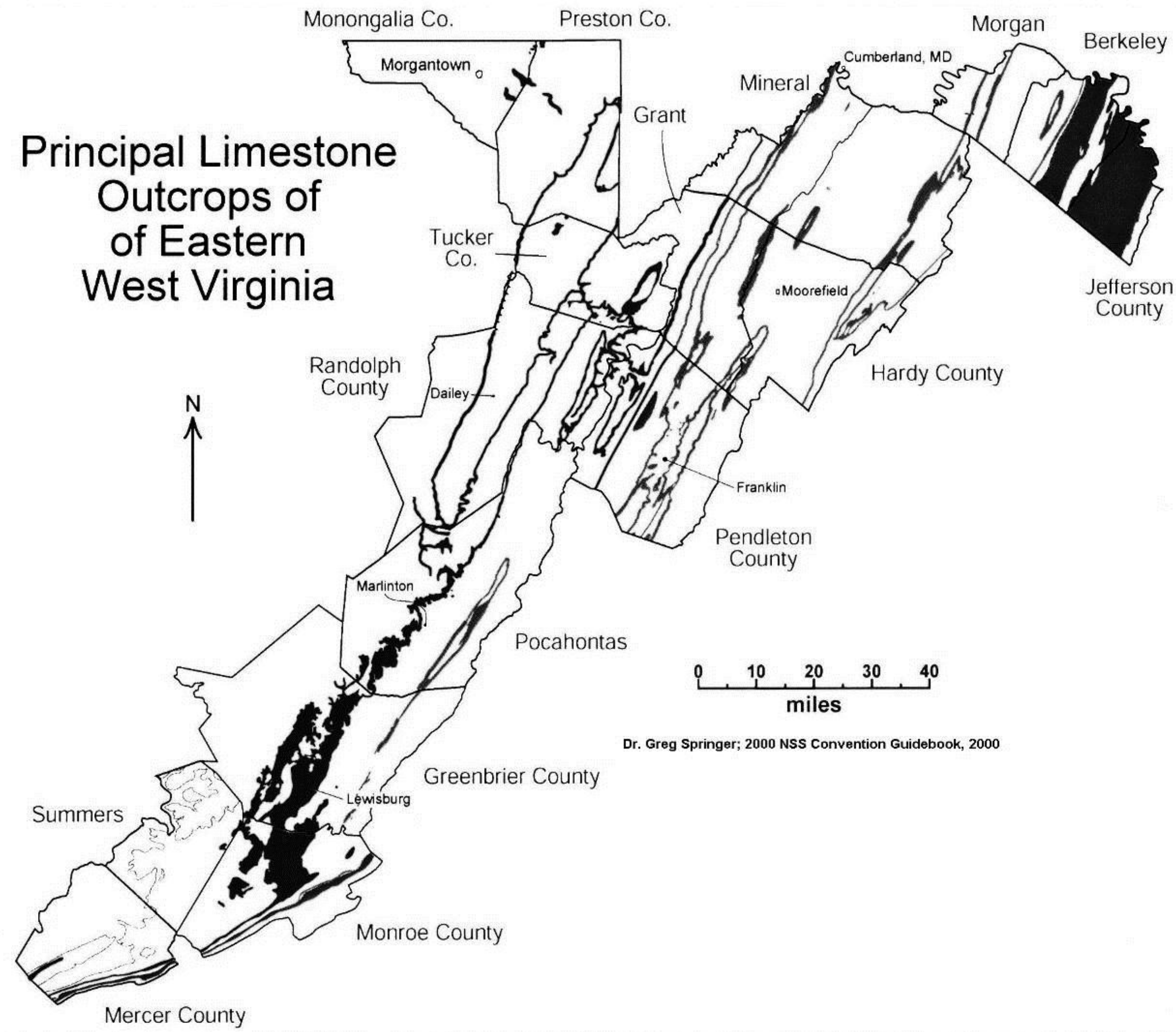
The word Karst is the German word for Kras, a region in Slovenia that sits on a limestone plateau. It was this location where the first scientific research of karst topography was made. Karst topography is a landscape of distinctive dissolution patterns marked by underground drainages. These are areas 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 and initial abstraction 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 and/or the initial abstraction value can be done using the roadway profile elevation and its corresponding headwater for the existing structure. This new curve number and/or initial abstraction 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

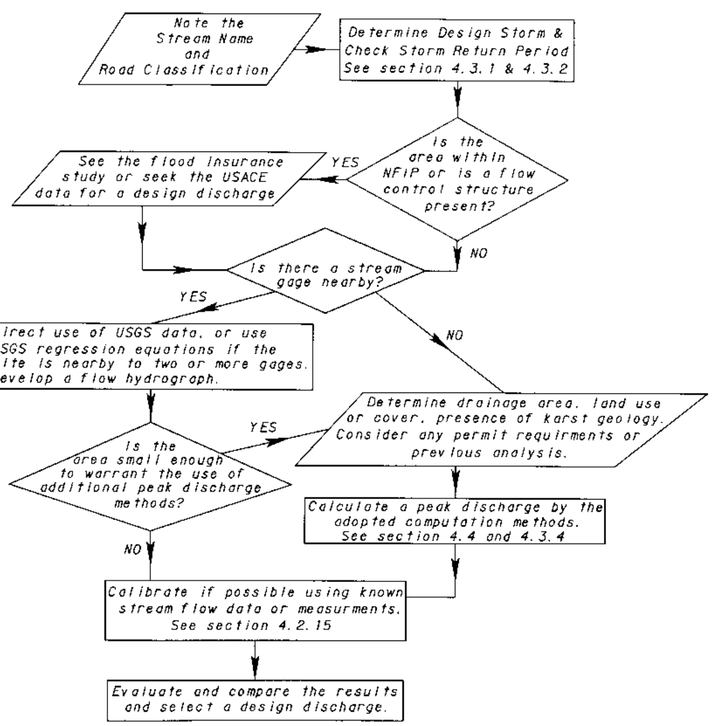
Unfortunately, stream flow gage records of sufficient length are seldom available and many small drainage areas have no records. In such cases, it is accepted practice to estimate peak discharge rates and hydrographs using statistical or empirical methods.

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	TR - 55 5 acres - 16,000 acres	USGS METHOD 10 square miles - 1,619 square miles
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" = __________ $C = \sum(CA)/\sum A$	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 $q_p = q_u (A \text{ in } mi^2)\, Q\, F_p$	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_p = __________ 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

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: ____________________	FIS DATE: ____________________	REGION: FROM MAP 4-9
COMMUNITY NAME AND NUMBER: ____________________ ____________________	COMMUNITY NAME AND NUMBER: ____________________ ____________________	EASTERN PANHANDLE ☐ CENTRAL MOUNTAINS ☐ WESTERN PLATEAUS ☐
LOCATION: ____________________ ____________________	LOCATION: ____________________ ____________________	
METHOD: RAINFALL RUNOFF MODEL ☐ REGIONAL REGRESSION EQUATIONS ☐ ANALYSIS OF GAGE RECORDS ☐ OTHER (PROVIDE DESCRIPTION) ☐	METHOD: RAINFALL RUNOFF MODEL ☐ REG. REGRESSION EQUATIONS ☐ ANALYSIS OF GAGE RECORDS ☐ OTHER (PROVIDE DESCRIPTION) ☐	EQUATION: FROM TABLE 4-15 Eqn:__________________ PRELIMINARY DESIGN DRAINAGE AREA 5 TO 10 MI^2
DESCRIPTION: ____________________ ____________________	DESCRIPTION: ____________________ ____________________	ADD THE STANDARD PREDICITION ERROR = __________ %
ATTACH COPY OF SUMMARY OF DISCHARGES TABLE FROM FIS	ATTACH COPY OF DATA OF FROM USACE	
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		
	Recommended	Value Range
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	
Broken soil, Clean, No Residual Vegetation	0.050	
Broken Soil, Vegetation Cover ≤ 20%	0.060	
Broken Soil, Vegetation Cover > 20%	0.170	
Open Field, Undisturbed Bare Soil, No Vegetation	0.025	0.020 - 0.030
Open Field, Short Grass, No Brush	0.030	0.025 - 0.035
Open Field, High Grass, No Brush	0.035	0.030 - 0.050
Scattered Brush Covered Area, Heavy Weeds	0.050	0.035 - 0.070
Light Brush Covered Area, With Small trees	0.055	0.037 - 0.070
Medium to Dense Brush Covered Area	0.085	0.057 - 0.135
Forest Area, Cleared Land With Stumps	0.050	0.040 - 0.065
Forest, Heavy Stand, Few Downed Trees	0.100	0.080 - 0.120
Natural Undisturbed Range Land Thick Mix Of Grasses	0.130	

Partial 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

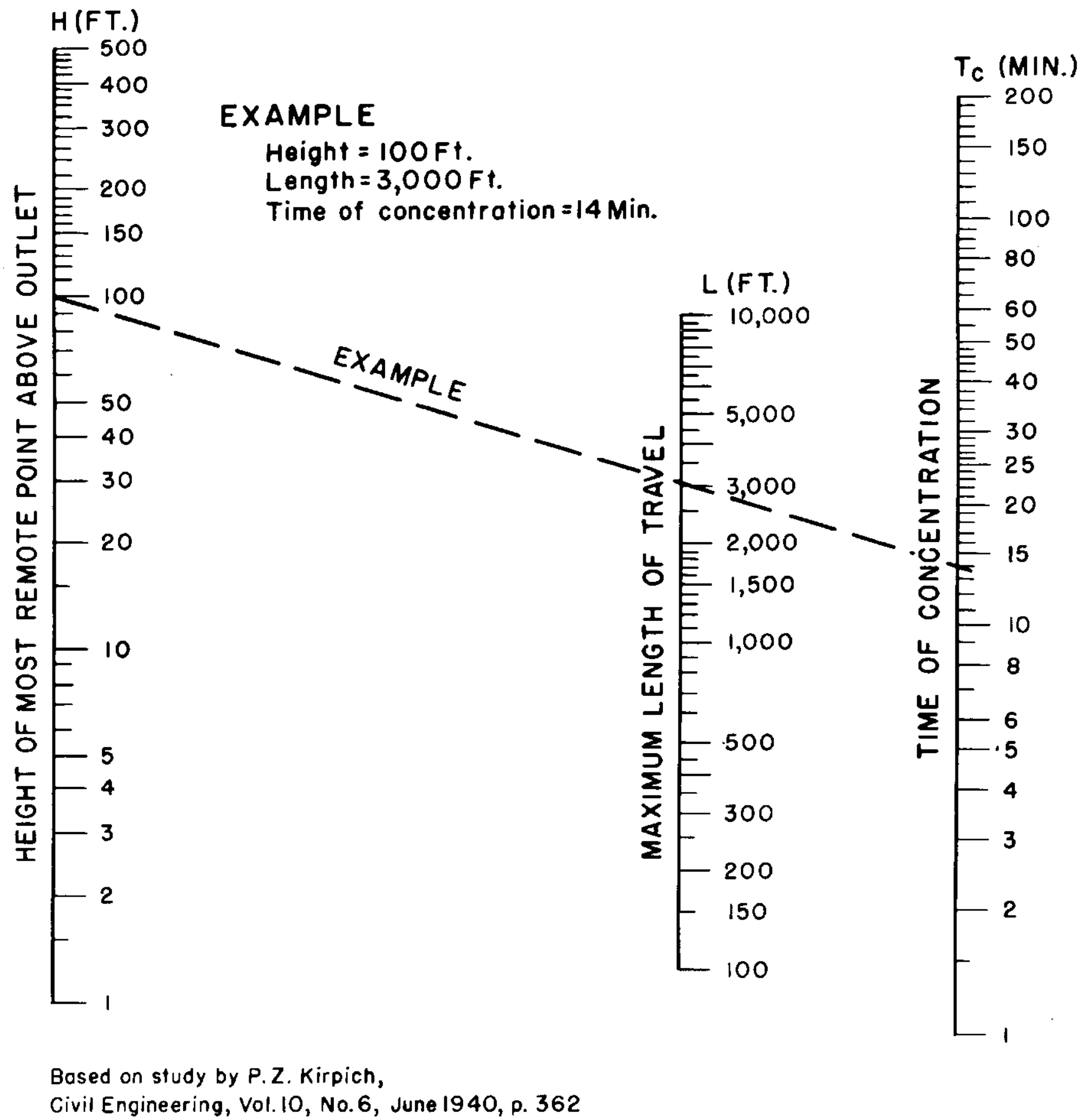
The rainfall intensity (i) is the average rainfall rate in inches per hour for a duration equal to the time of concentration for a selected return period. The rational method assumes uniform rainfall intensity over the total watershed.

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



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

Map 4-2

Rainfall Intensity-Frequency Regions of West Virginia

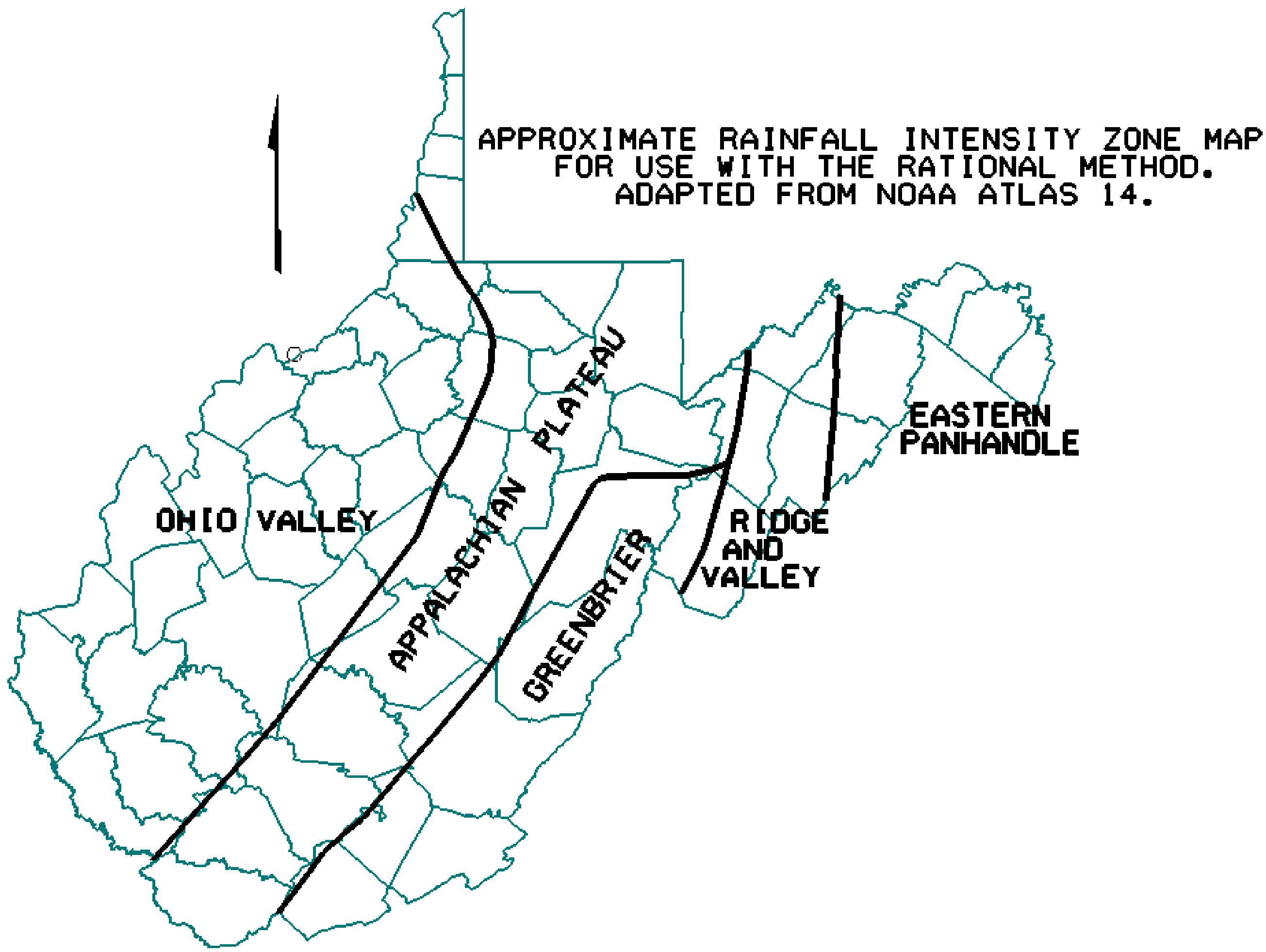
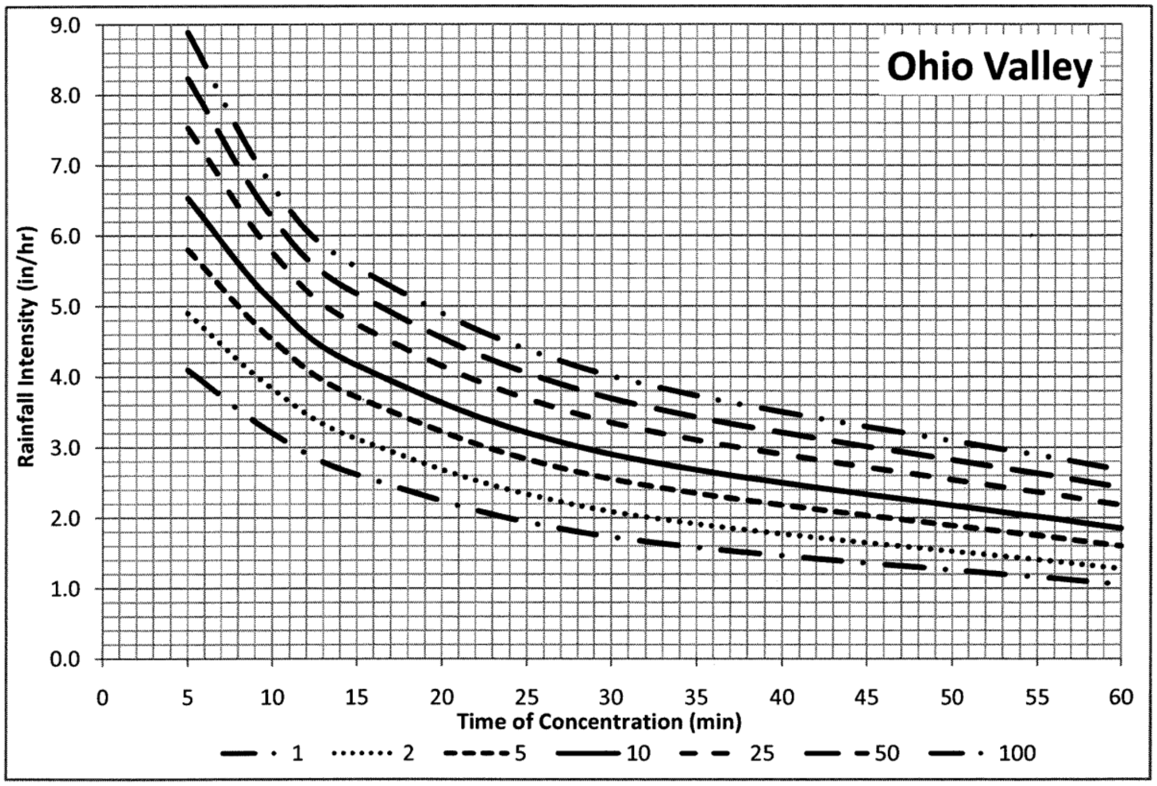


Chart 4-2

Intensity-Duration-Frequency Curves for West Virginia



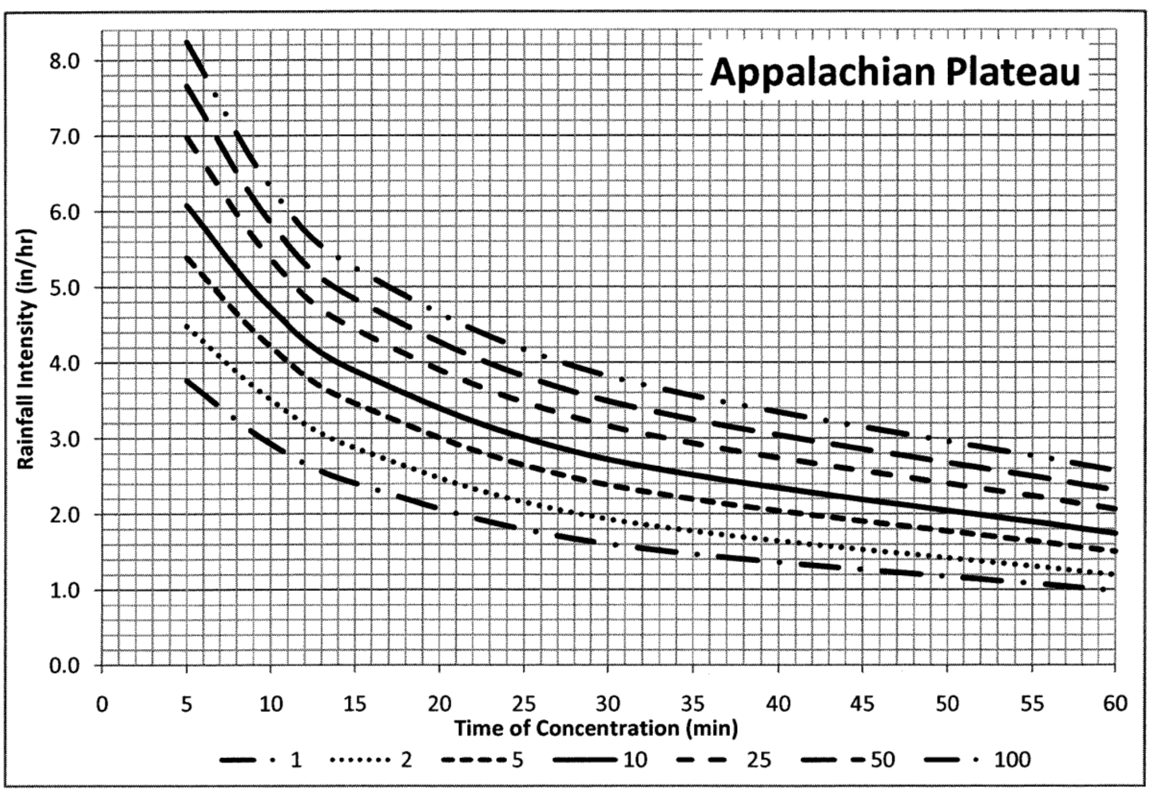
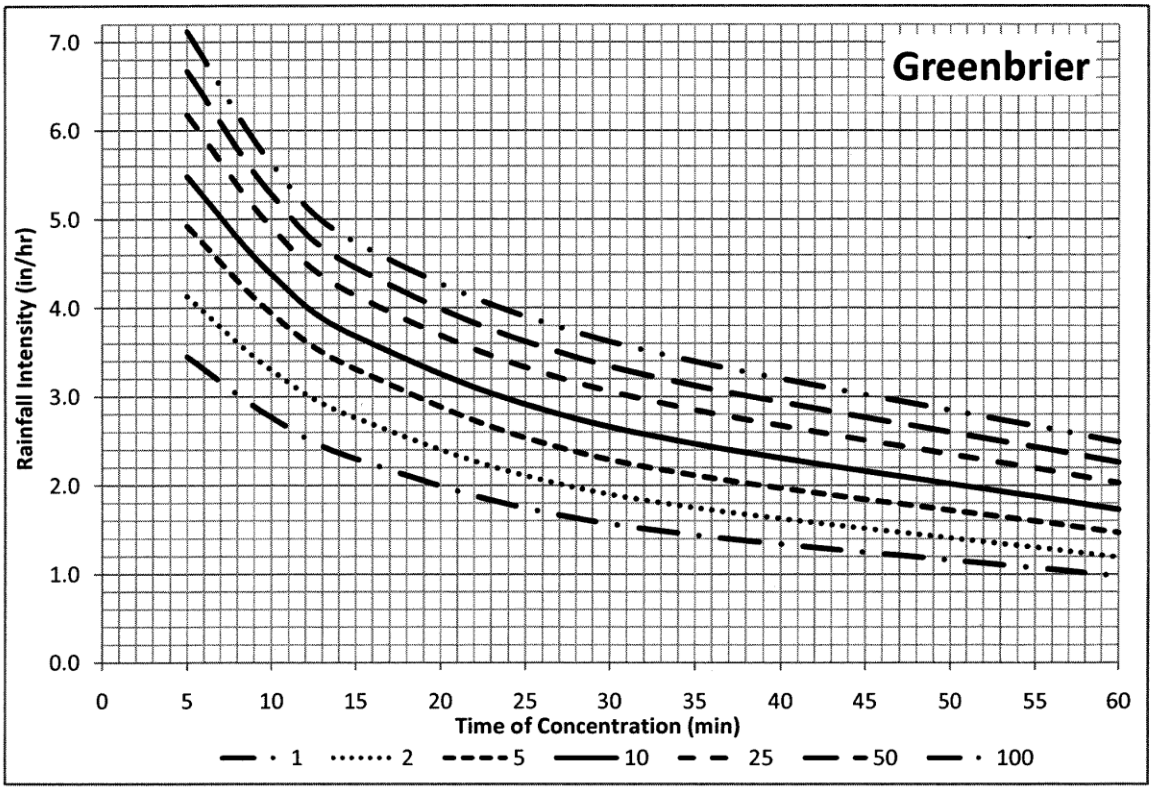


Chart 4-3

Intensity-Duration-Frequency Curves for West Virginia



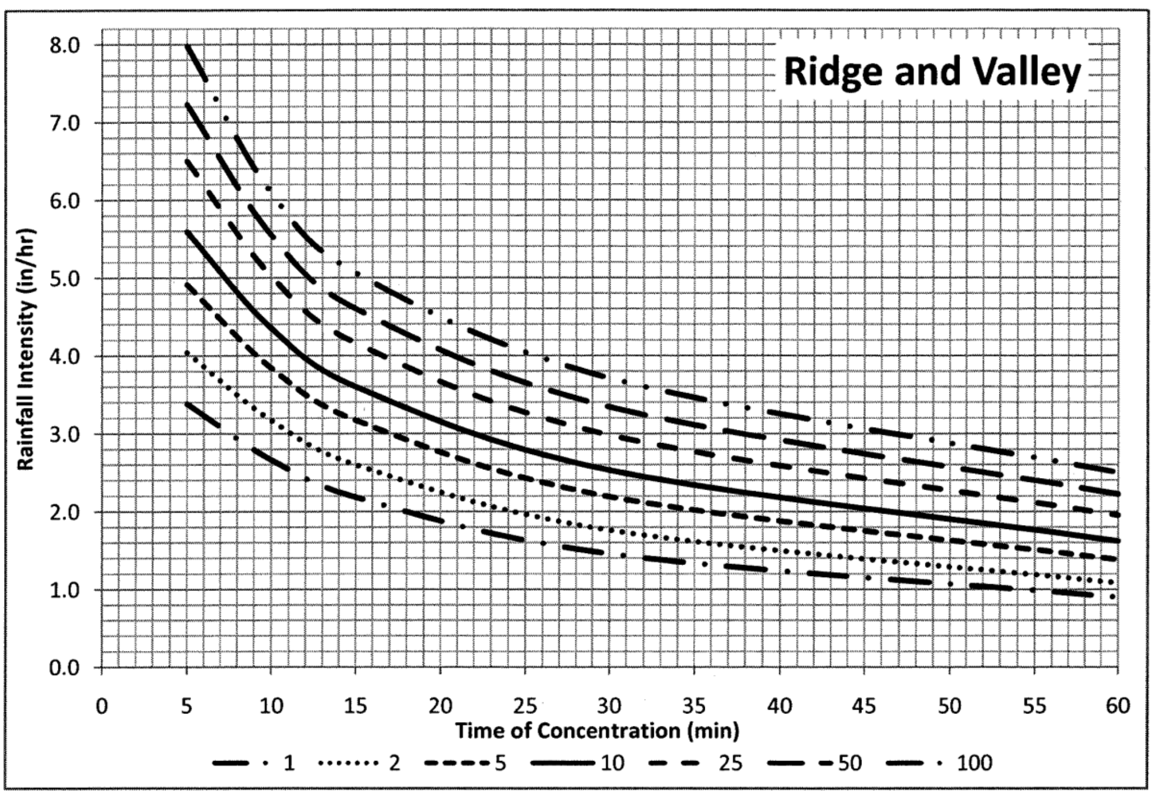
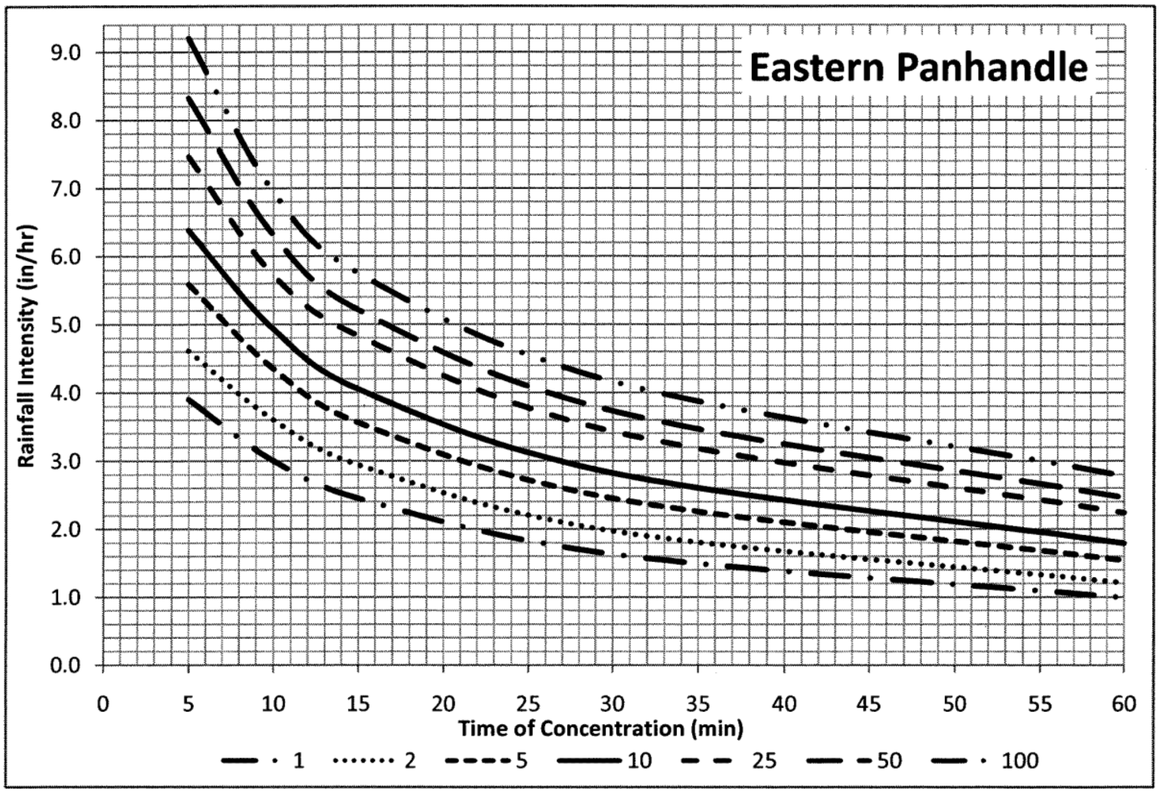


Chart 4-4

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.

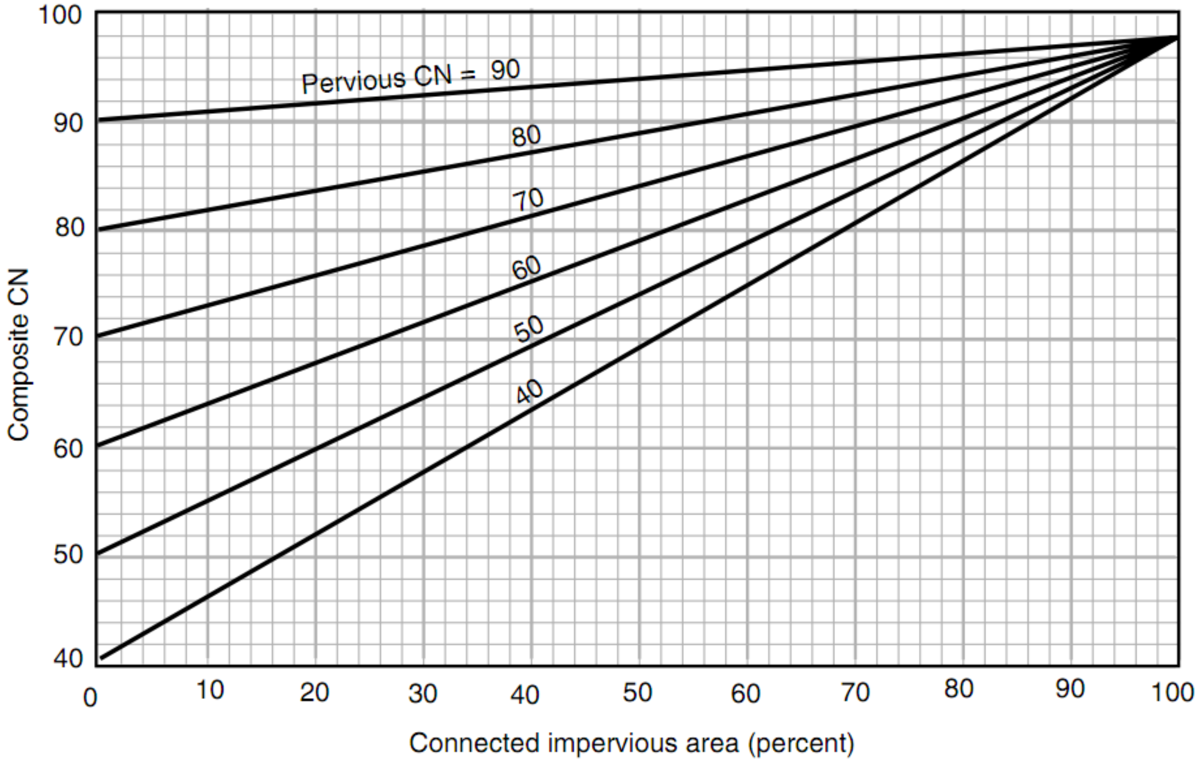
The urban curve numbers in Table 4-10 were developed on the assumptions that pervious urban areas are equivalent to pasture in good hydrologic condition with impervious areas that have a curve number of 98 and are directly connected to the drainage system. If all of the impervious area is directly connected 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

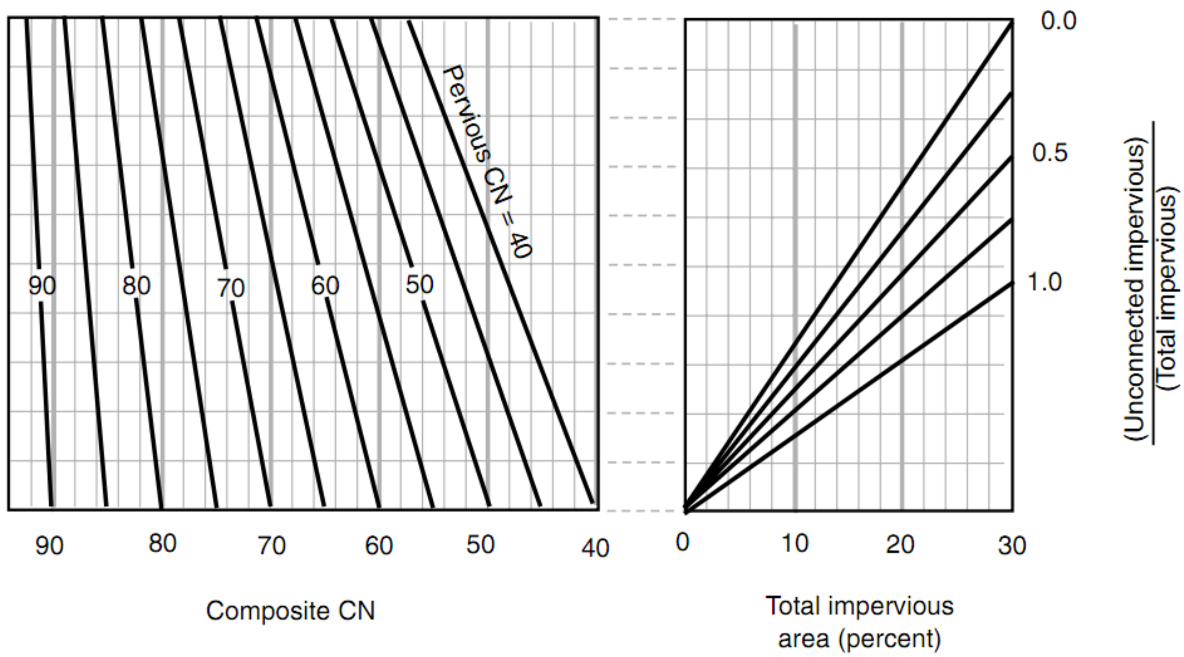
Runoff from these areas is spread over a pervious area as sheet flow before it reaches the drainage system. To determine the curve number when all or part of the impervious area is not directly connected to the drainage system and the total impervious area is less than 30%, use the bottom part of Chart 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:

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

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

$P_{2\text{-}24}$ = 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.

$$\text{Unpaved} \quad V = 16.1345\sqrt{S}$$
$$\text{Paved} \quad V = 20.3282\sqrt{S}$$

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.

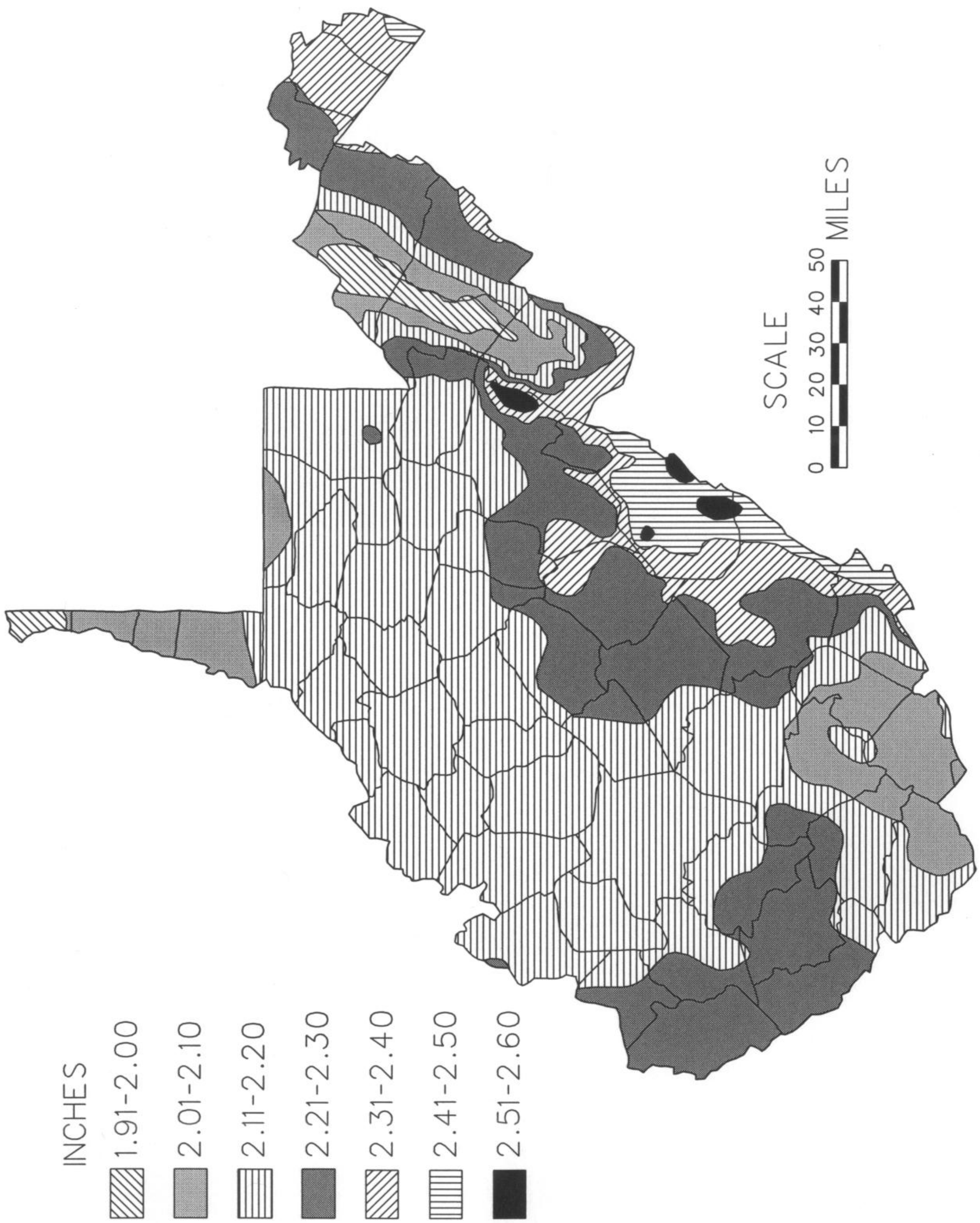
4.4.3.6 COMPUTATION PROCEDURE

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:

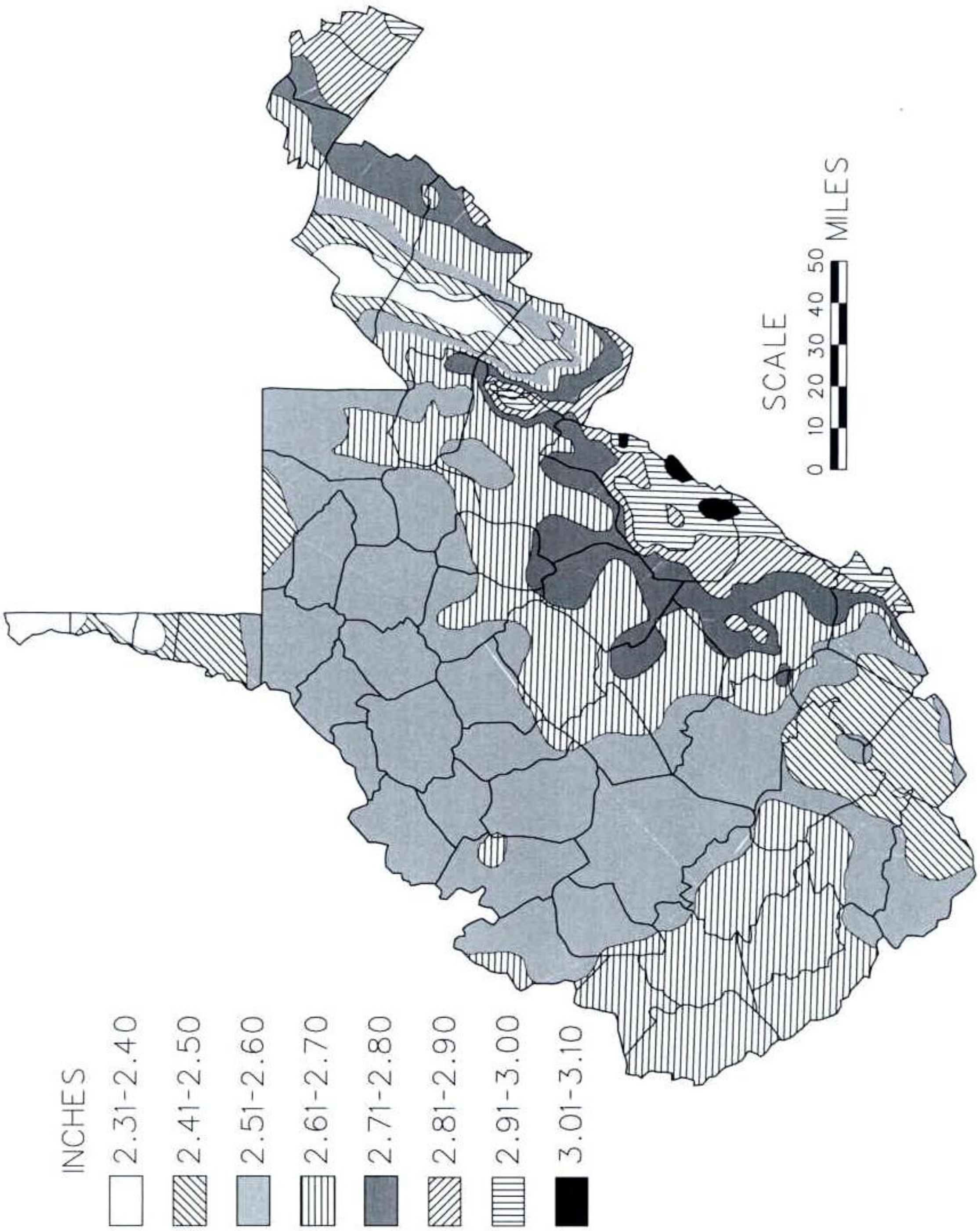
- Determine the Drainage Area (A_m) in square miles from a topographic map.
- Determine the Runoff Curve Number (CN) using Worksheet 4-1.
- Obtain the 24-hour Rainfall Depth (P) in inches for a selected frequency or return period from Map 4-3 through Map 4-9 (NOAA Atlas 14 rainfall maps).
- Map 4-3 through 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.

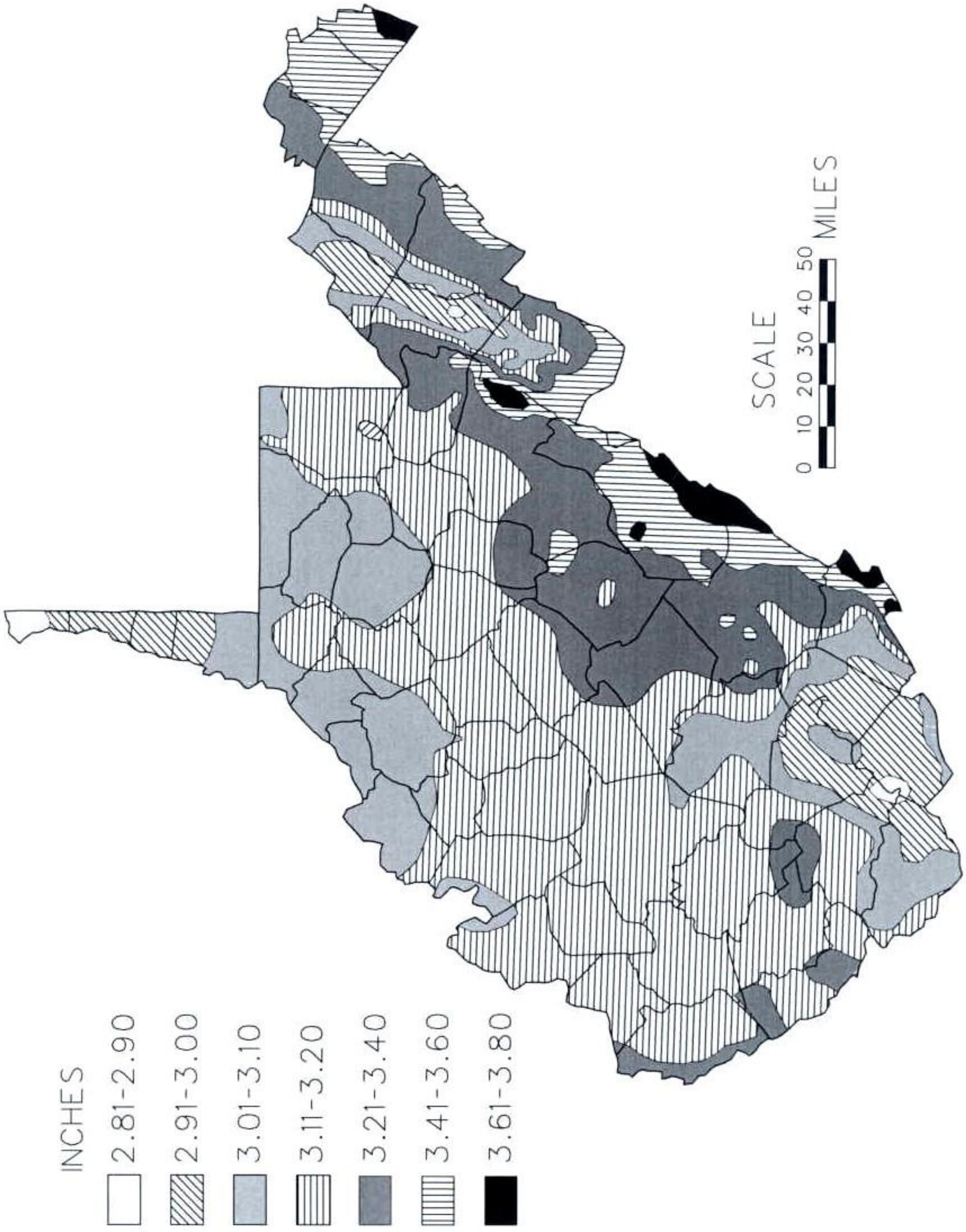
Map 4-4
24-hour Rainfall Depth for a 2 Year Return Period



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

Map 4-5

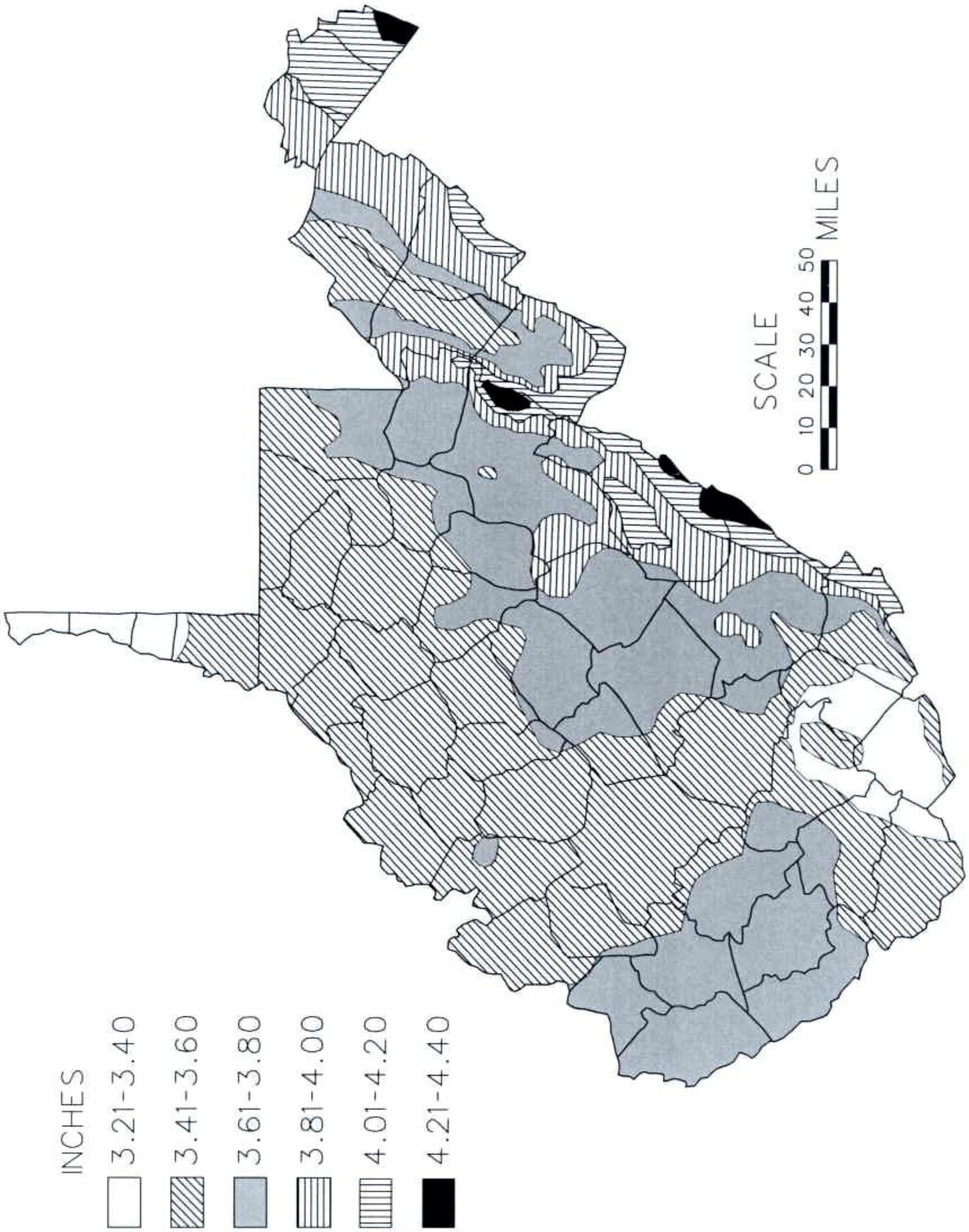
24-hour Rainfall Depth for a 5 Year Return Period



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

Map 4-6

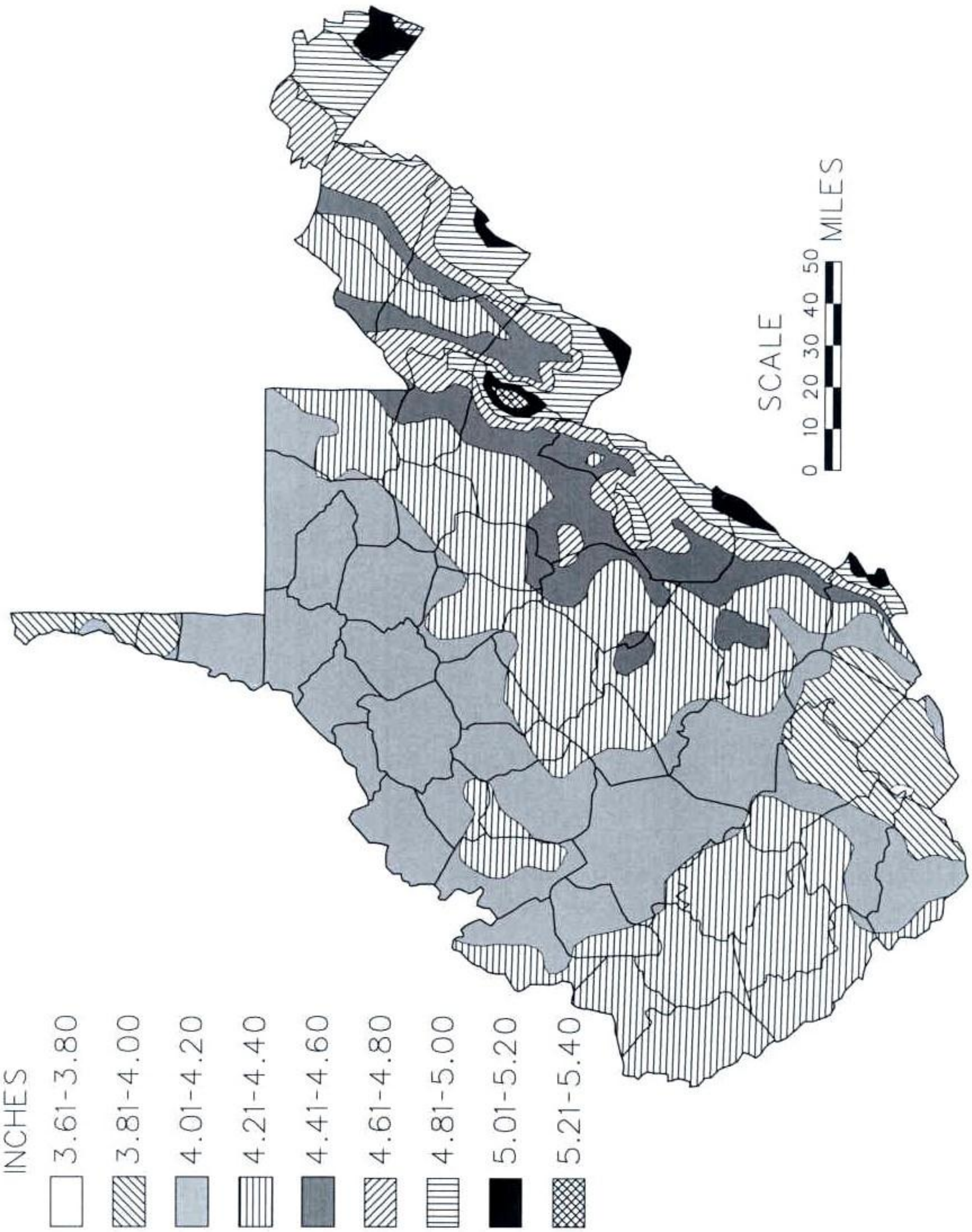
24-hour Rainfall Depth for a 10 Year Return Period



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

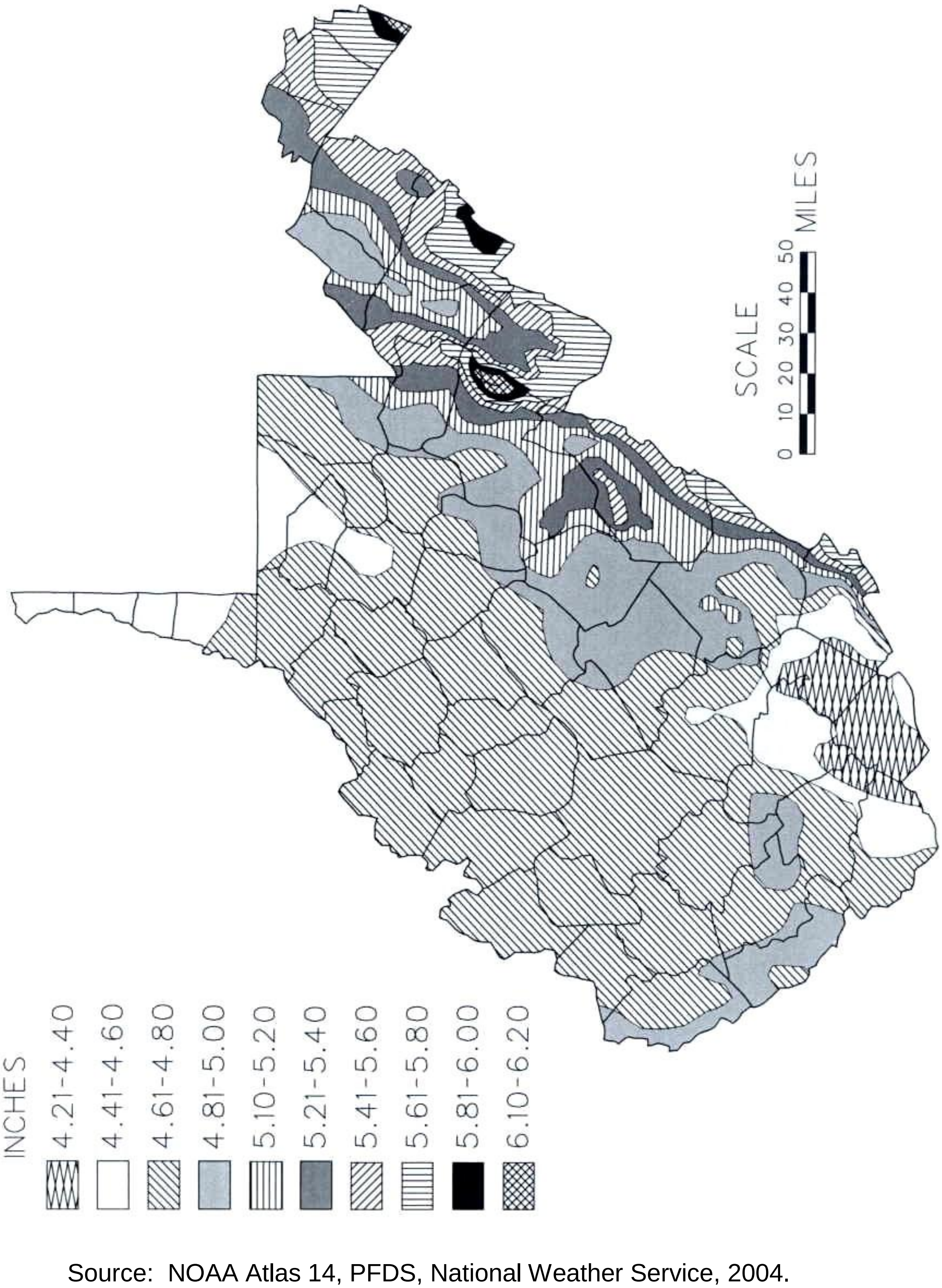
Map 4-7

24-hour Rainfall Depth for a 25 Year Return Period



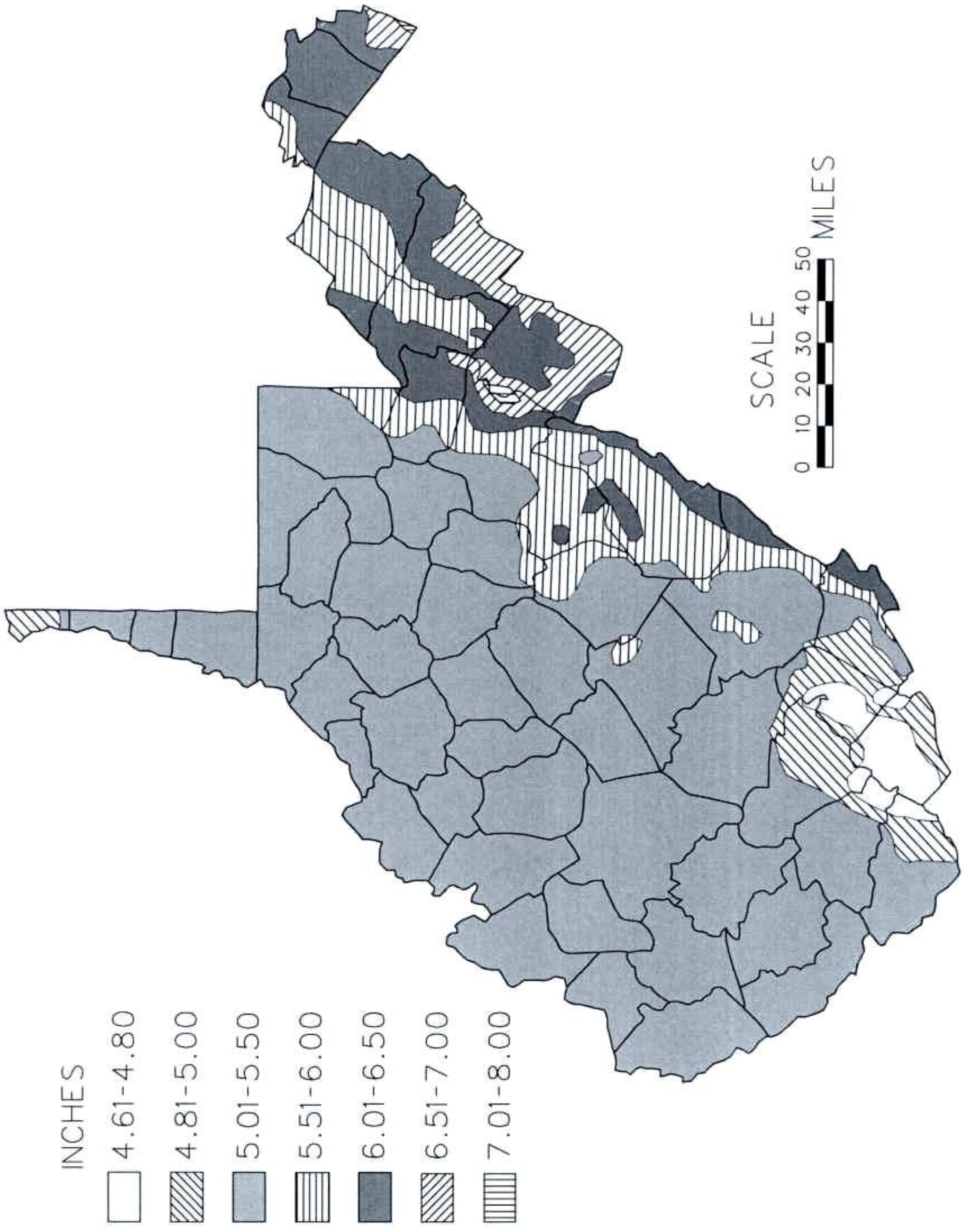
Source: NOAA Atlas 14, PFDS, National Weather Service, 2004.

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

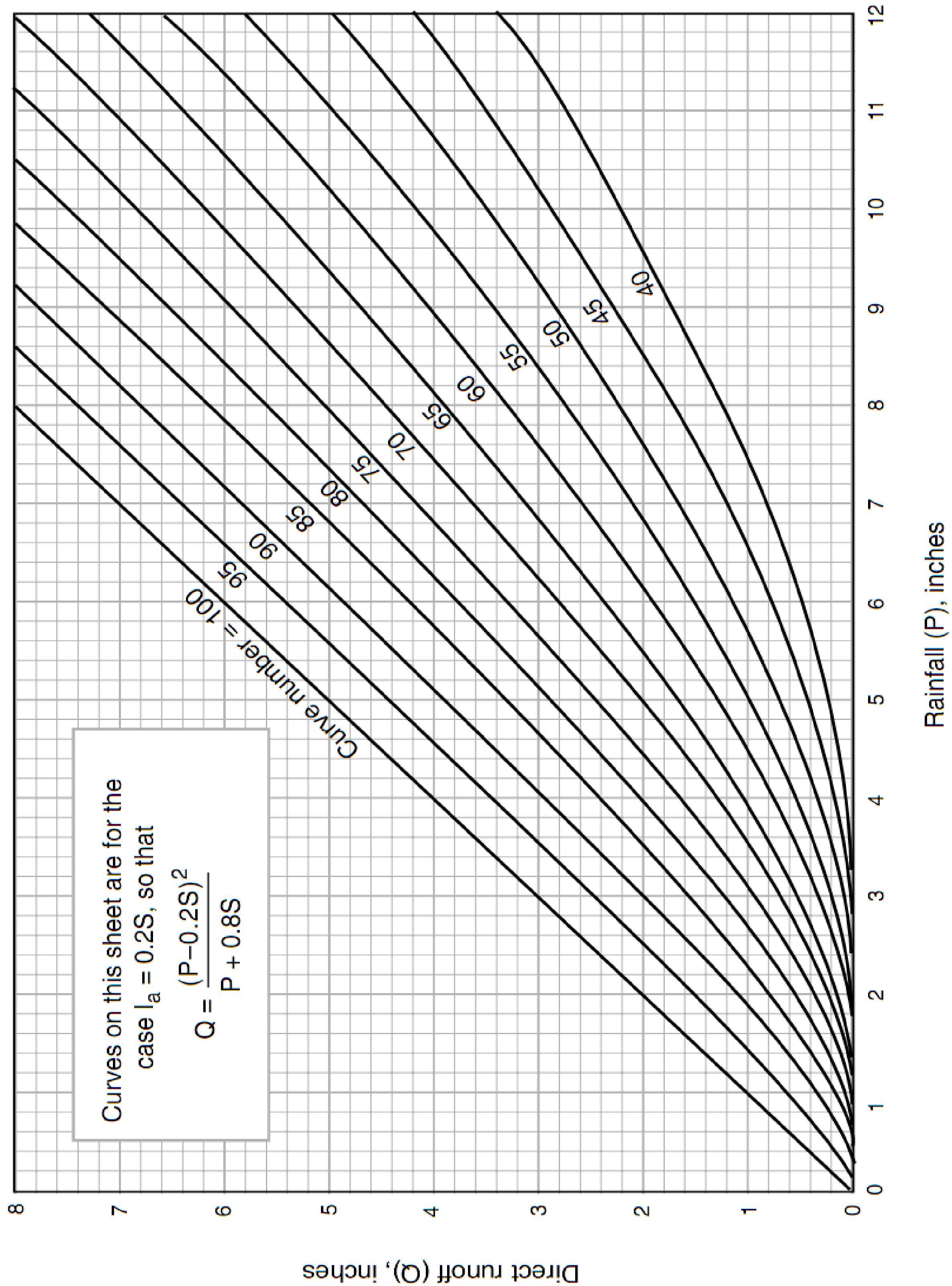
Rainfall	Runoff depth for curve number of—												
	40	45	50	55	60	65	70	75	80	85	90	95	98
	inches												
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	.00	.00	.00	.00	.00	.00	.03	.07	.15	.27	.46	.74	.99
1.4	.00	.00	.00	.00	.00	.02	.06	.13	.24	.39	.61	.92	1.18
1.6	.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: ___________

CHECKED BY: ____________ DATE: ___________

PROJECT NAME: ______________________________

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

	Area in mi^2	CN X Area
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		

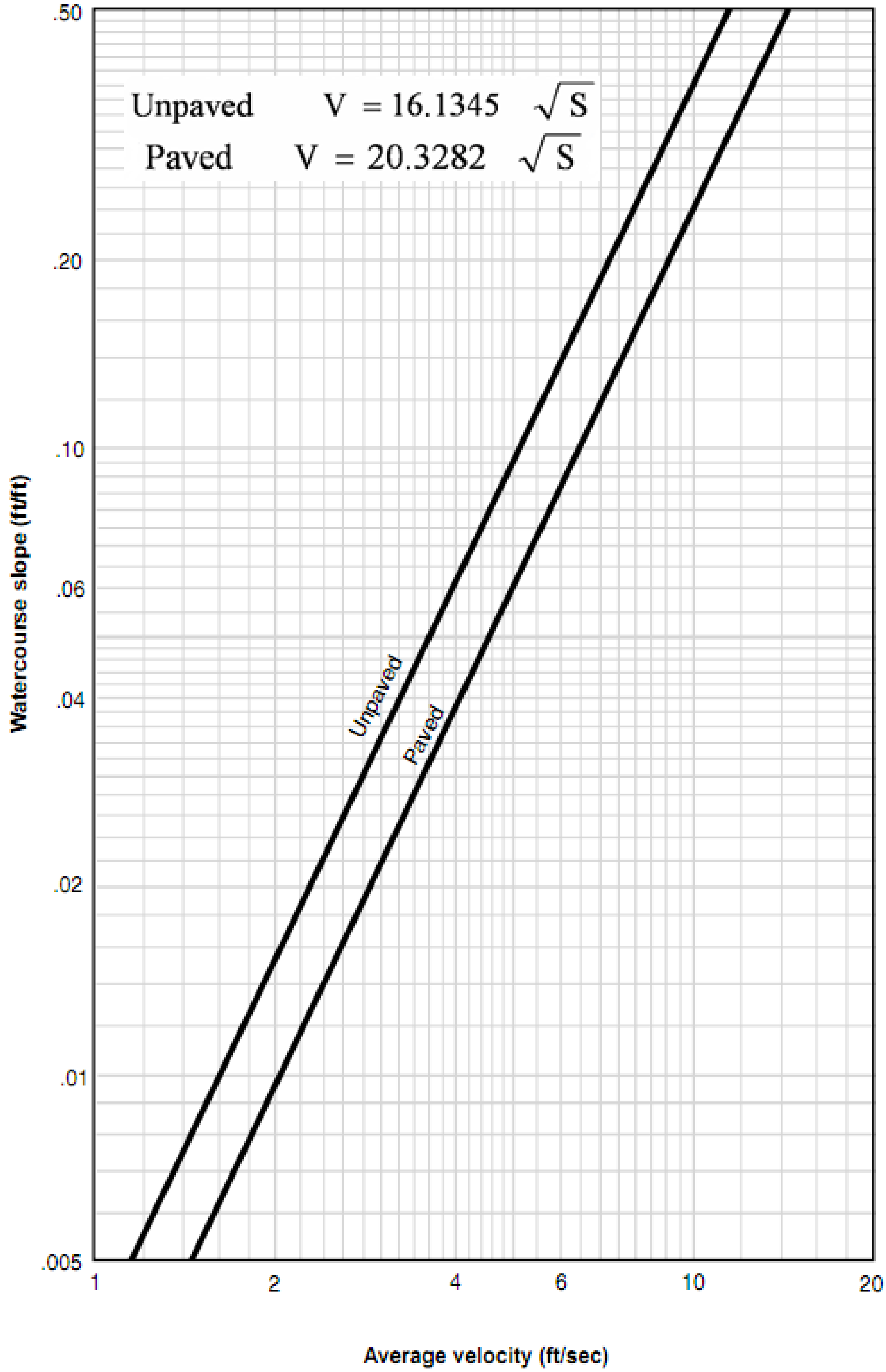
24 hour Rainfall Depth from Table 4-11, or Map 4-3 through Map 4-8

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 $\leq$ 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

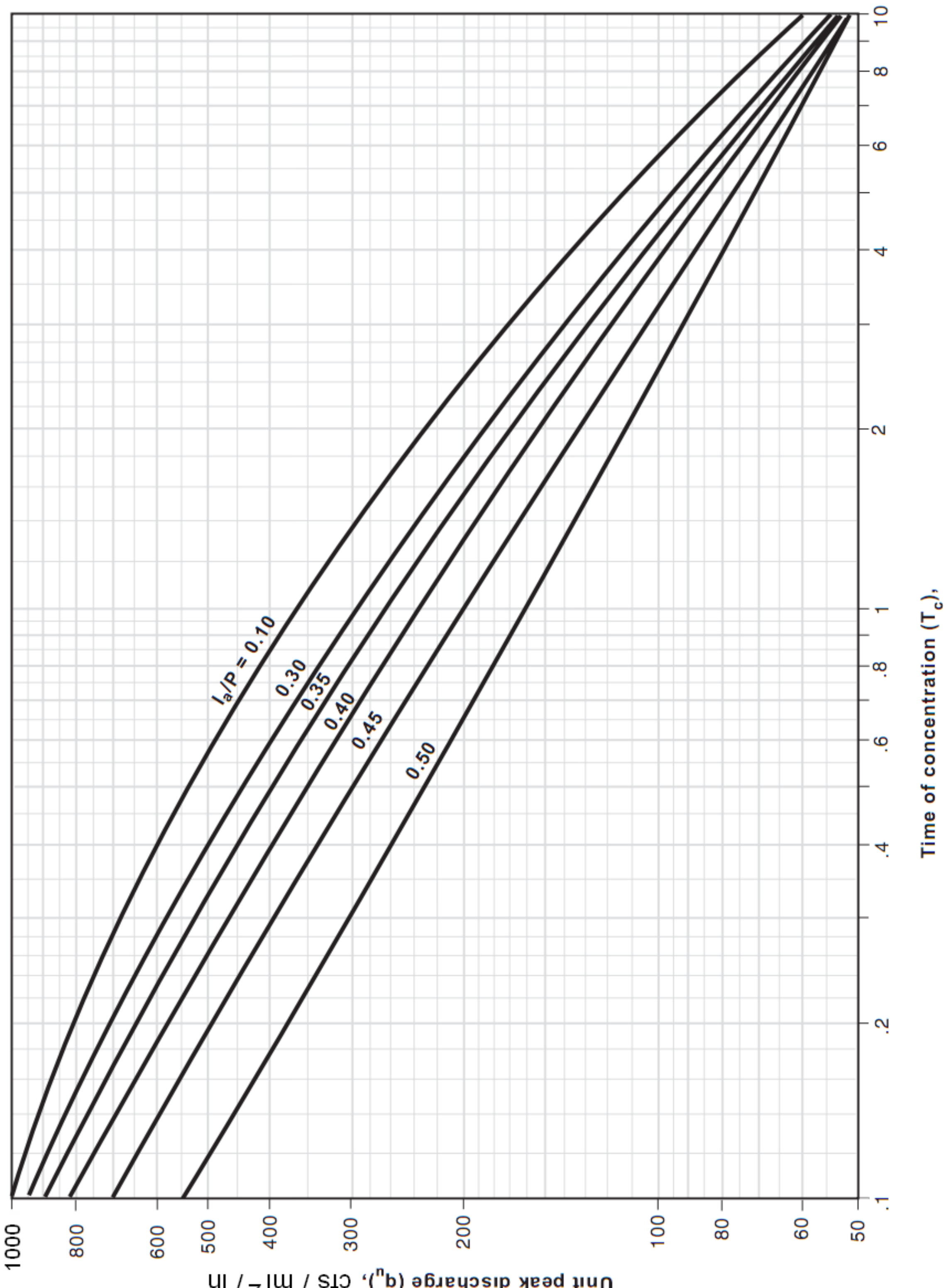
Table 4-12
Initial Abstraction Values (I_a) for Curve Numbers

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

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

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:

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

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.

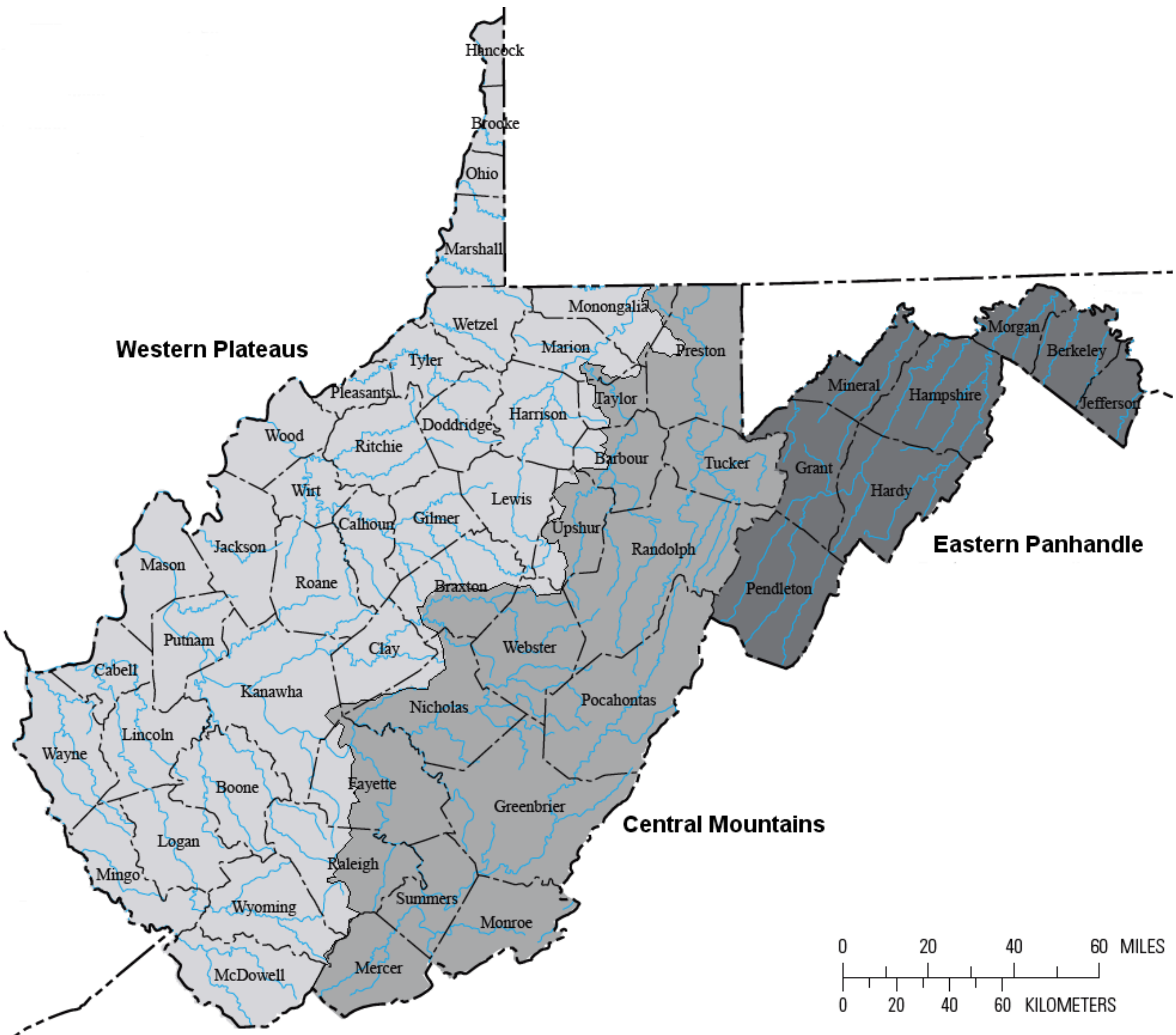
Engineering applications of the USGS Method primarily pertain to the hydraulic design of bridges and culverts.

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 here. The design of a stormwater management facility requires a volume of runoff from an area before the land use or land cover was changed due to development. Each of these methods can be used to determine this pre-development runoff volume. 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 trapezoidal shaped hydrographs for small drainage areas of less than 200 acres, after calculating the peak discharge using the Rational Method. Hydrographs produced by this method can be used for sizing storm water detention ponds, 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.

4.5 REFERENCES

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(2) A Method for Estimating Volume and Rate of Runoff in Small Watersheds, SCS-TP-149, NRCS, April 1973

(3) Drainage Manual, West Virginia Division of Highways, 1984

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(9) Highway Drainage Guidelines, Metric Edition, AASHTO, 1999

(10) National Flood Frequency Computer Program, USGS, Website: http://water.usgs.gov/software/nff.html.

(11) Precipitation Frequency Data Server (PFDS), National Weather Service, Web address: http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=wv

(12) Runoff Estimates for Small Watersheds and Development of a Sound Design Method, Volume II, FHWA-RD-77-159, Fletcher et al., FHWA, 1977

(13) SCS National Engineering Handbook, Section 4, Hydrology, August 1972

(14) Technical Release No. 20 (TR-20), Computer Program for Project Formulation-Hydrology, Natural Resources Conservation Service (Soil Conservation Service), U.S. Department of Agriculture, 1983

(15) 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

(16) Technical Release No. 55 (TR-55) computer program, NRCS Website: http://go.usa.gov/KoZ

(17) Time of Concentration of Small Agricultural Watersheds, Civil Engineering, Volume 10, No. 6, P.Z. Kirpich, , June 1940

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 5 feet or the shoulder width, whichever is greater. If the design speed is less than 40 mph, spread is generally limited to 5 feet, or the shoulder width plus 3 feet into the traveled way, whichever is greater. 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

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:

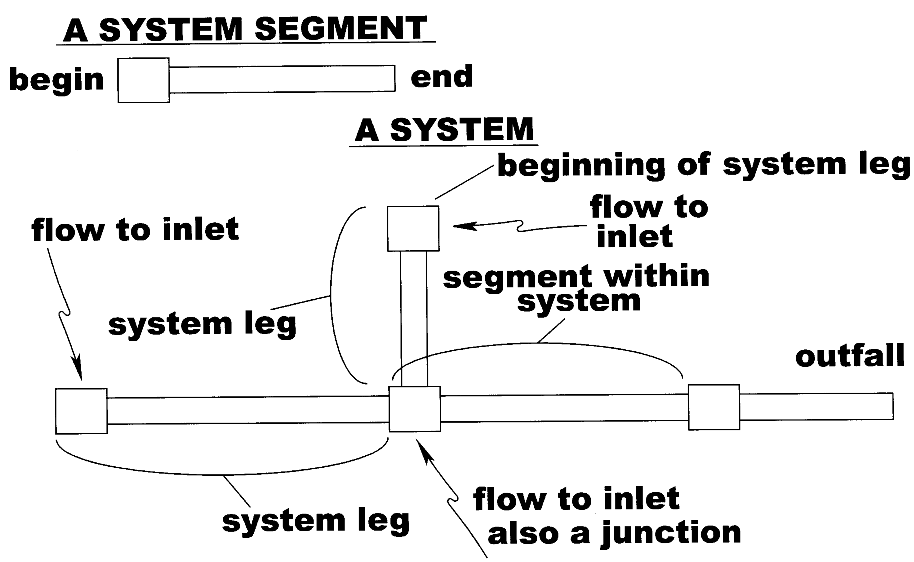
$$E_o' = E_o\left(\frac{A_W'}{A_W}\right)$$

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

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.

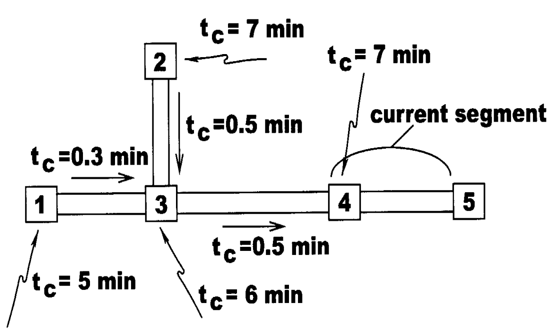


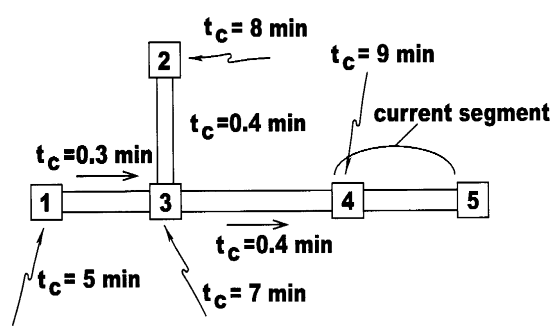
STEP 1, PIPE ID: Start at the beginning of a system leg on line 1. Fill in column 1 (FROM POINT) with the inlet label from the plan sheet. Fill in column 2 (TO POINT) with the proceeding inlet label from the plan sheet. All legs attached to the next segment shall be designed before continuing on to a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

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CHAPTER 8:
CULVERTS

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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

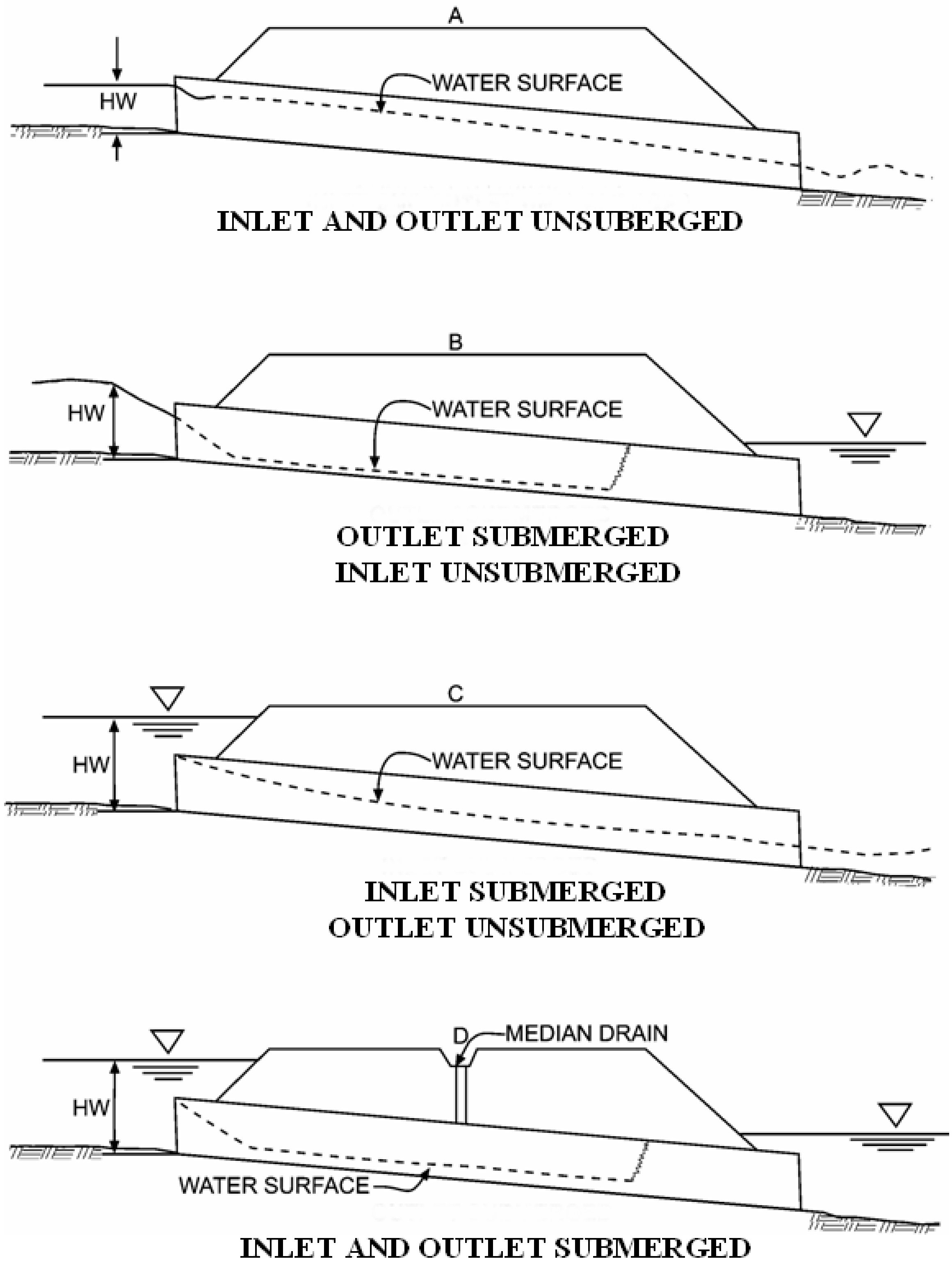
A culvert flowing in inlet control has shallow, high-velocity, supercritical flow with the control section located at the upstream end of the barrel. Inlet control is influenced by the headwater depth and inlet area, edge configuration and shape. Figure 8–1 shows several examples of inlet control flow 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.

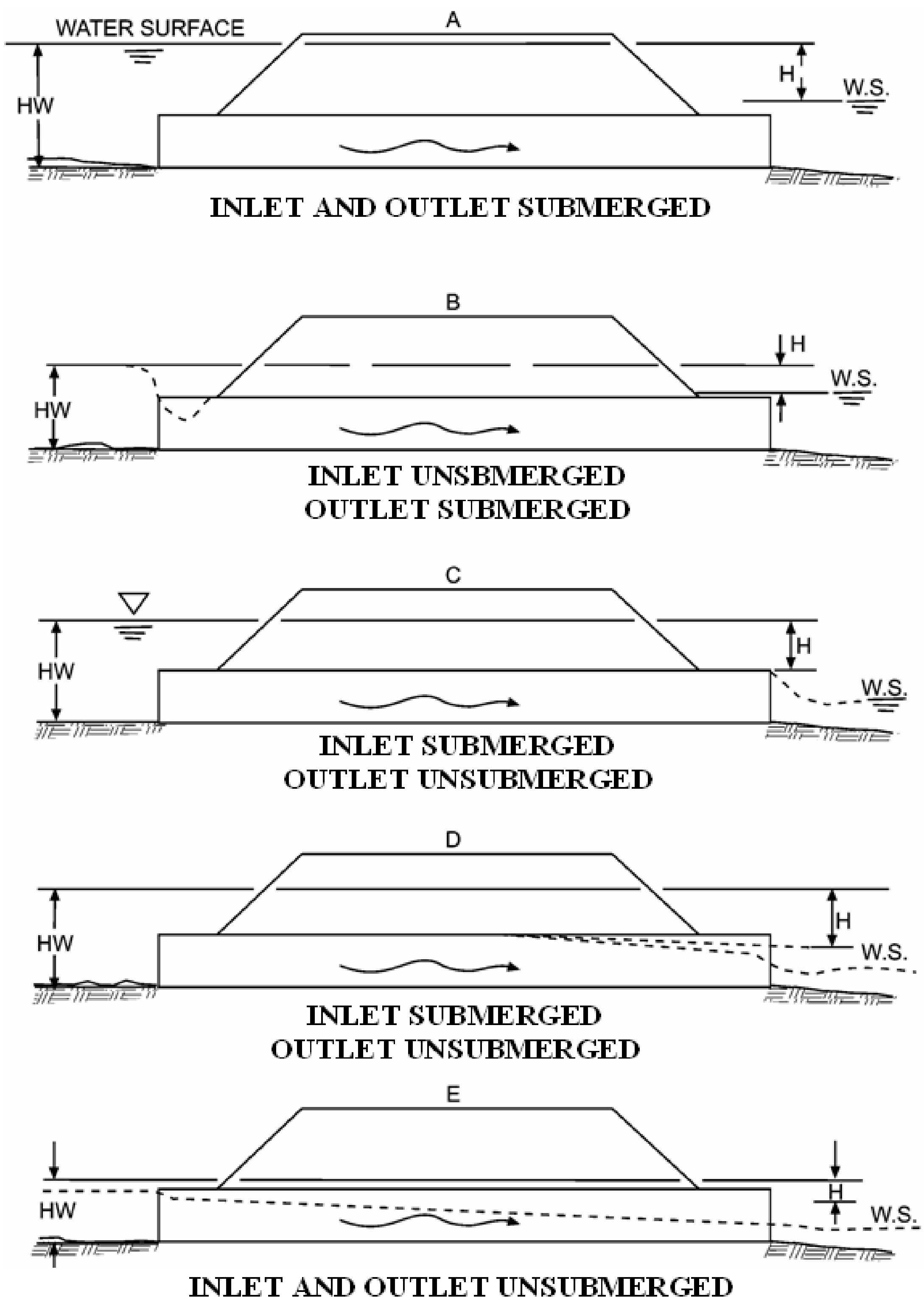
Figure 8–1

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) $$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M - 0.5S^2$$

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) $$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M + 0.7S^2$$

Form (2) $$\frac{HW_i}{D} = K\left[\frac{K_u Q}{AD^{0.5}}\right]^M$$

SUBMERGED

$$\frac{HW_i}{D} = c\left[\frac{K_u Q}{AD^{0.5}}\right]^2 + Y - 0.5S^2$$

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: $V = \dfrac{Q}{A}$

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: $H_v = \dfrac{V^2}{2g}$

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:

$$H_f = \left[\frac{29n^2L}{R^{1.33}} \right] \frac{V^2}{2g}$$

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 <u>less than</u> 24 inches in diameter shall have an n value of 0.015. Corrugated metal pipes <u>greater than</u> 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 <u>helical</u> corrugations, <u>not annular</u> 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

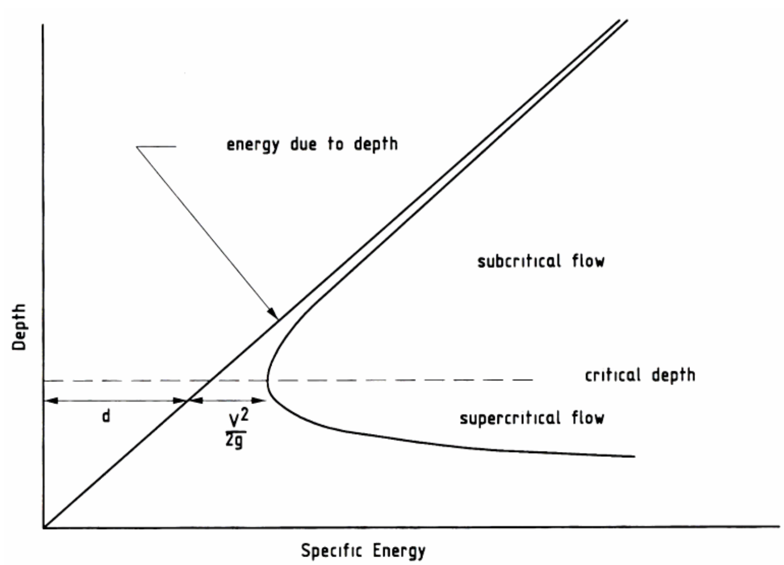
Specific Energy

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.

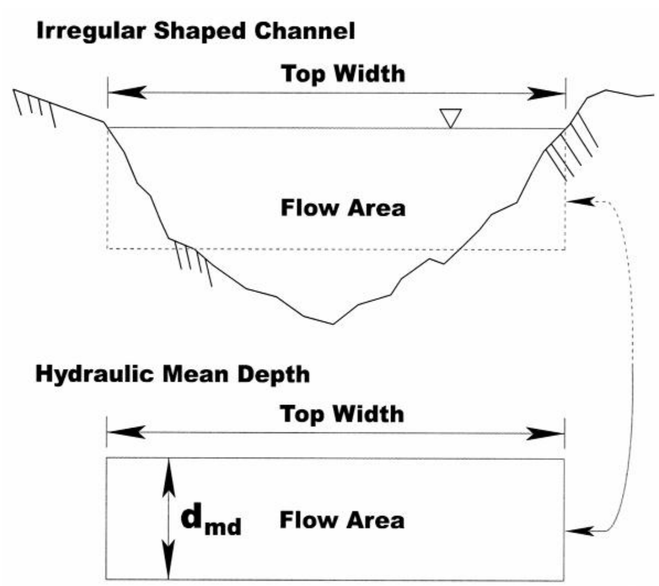
Normal depth is a function of discharge, size of channel, shape of channel, slope of channel, and frictional resistance to flow. It can be calculated using the familiar Manning's Equation.

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

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.

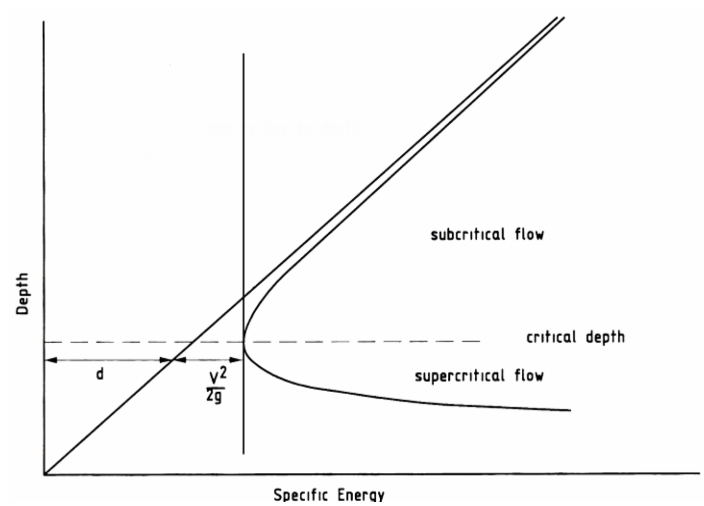
Where: T = flow top width



$$d_{md} = \frac{A}{T}$$

<u>Critical Depth</u>

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).

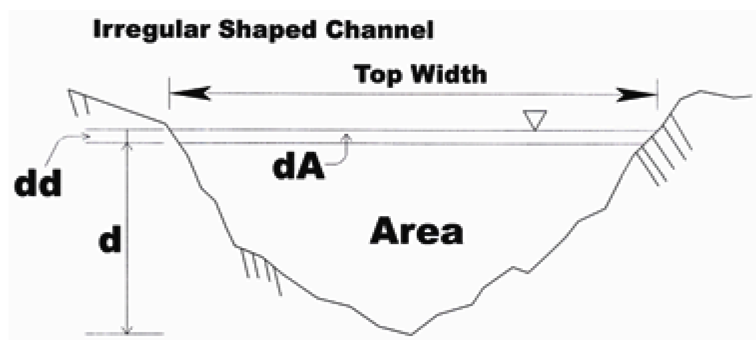


$$E = \frac{Q^2}{2gA^2} + d$$

$$E = \frac{Q^2}{2g}\frac{1}{A^2} + d$$

$$\frac{dE}{dd} = \frac{Q^2}{2g}\frac{-2}{A^3}\frac{dA}{dd} + 1$$

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: $\frac{V^2}{g\, d_{md}} = 1$ or $\frac{V}{\sqrt{g\, d_{md}}} = 1$

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.

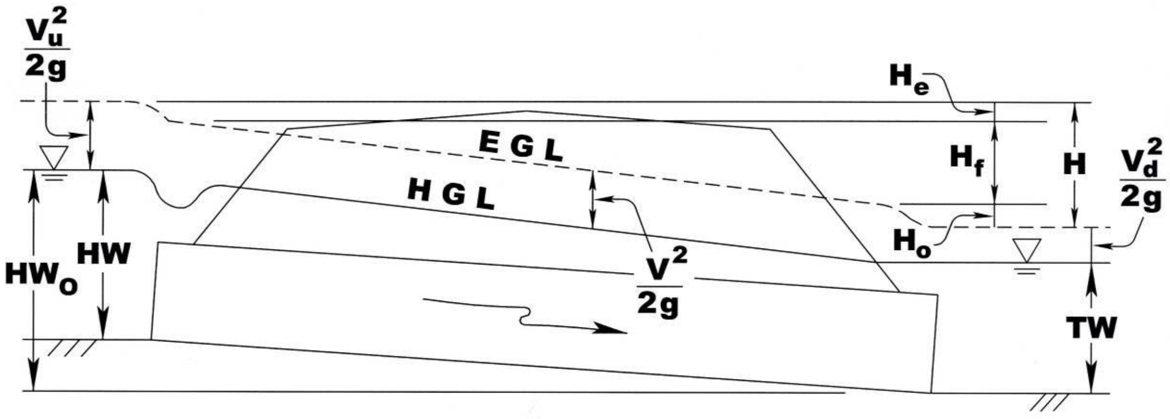
The roughness of the interior of the pipe varies with the type of backfill. When the non-compacted cementitious material, referred to as controlled low-strength material (see DOH specification 219) is used as backfill (type F trench see WVDOH Typical Sections and Related Details page 54) the interior 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_o 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

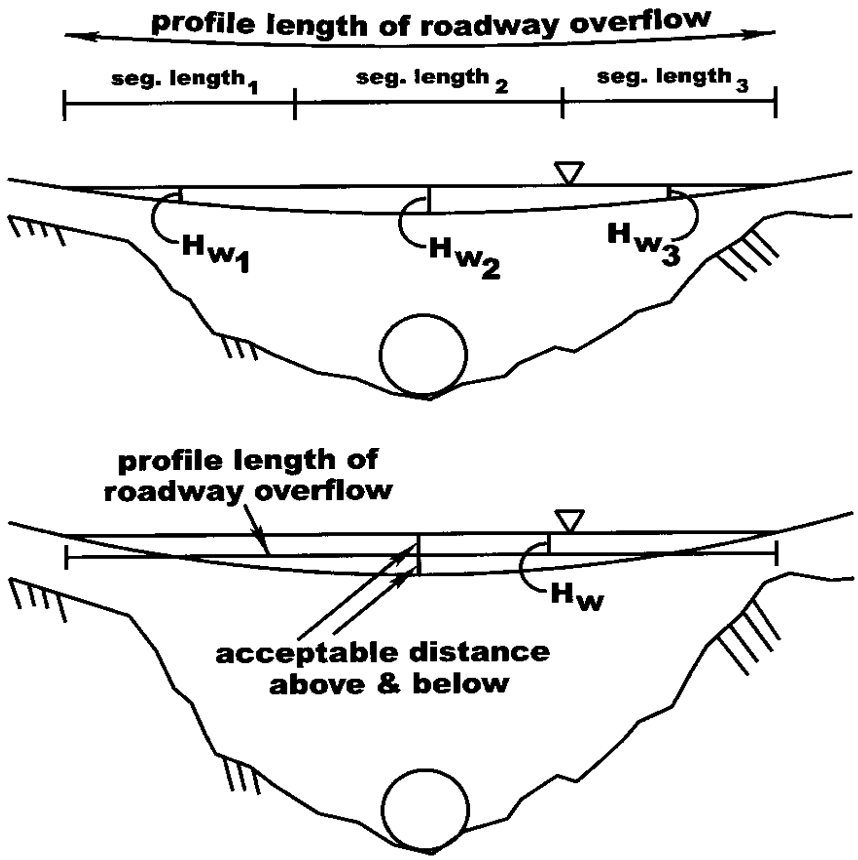
L = Profile length of the roadway overflow (ft)

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

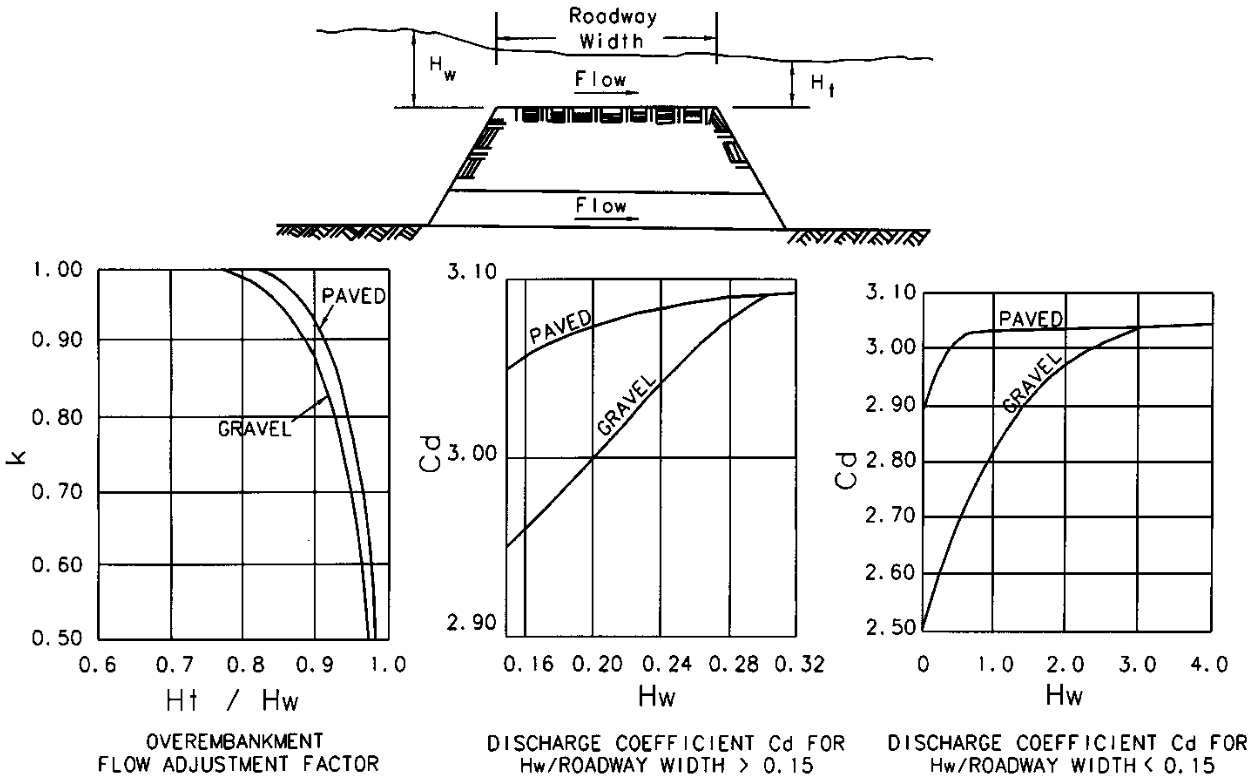
Roadway Overtopping



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

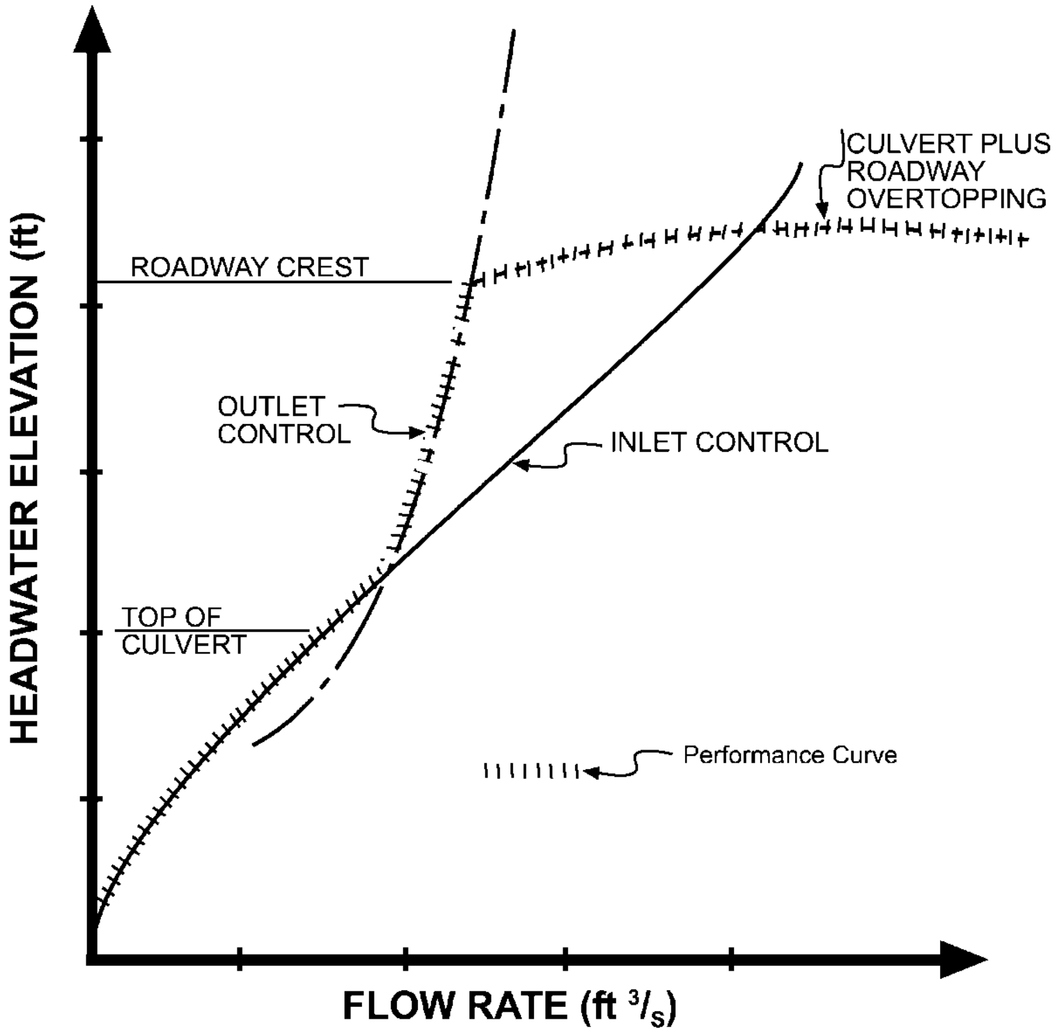


8.4.8 PERFORMANCE CURVES

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

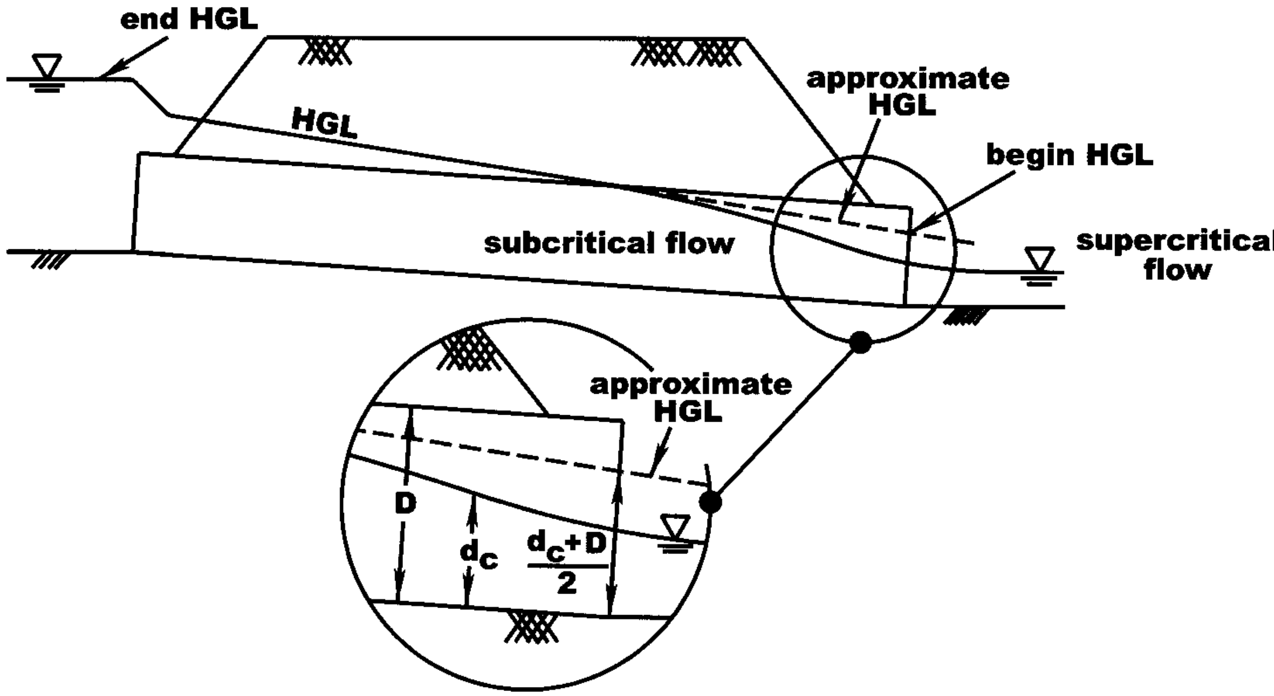
The tailwater elevation above the outlet invert at the design flow rate may be obtained from backwater or normal depth calculations, or from field observations. A field inspection and a review of flood insurance studies should be made to check the effect of downstream controls on tailwater conditions 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.

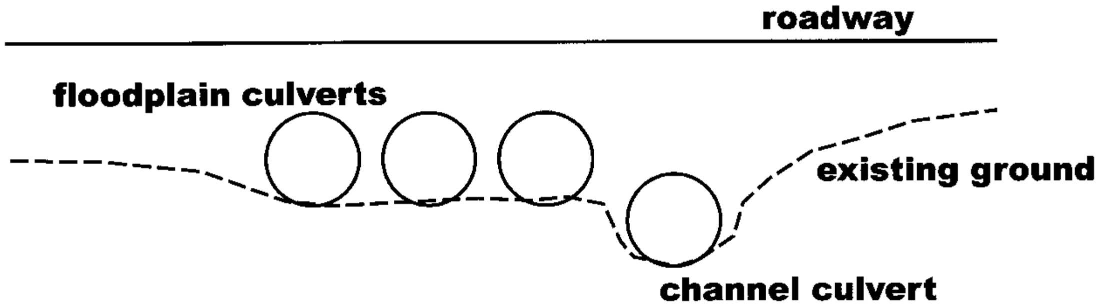
The type of debris protection will depend on the individual location. Debris interceptors can be placed at the entrance to the culvert or upstream of the culvert. Upstream interceptors come in the form of debris racks, floating drift booms and debris basins. Culvert entrance interceptors include debris 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

<u>Background</u>

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 ____________________

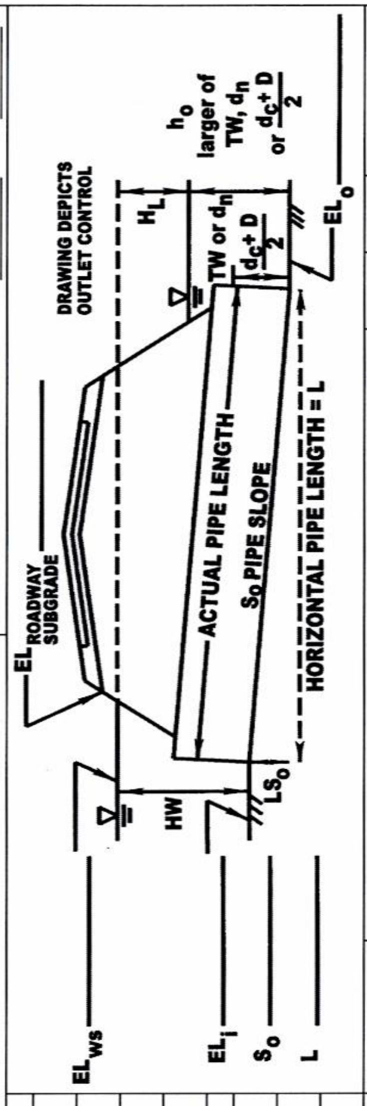
STATION ____________________
DESIGNER __________ DATE __________
REVIEWER __________ DATE __________

OUTLET CHANNEL TAILWATER

	RETURN PERIOD	TW OR NORMAL DEPTH
DESIGN STORM		
CHECK STORM		

CULVERT FLOWS

	RETURN PERIOD	FLOW
DESIGN STORM		
CHECK STORM		

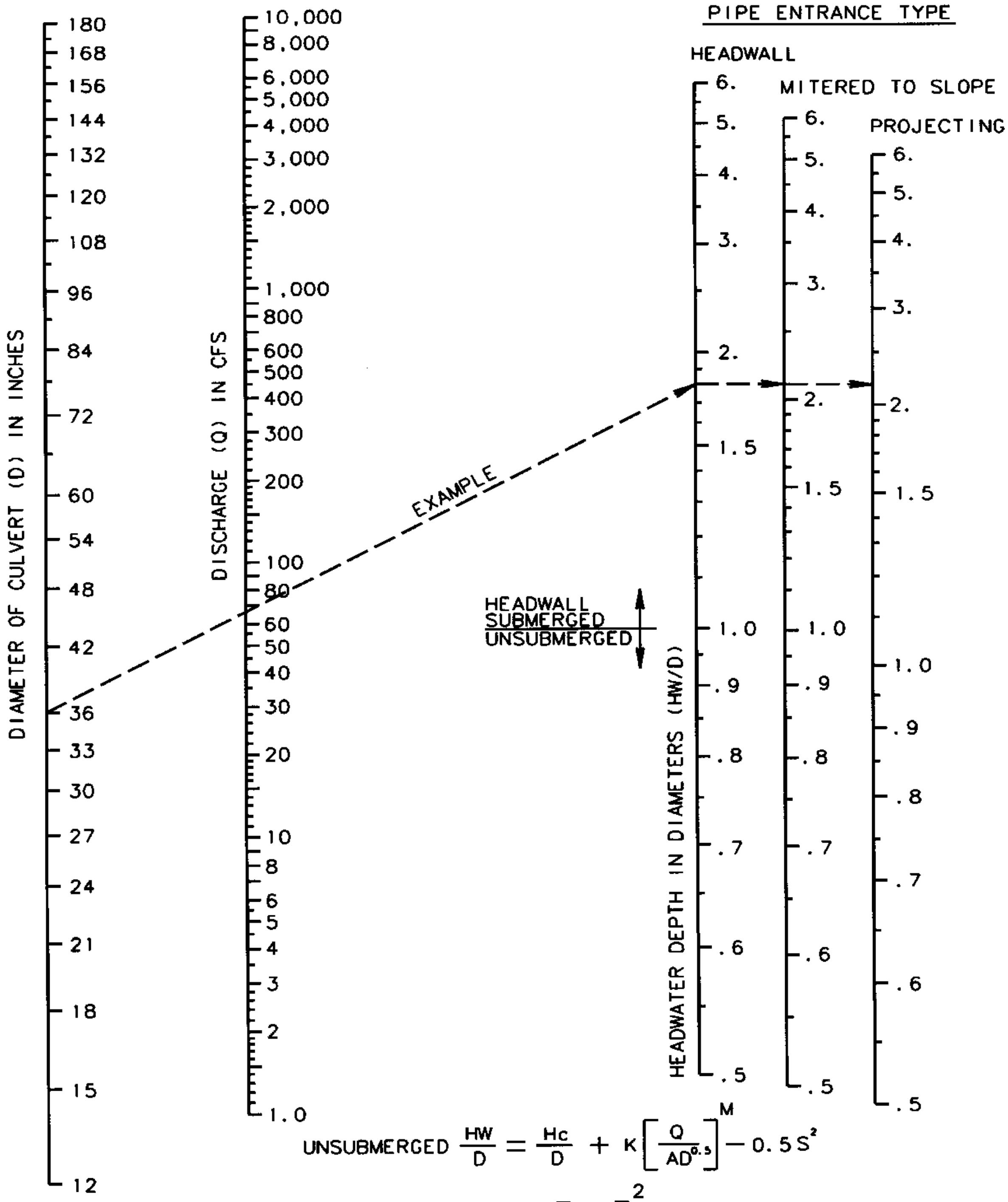


CULVERT DESCRIPTION: MATERIAL, ENTRANCE, ROUGHNESS	CULVERT DIAMETER OR SHAPE	TOTAL FLOW	MAX ALLOWABLE HEADWATER	INLET CONTROL: HEADWATER TO DEPH RATIO	INLET CONTROL: HW AT INLET DUE TO INLET CONTROL	OUTLET CONTROL: TAILWATER OR NORMAL DEPTH IN CHANNEL	OUTLET CONTROL: CRITICAL DEPTH IN PIPE	OUTLET CONTROL: PIPE CRITICAL DEPTH & DIAMETER AVERAGE	OUTLET CONTROL: WSEL AT THE PIPE OUTLET	OUTLET CONTROL: ENTRANCE LOSS COEFFICIENT	OUTLET CONTROL: HEAD LOSS DUE TO FLOW THROUGH THE CULVERT	OUTLET CONTROL: FALL THROUGH THE CULVERT	OUTLET CONTROL: 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	LS_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 $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^{2}$

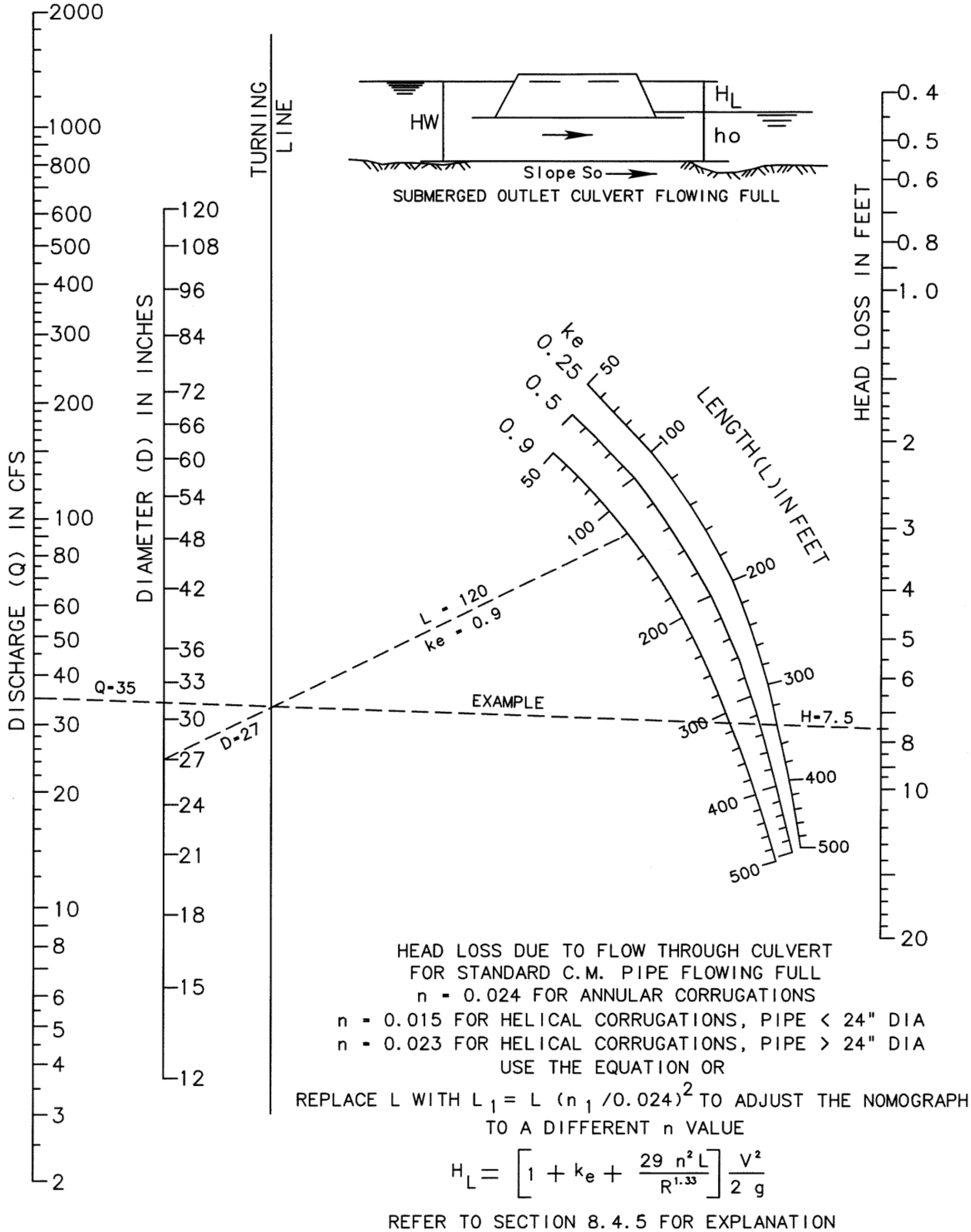
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^{2}$

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

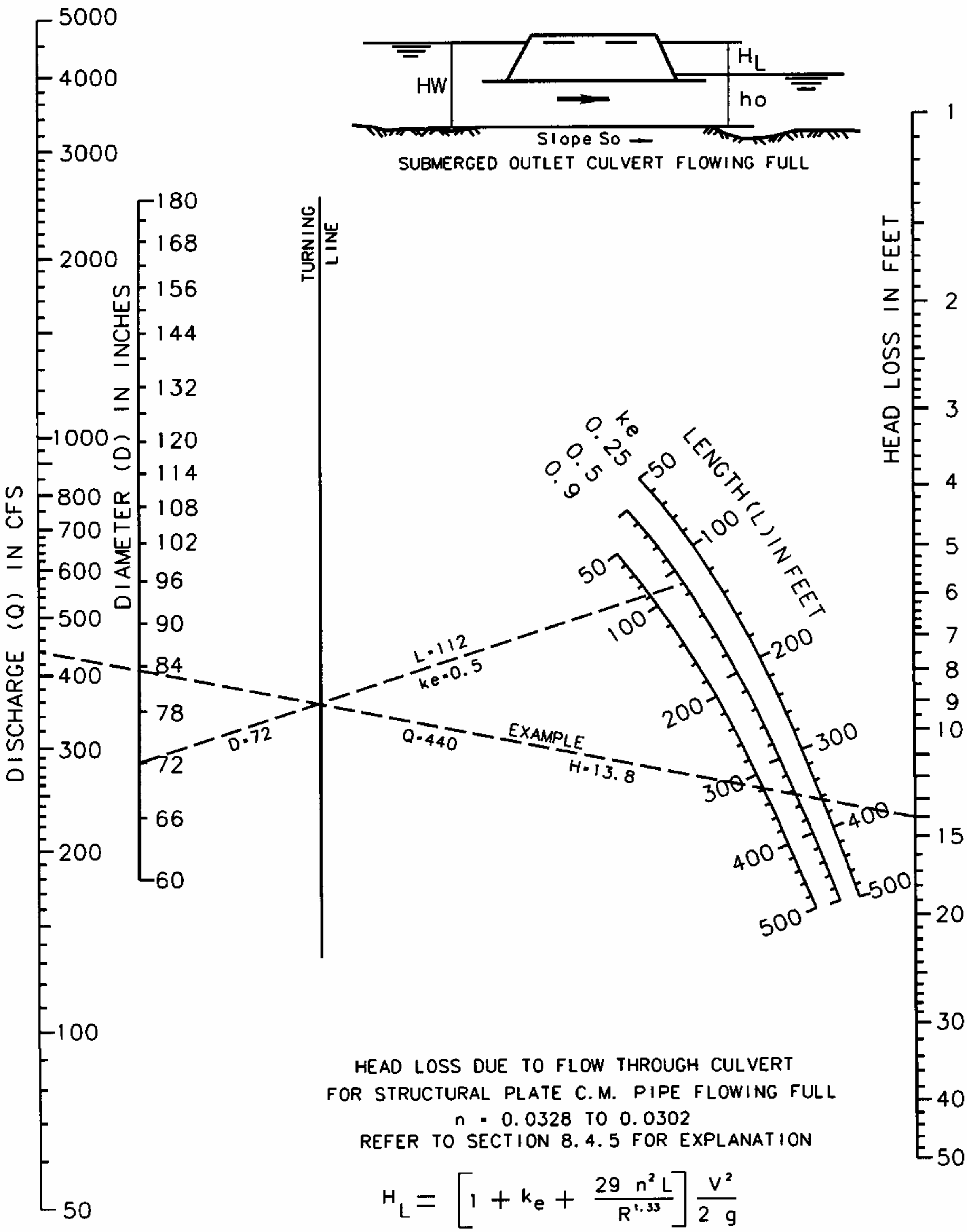
Standard C.M. Pipe Flowing Full



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

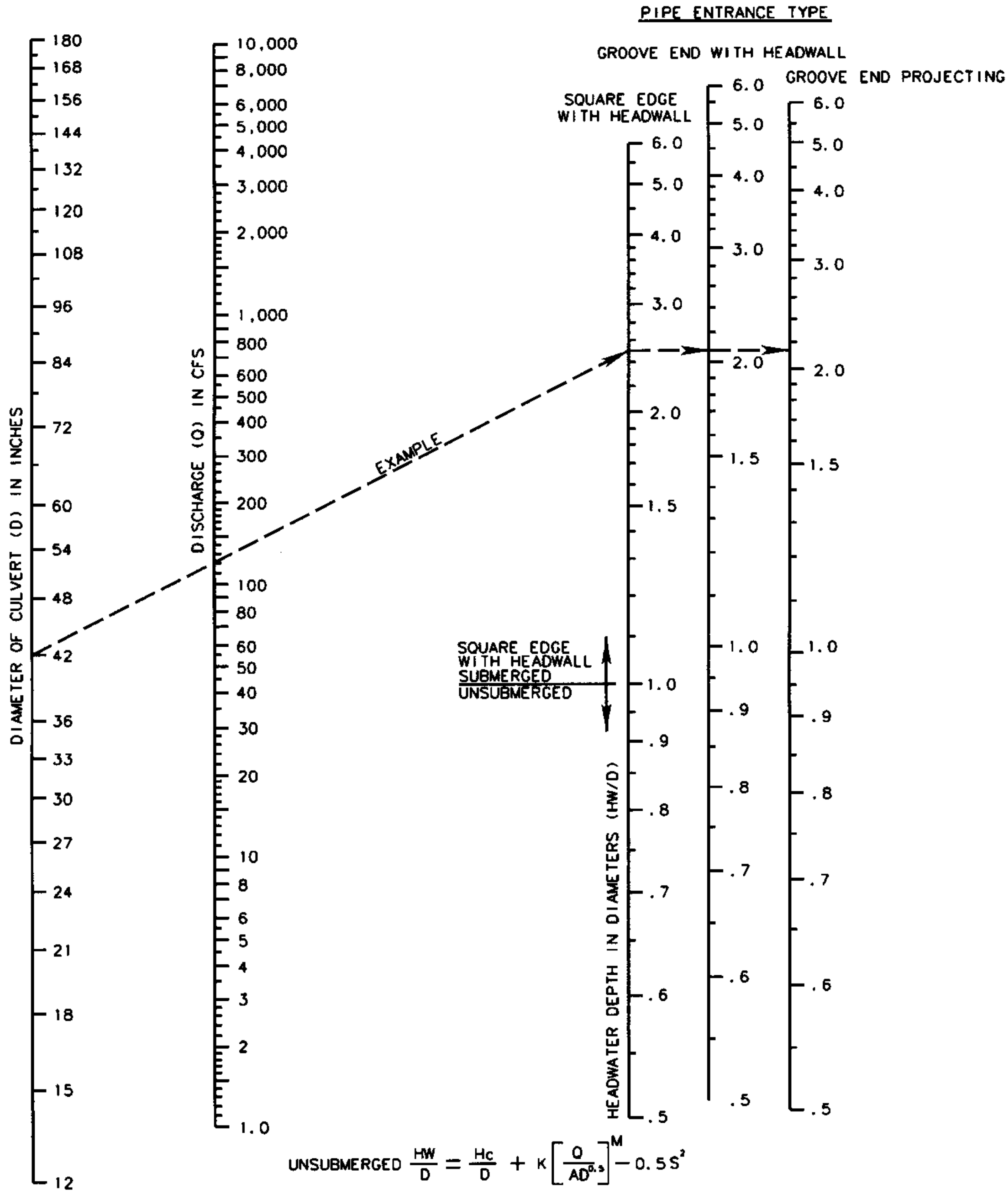
Chart 8-3

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 $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^{2}$

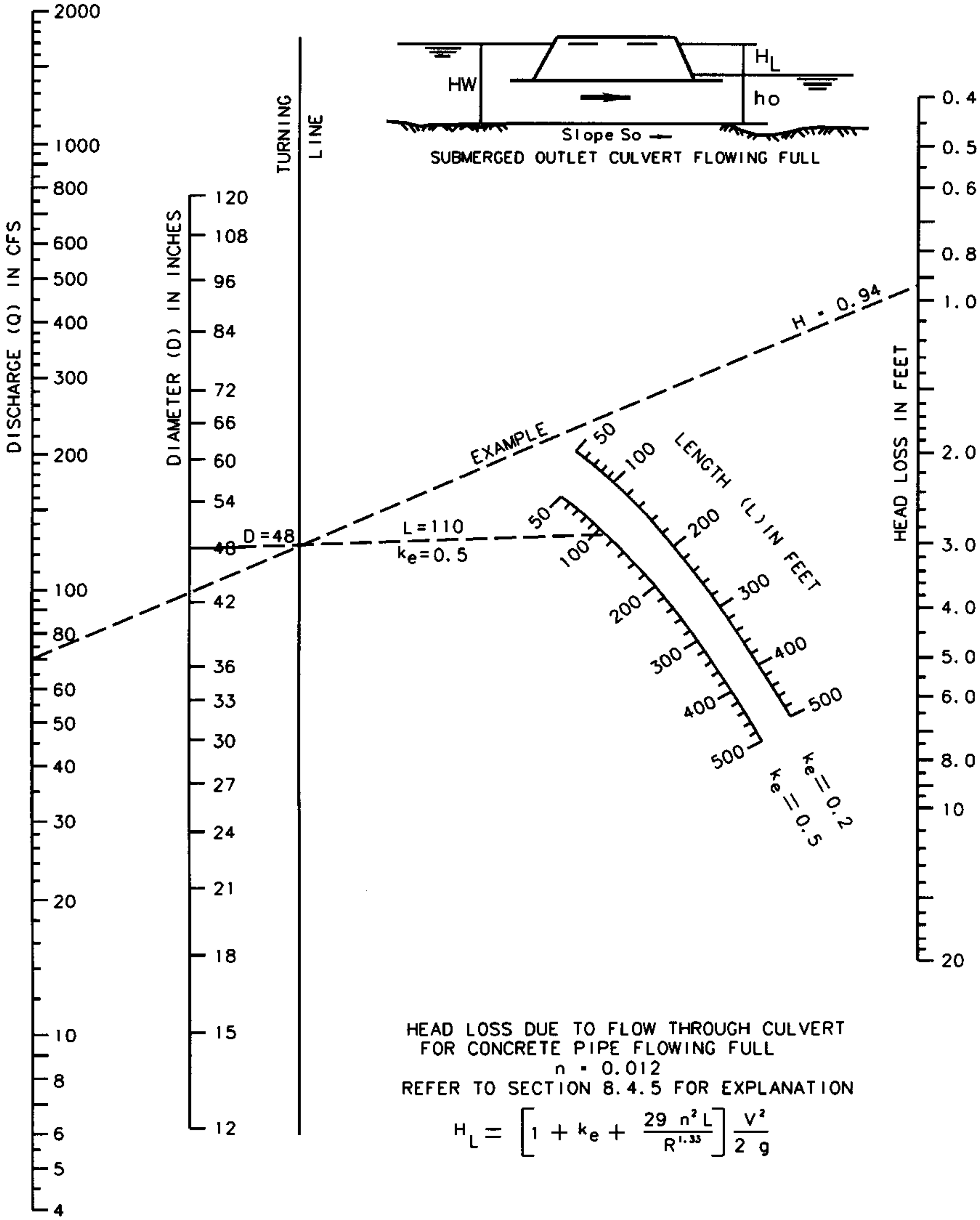
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^{2}$

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

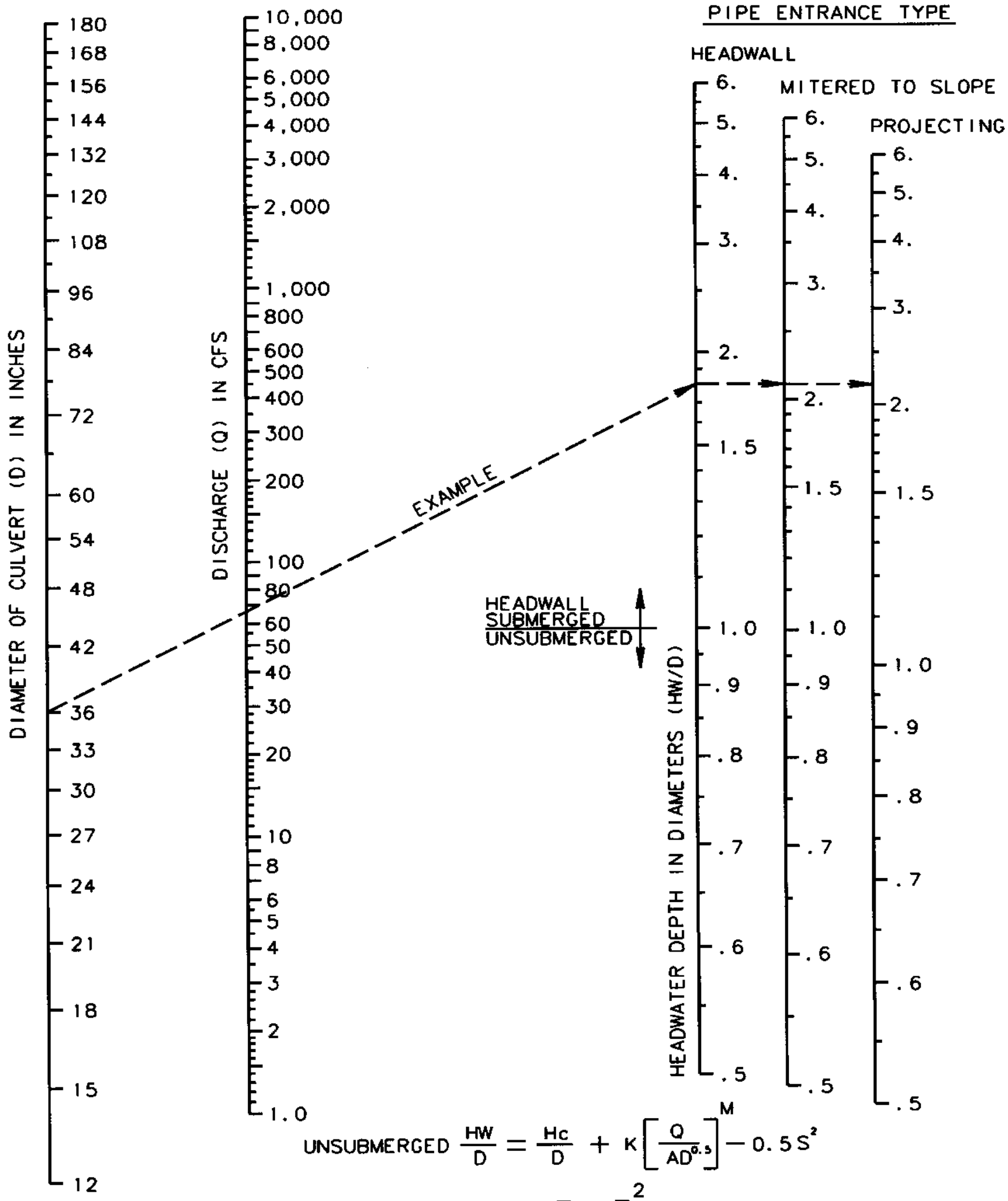
Concrete Pipe Flowing Full



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-6

HDPEP Pipe Culverts with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^{2}$

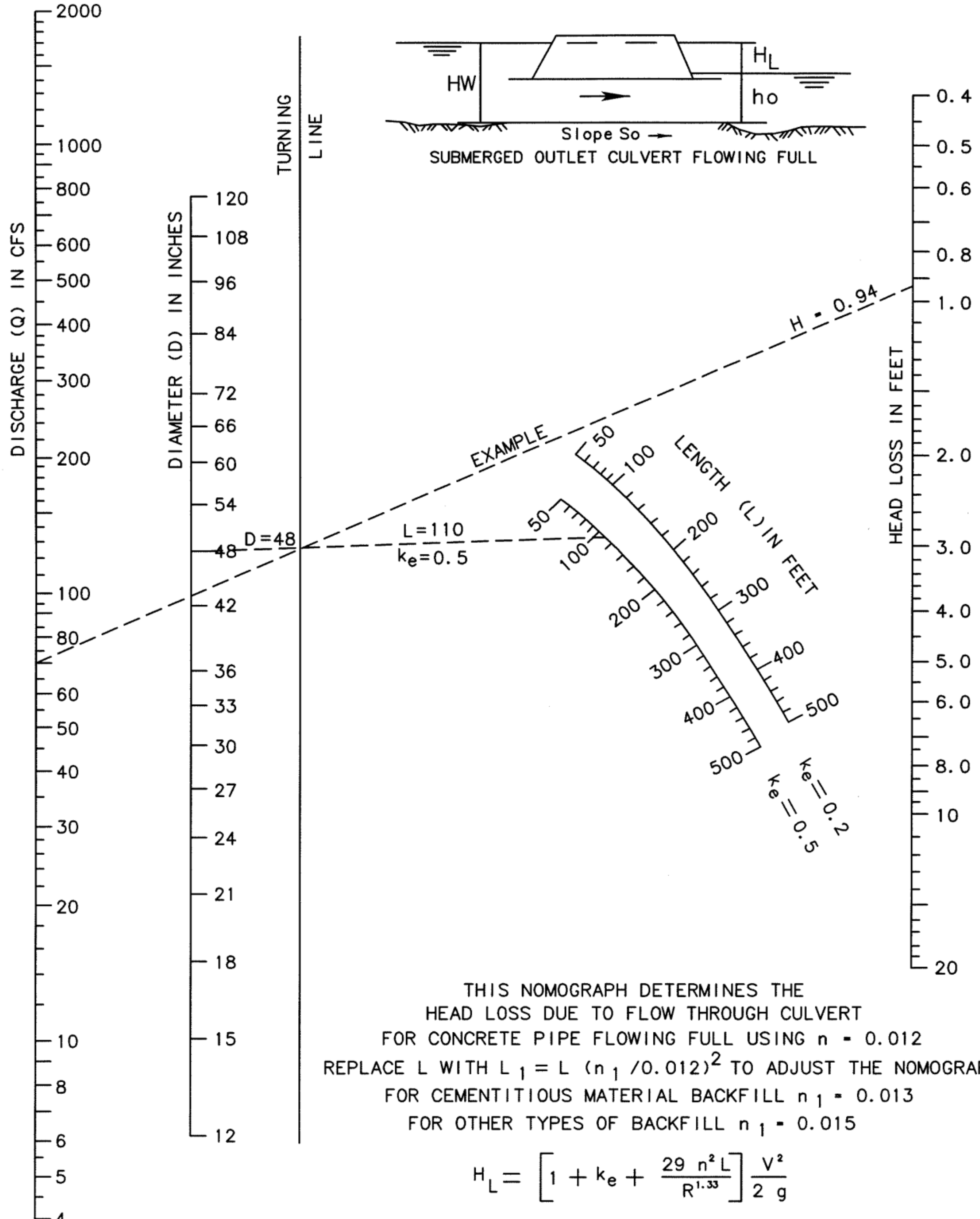
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^{2}$

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

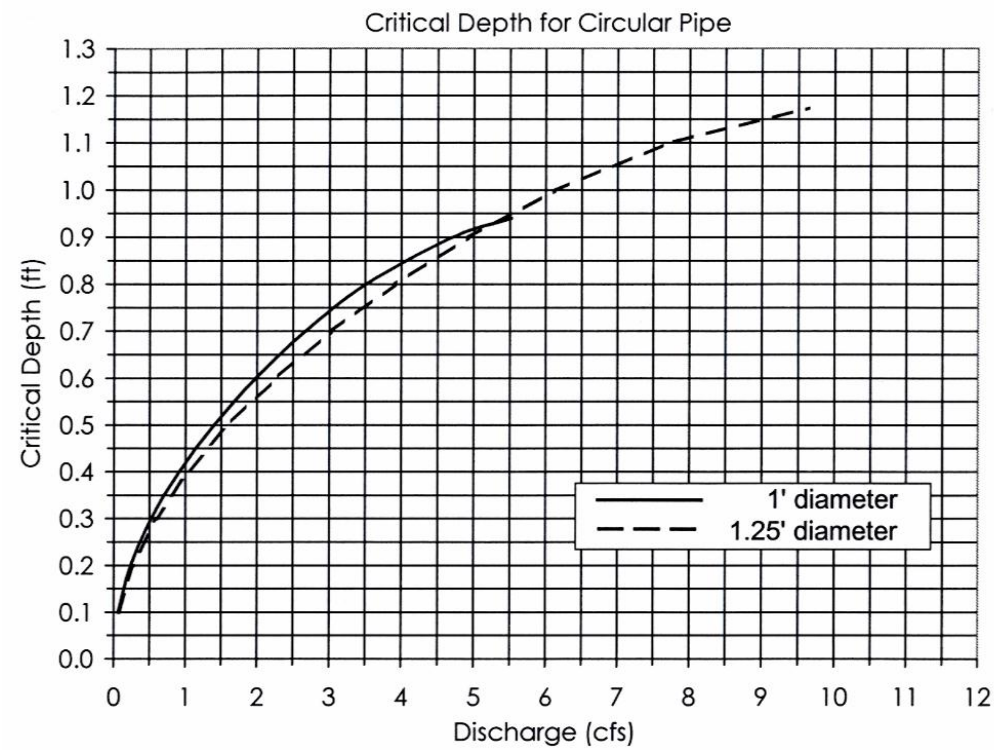
$$H_L = \left[1 + k_e + \frac{29\, n^2 L}{R^{1.33}}\right]\frac{V^2}{2g}$$

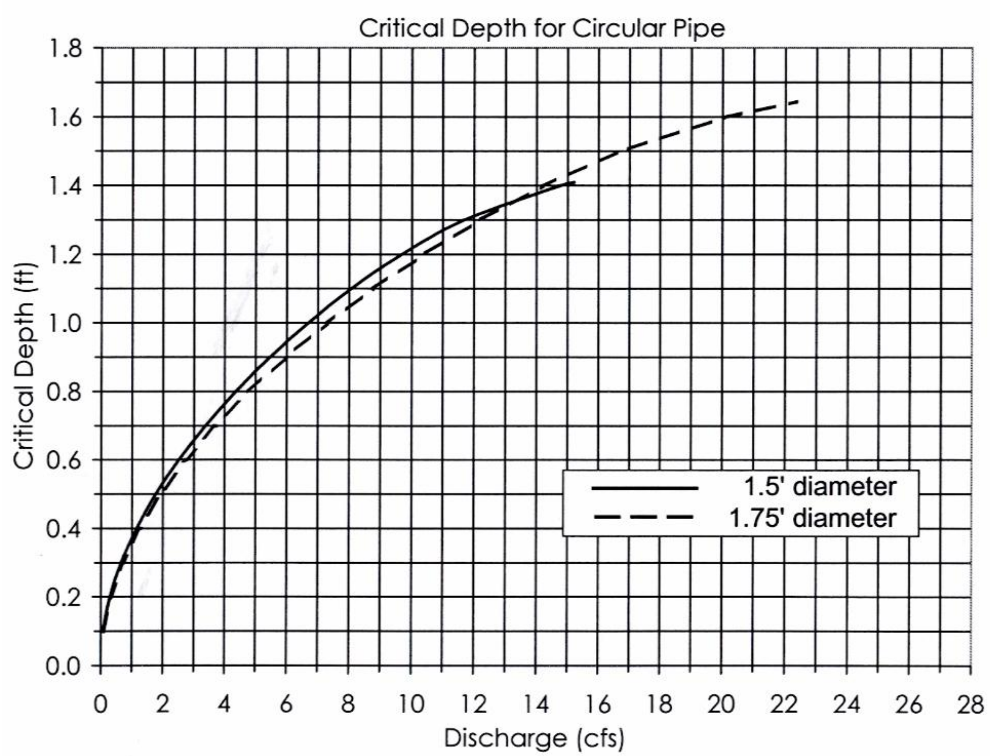
REFER TO SECTION 8.4.5.2 FOR EXPLANATION

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-8

Critical Depth for Circular Pipe

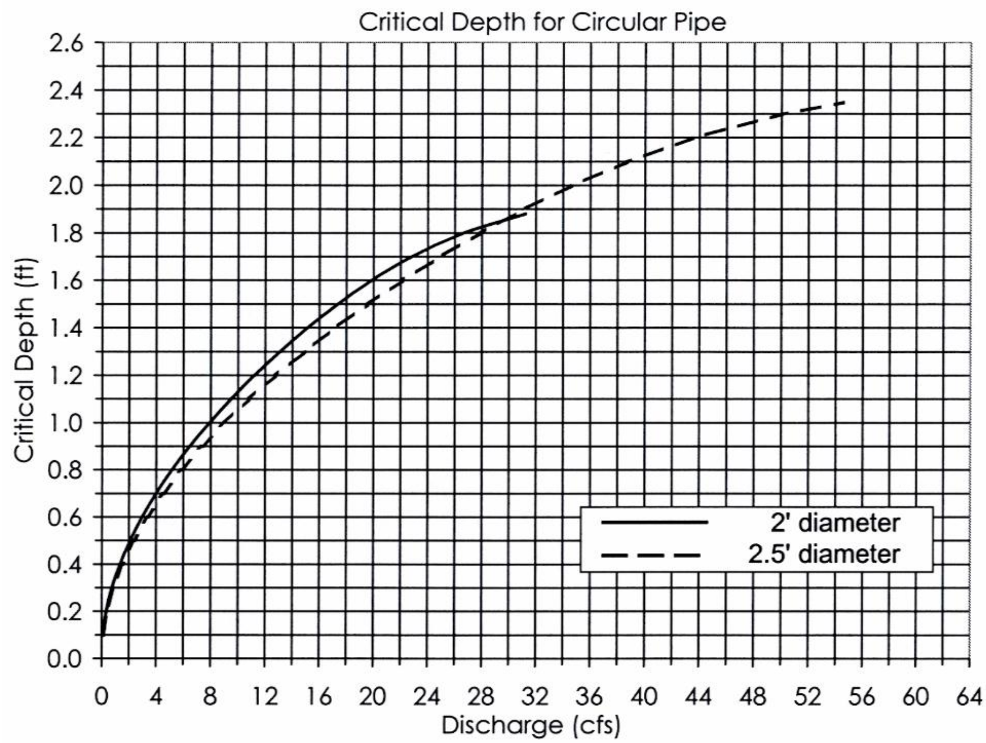


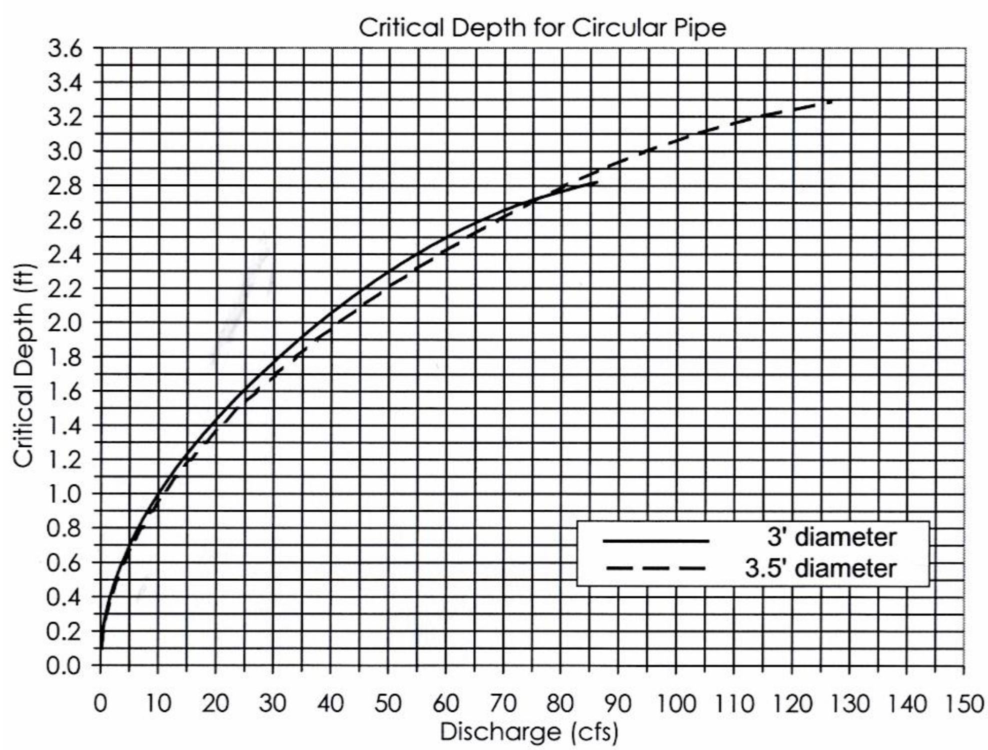


Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-9

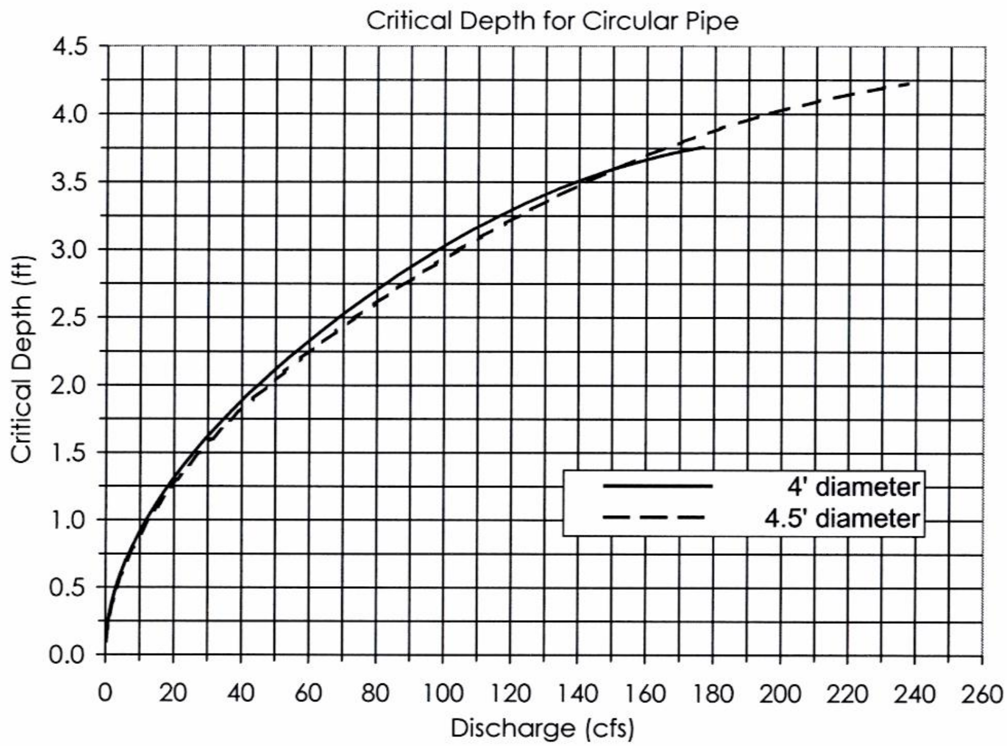
Critical Depth for Circular Pipe

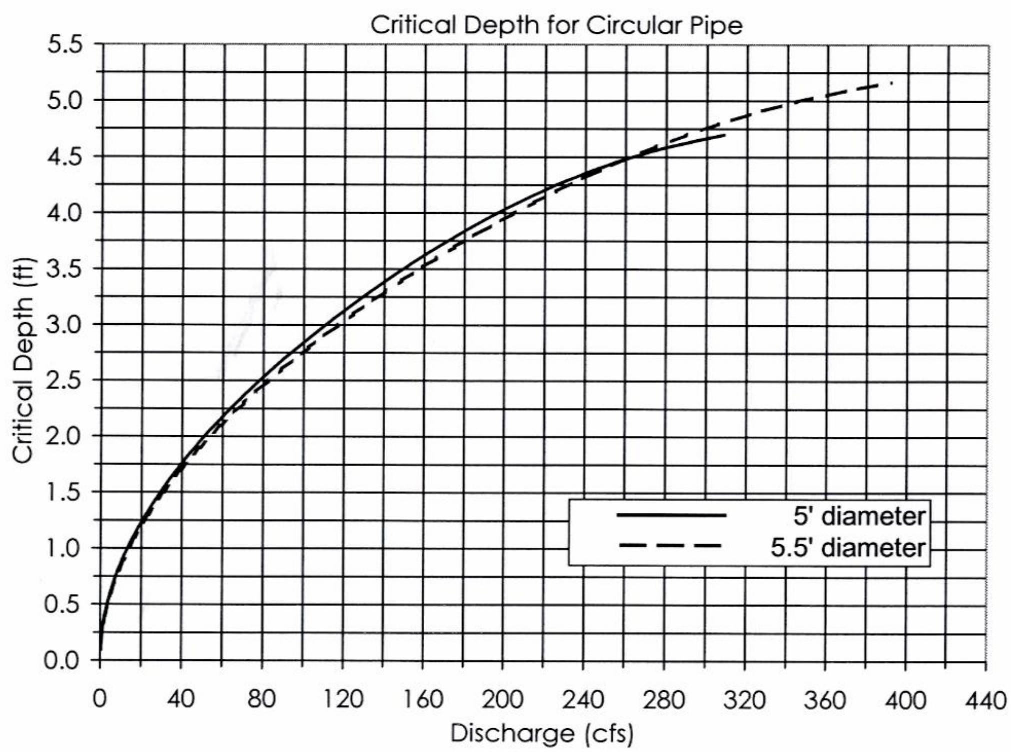




Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-10
Critical Depth for Circular Pipe

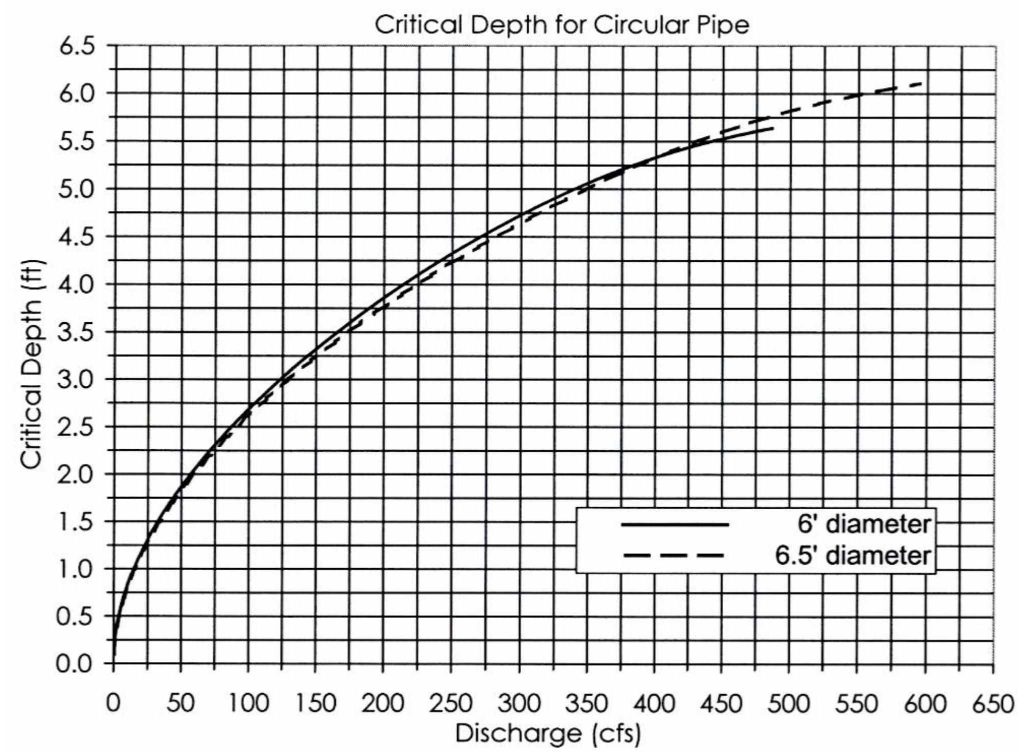


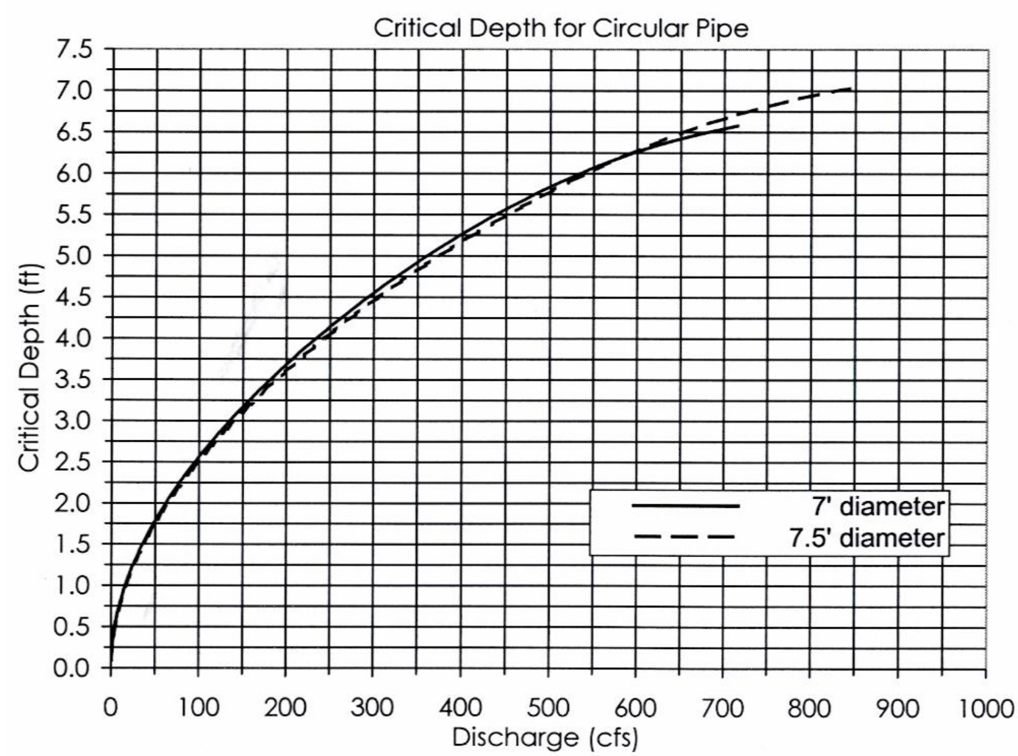


Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-11

Critical Depth for Circular Pipe

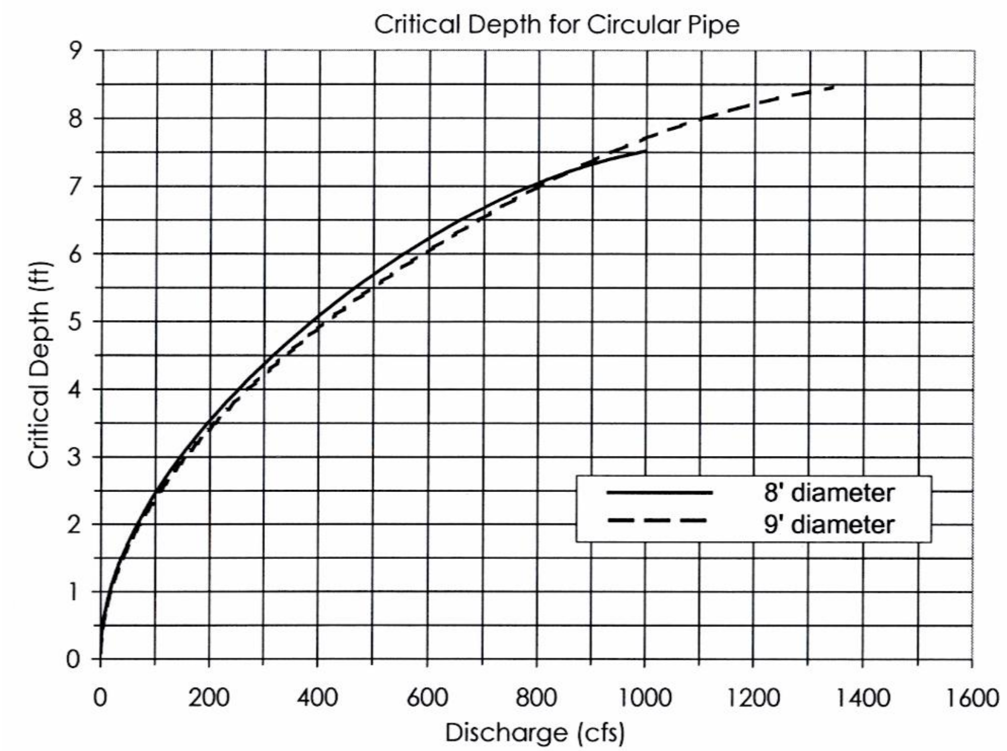


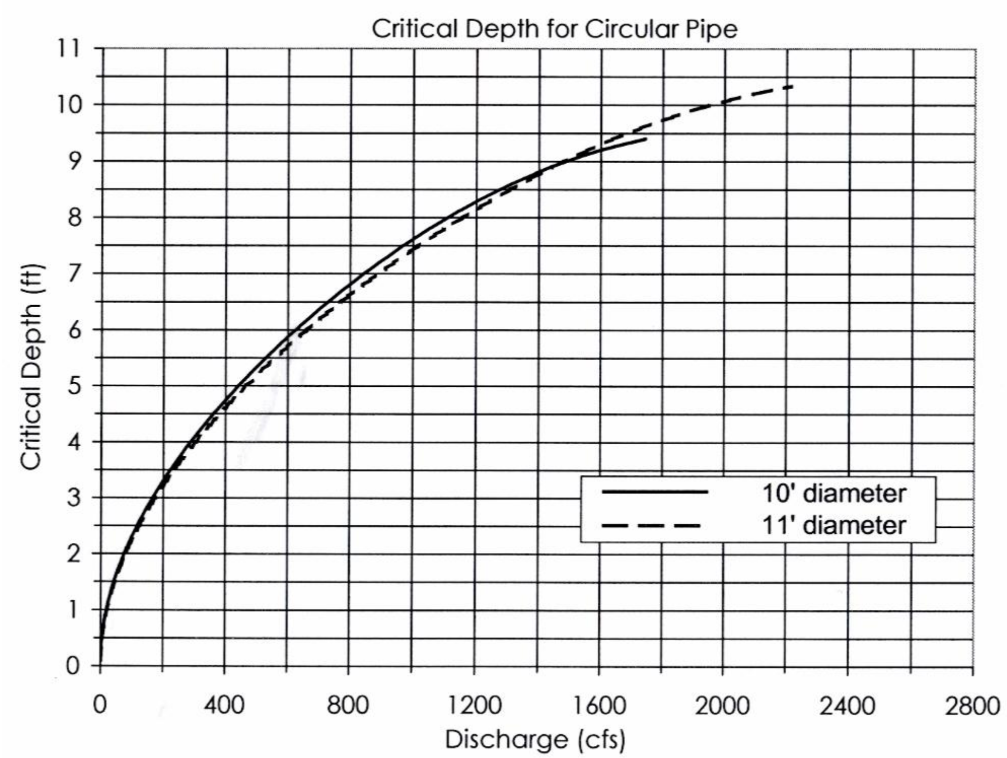


Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-12

Critical Depth for Circular Pipe

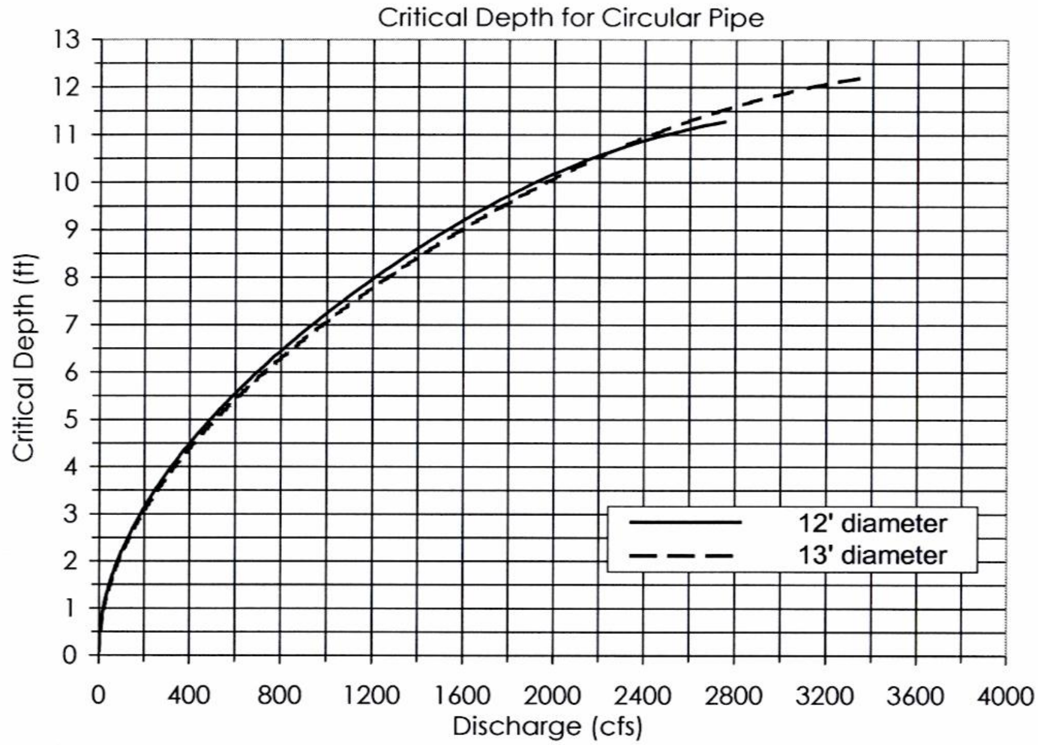


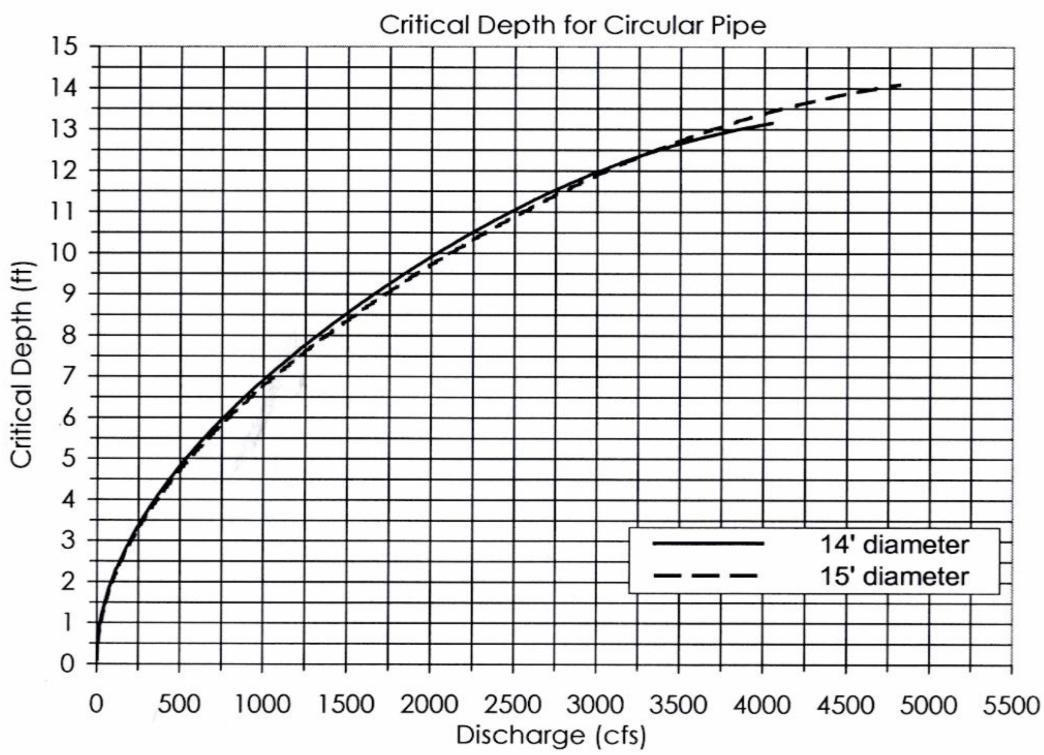


Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-13

Critical Depth for Circular Pipe

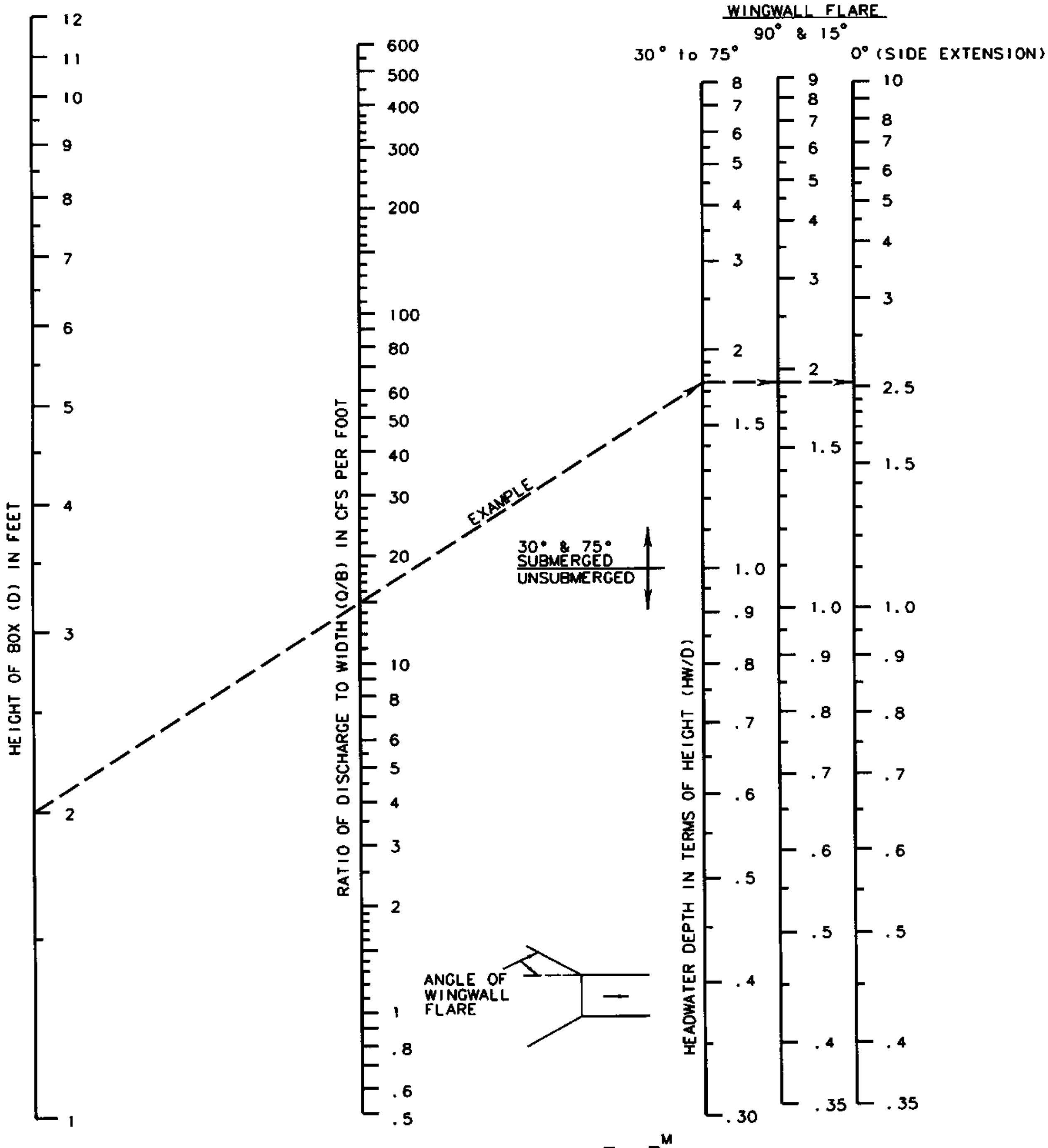




Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-14

Box Culverts with Inlet Control



UNSUBMERGED $\frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^M - 0.5S^2$

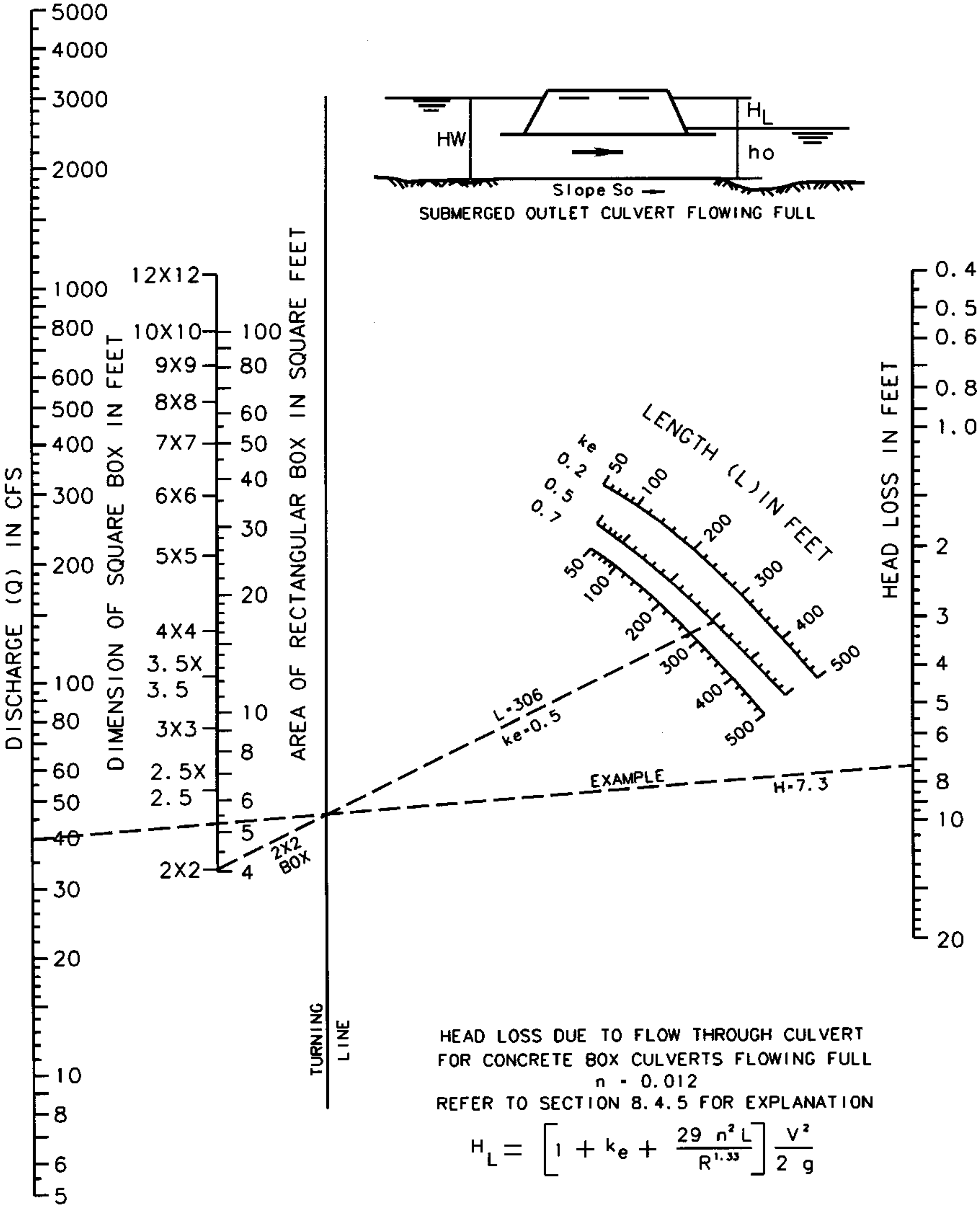
SUBMERGED $\frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S^2$

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

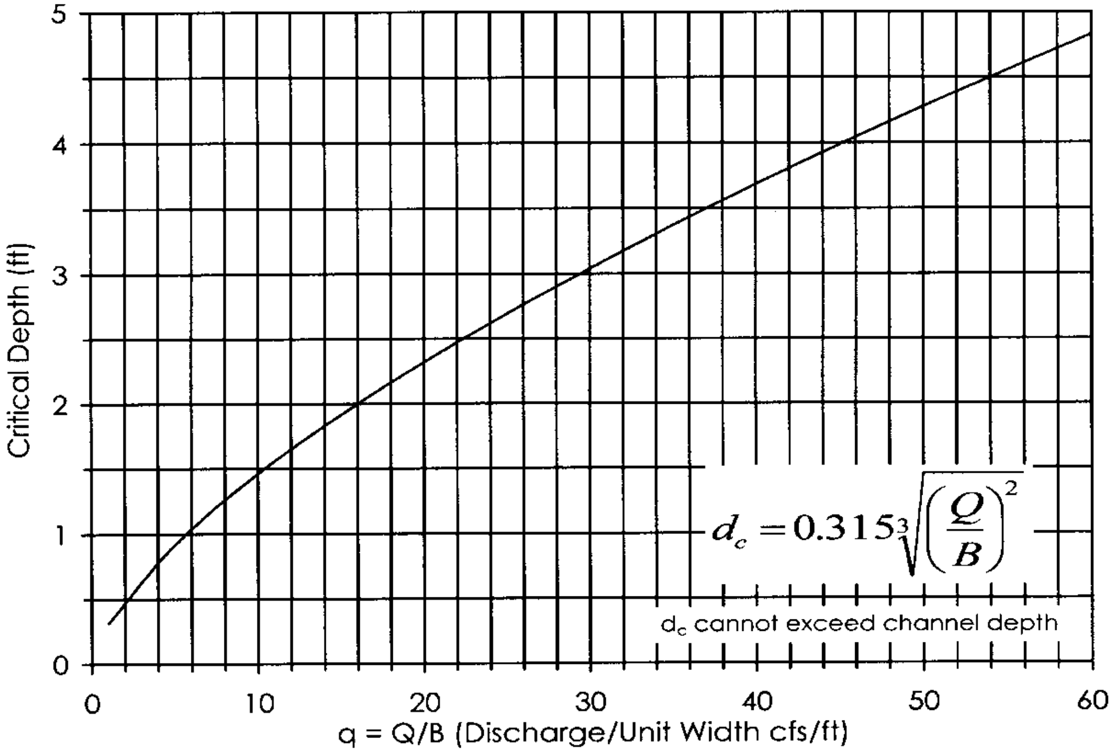


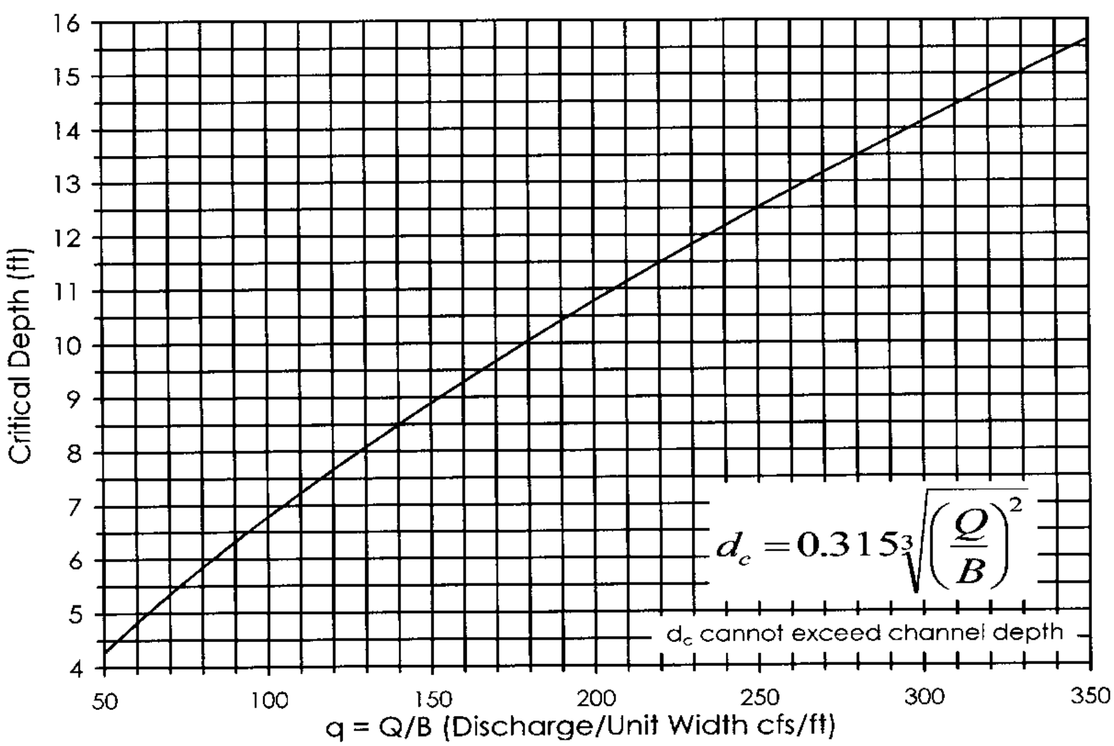
$$H_L = \left[1 + k_e + \frac{29\ n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-16

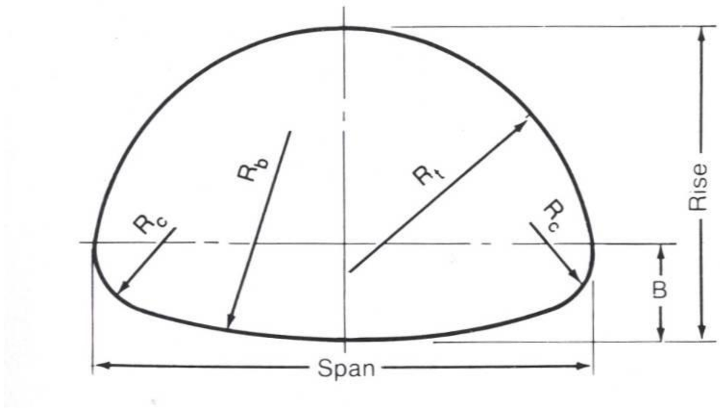
Critical Depth for Box Culverts





Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-17
Standard Arch Pipe Details

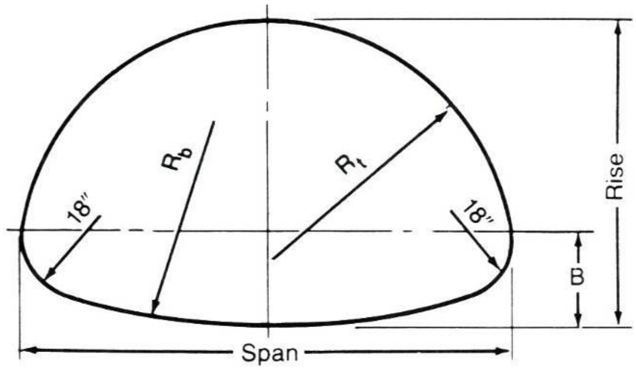


Corrugated Steel Pipe Arch												
NOMINAL SIZE	Rise			Span			Corrugation	Area	B	R_c	R_t	R_b
	in	ft	ft - in	in	ft	ft - in	in x in	ft_2	in	in	in	in
13 x 17	13	1.08	1 - 1.0	17	1.42	1 - 5.0	2 2/3 x 1/2	1.1	4 1/8	3 1/2	8 5/8	25 5/8
15 x 21	15	1.25	1 - 3.0	21	1.75	1 - 9.0	2 2/3 x 1/2	1.6	4 7/8	4 1/8	10 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

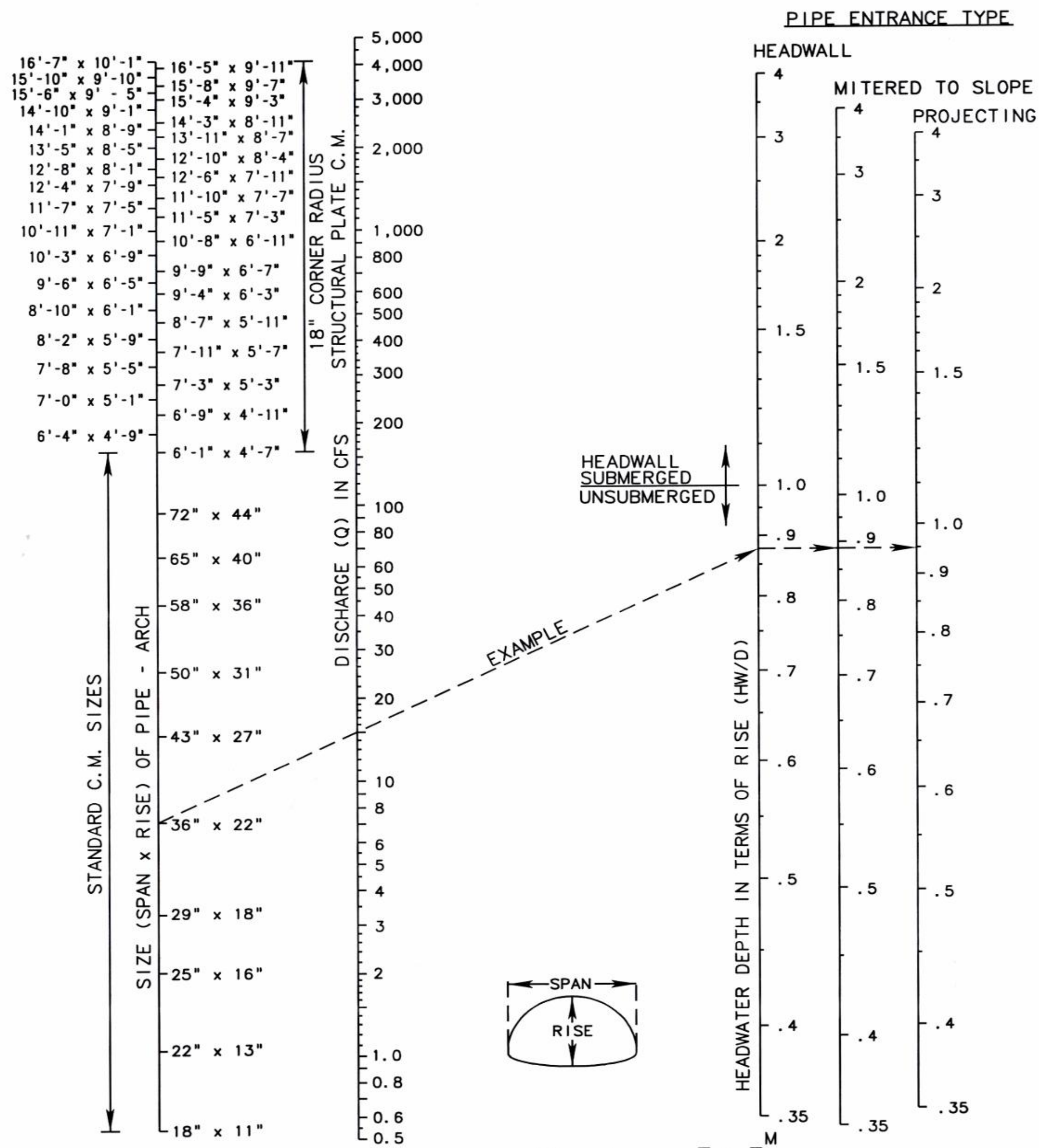


Corrugated Steel Pipe Arch 18 inch corner radius											
Rise			Span			Corrugation	Area	B	R_c	R_t	R_b
in	ft	ft - in	in	ft	ft - in	in x in	ft_2	in	in	ft	ft
55	4.58	4 - 7.0	73	6.08	6 - 1.0	6 x 2	22	21.0	18	3.07	6.36
57	4.75	4 - 9.0	76	6.33	6 - 4.0	6 x 2	24	20.5	18	3.18	8.22
59	4.92	4 - 11.0	81	6.75	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



M

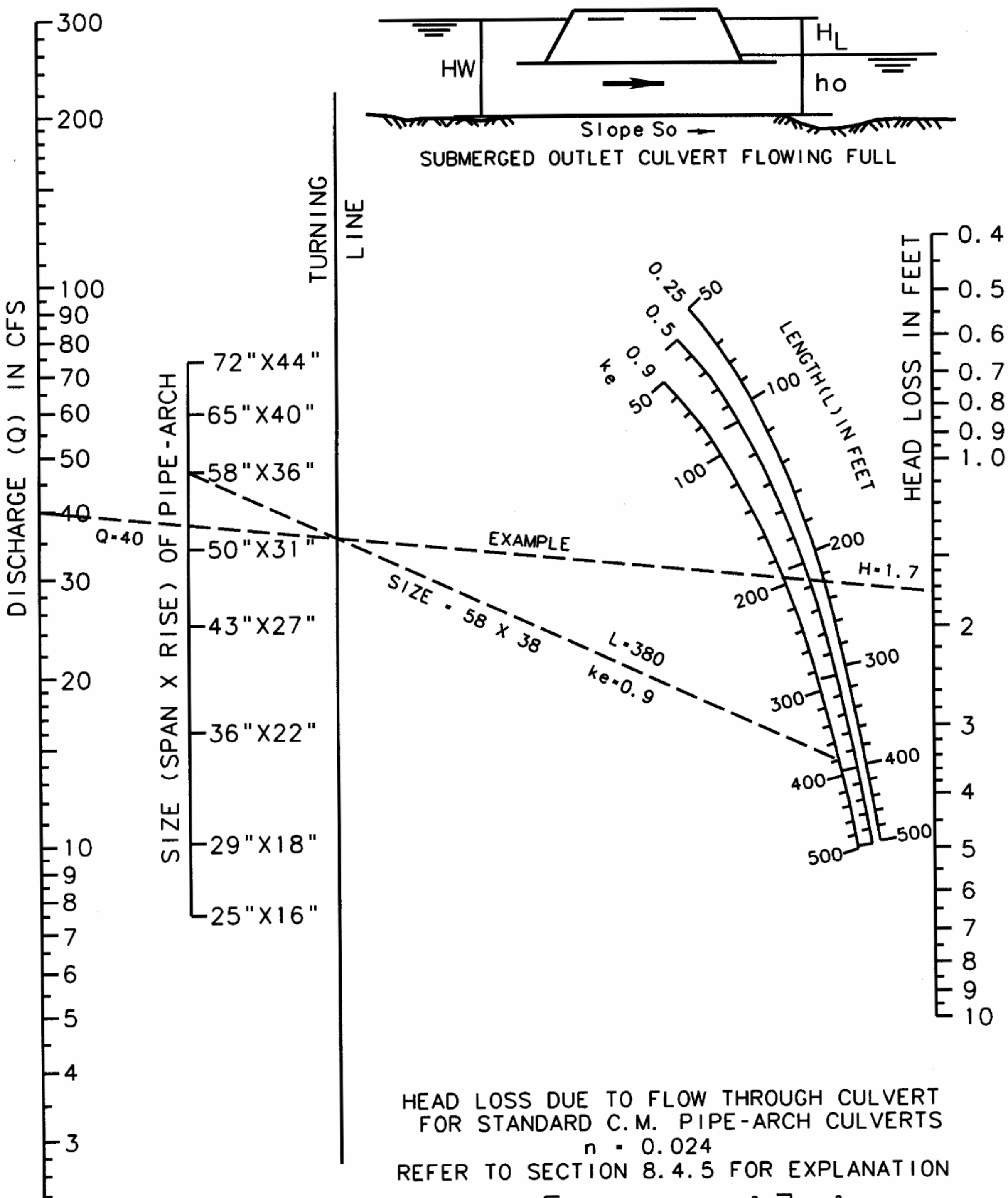
$$\text{UNSUBMERGED} \quad \frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5\,S^2$$

$$\text{SUBMERGED} \quad \frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5\,S^2$$

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

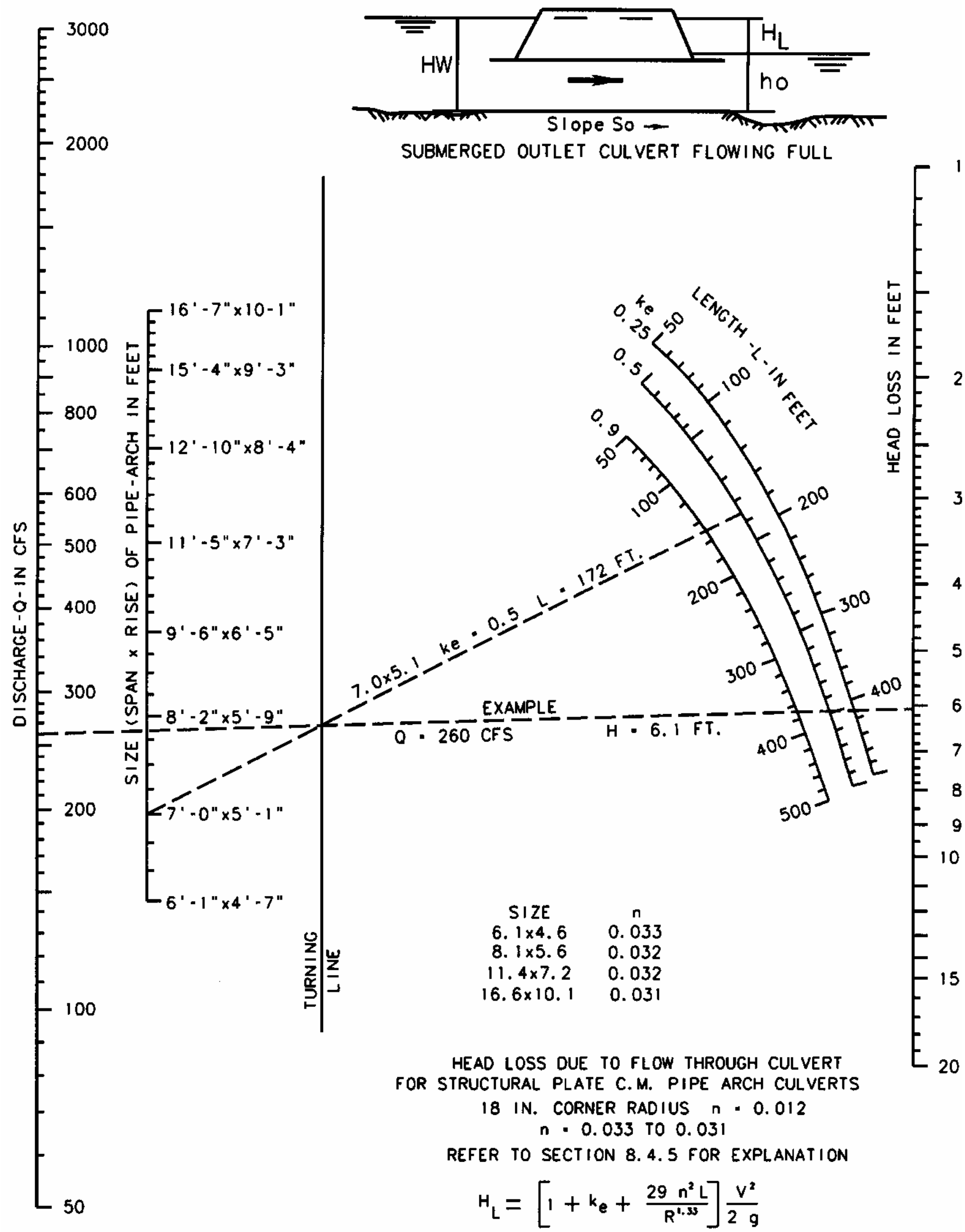


$$H_L = \left[1 + k_e + \frac{29\ n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-21

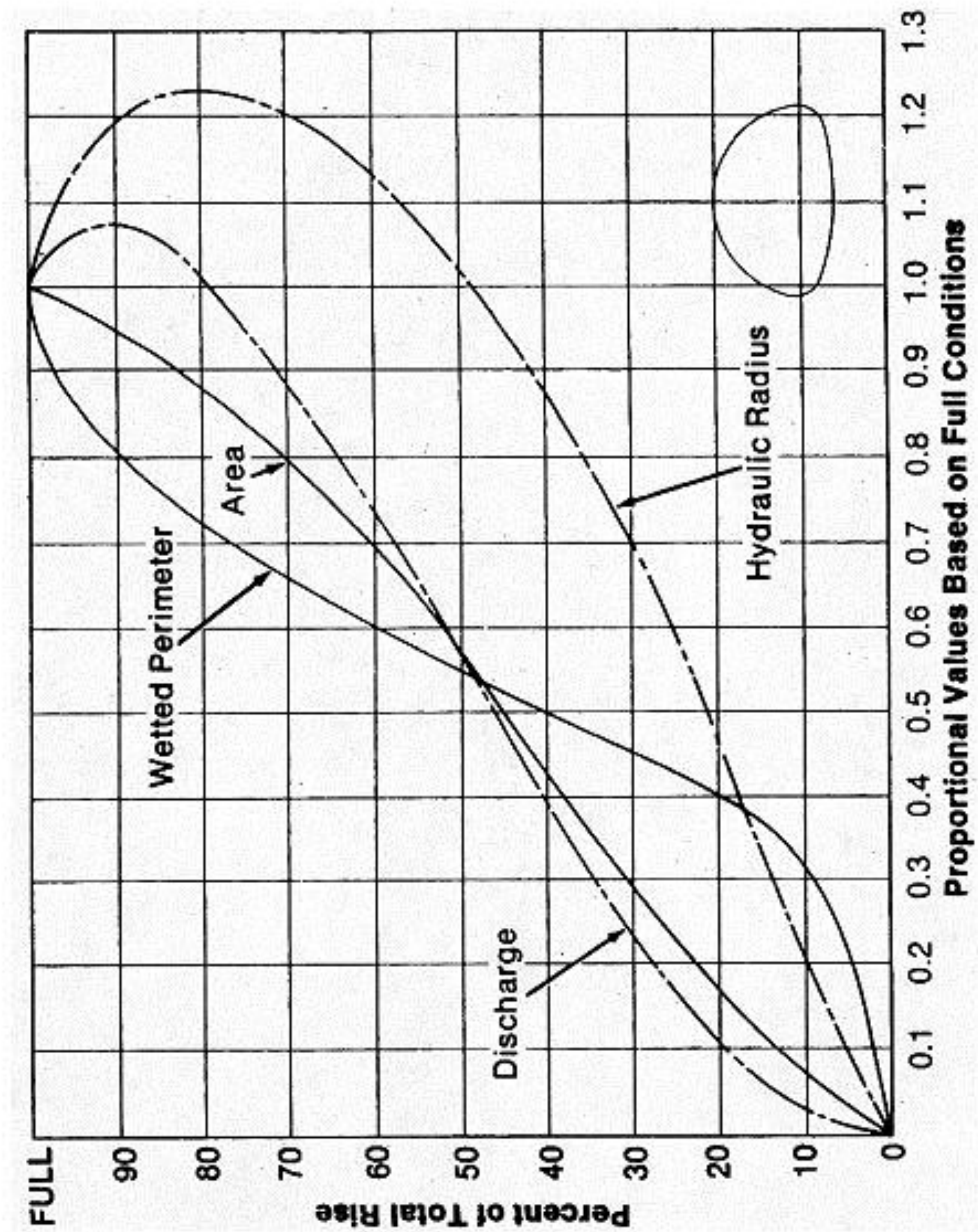
Str. Plate Pipe Arch (18" corner rad.) Flowing Full



Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-22

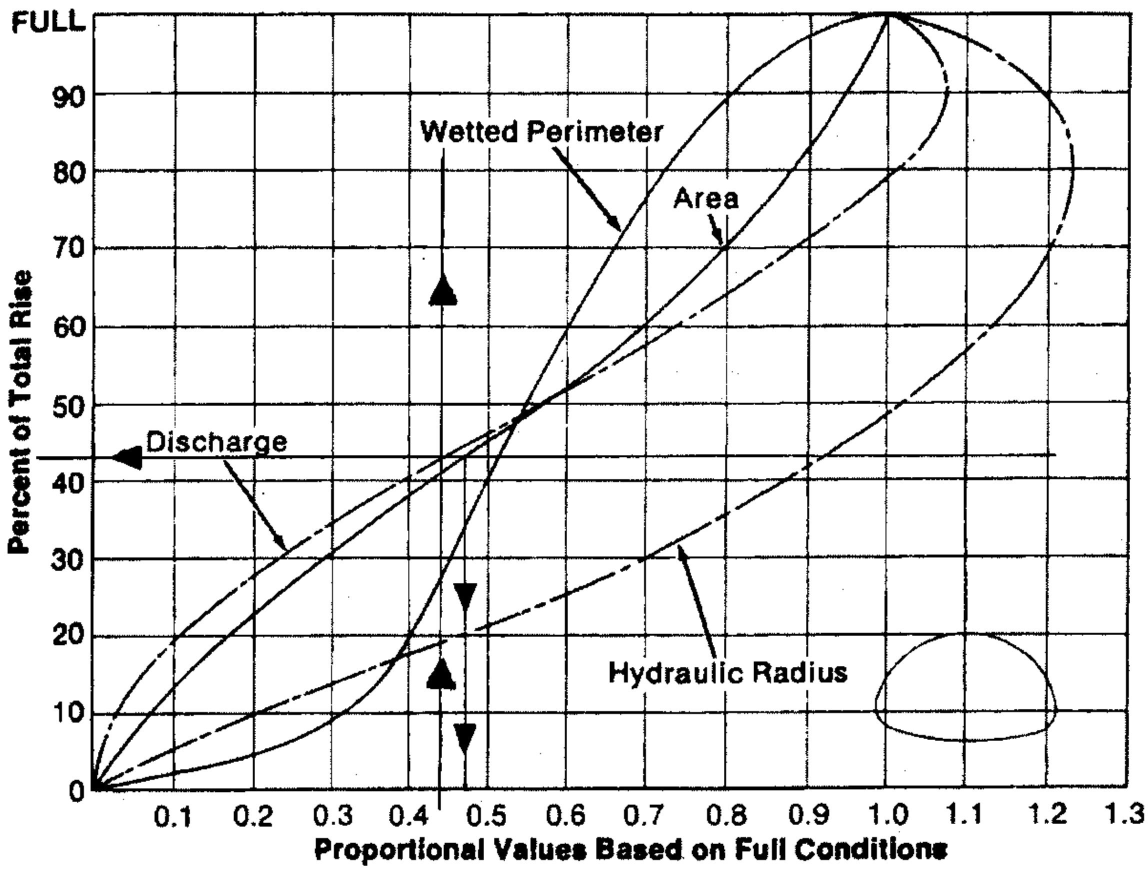
Hydraulic Elements for Partially Full C.M. Pipe Arch



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, $d / d_{full} = 0.435$

$d = d_{full} \times 0.435 = 10.4$ in

From figure for hydraulic elements, $A / A_{full} = 0.465$ (A_{full} from Table 8-5)

$A = A_{full} \times 0.465 = 2.1\ ft^2$

V = Q / A = 12 / 2.1 = 5.7 ft/s

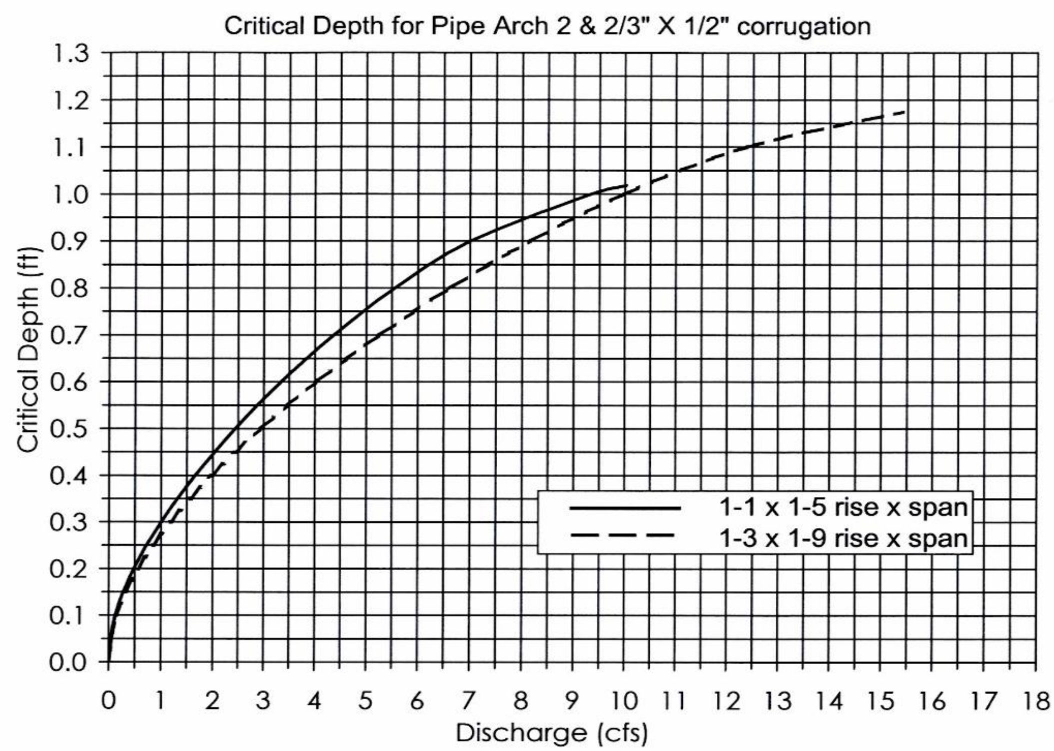
Table 8-5
Full Flow Values for C.M. Pipe Arch

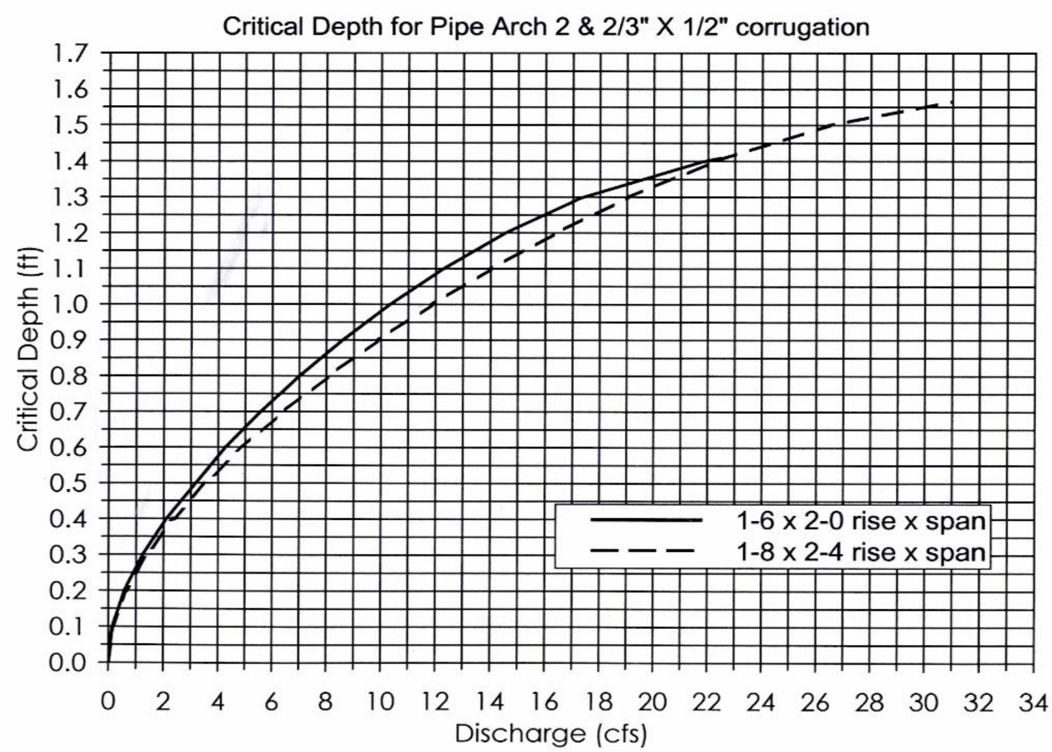
Pipe Size Rise X Span	Corrugations	Equivalent Full Flow Area for Circular Diameter inches	A Area ft ^{2}	R Hydraulic Radius ft	Value of Full Flow Conveyance K $\frac{Q}{\sqrt{S}} = \frac{1.486}{n} A R^{\frac{2}{3}}$ n = 0.024	n = 0.027	n = 0.033
13" X 17"	2 2/3 X 1/2	15	1.1	0.280	29	26	21
15" X 21"	2 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

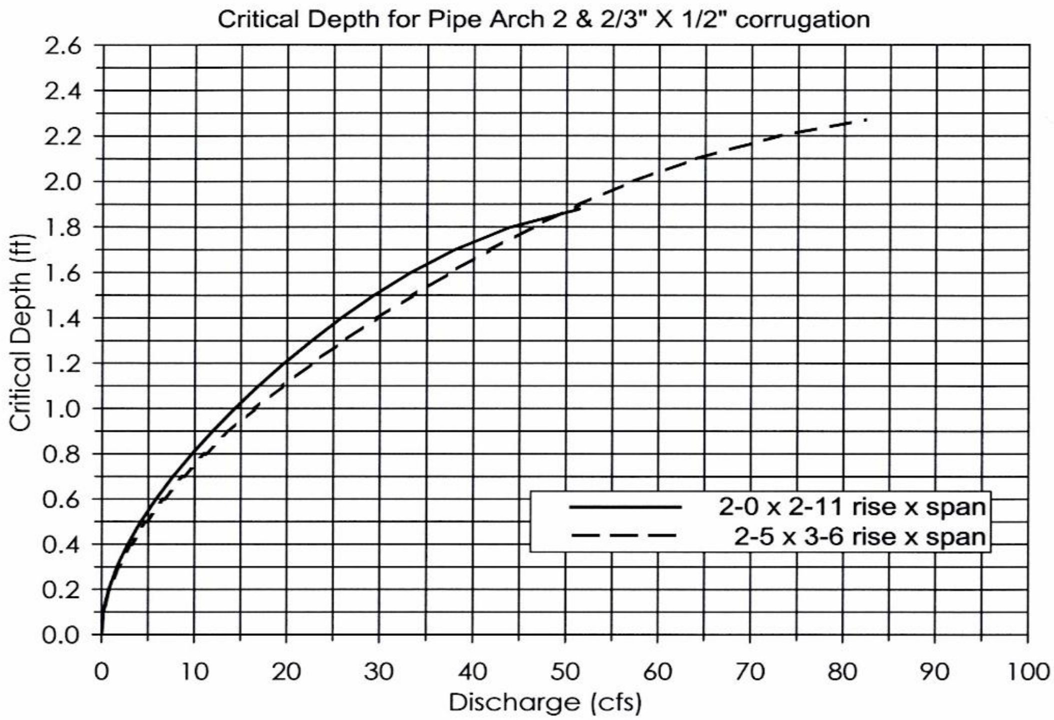


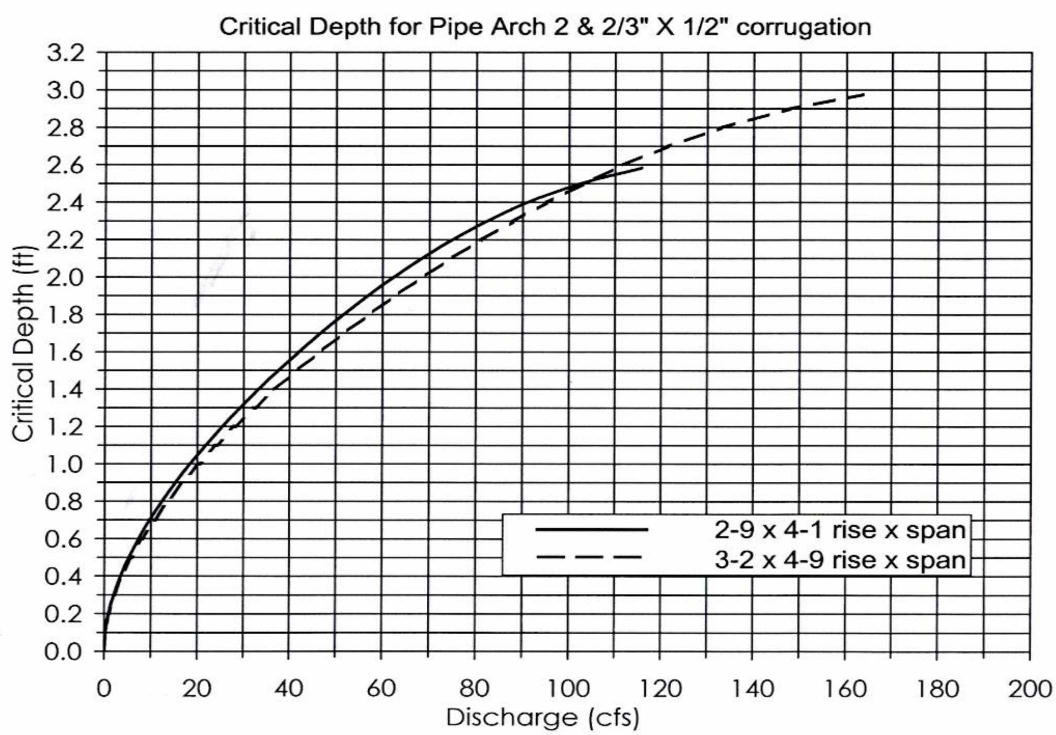


Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-25

Critical Depth for Pipe Arch 2-2/3 x 1/2 in. Corrugation





Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-26

Critical Depth for Pipe Arch 2-2/3 x 1/2 in. Corrugation

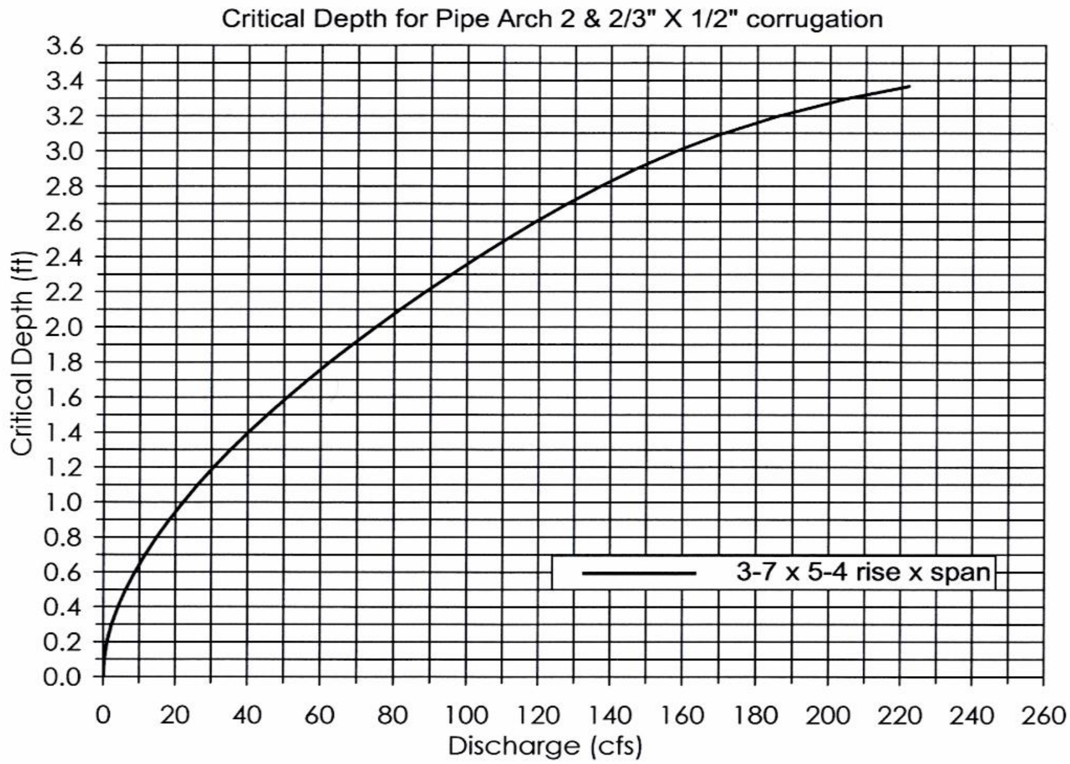


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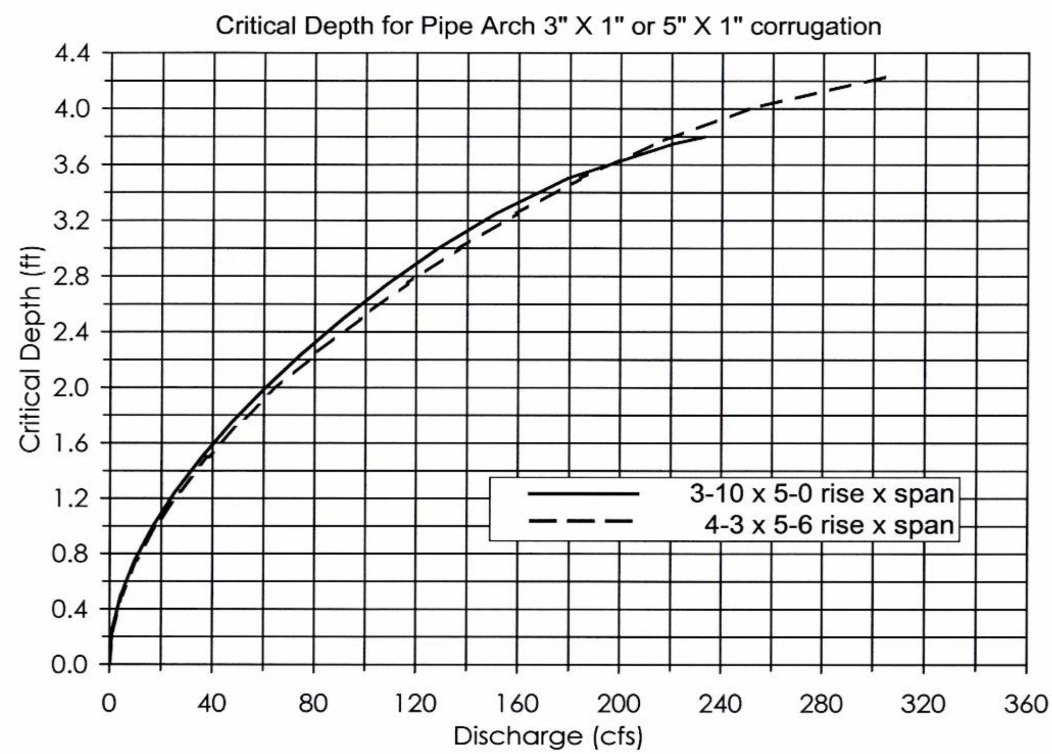
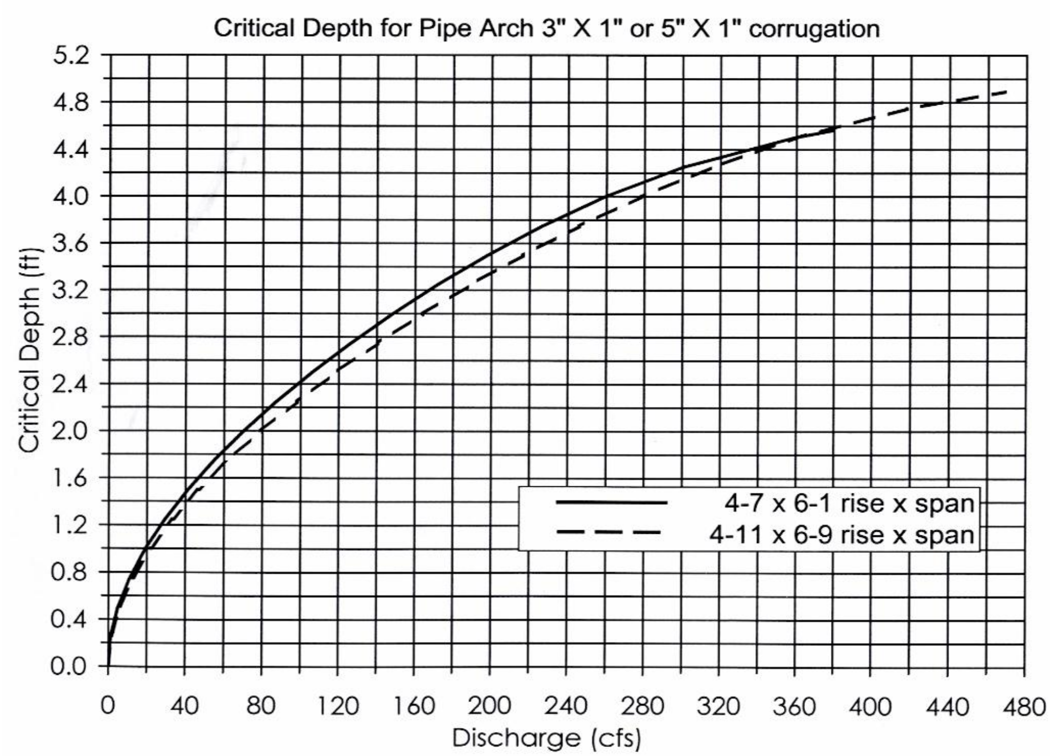
Chart 8-27

Critical Depth for Pipe Arch 3 x 1 or 5 x 1 in. Corrugation

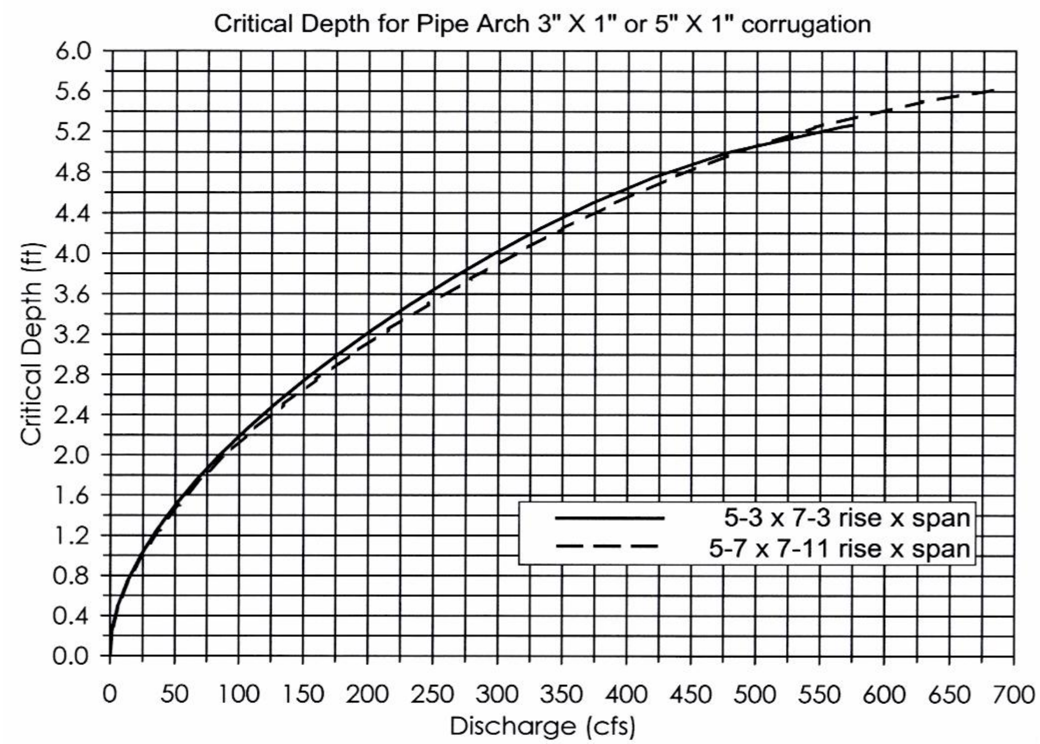




Created by the WVDOH Hydraulic and Drainage Unit

Chart 8-28

Critical Depth for Pipe Arch 3 x 1 or 5 x 1 in. Corrugation



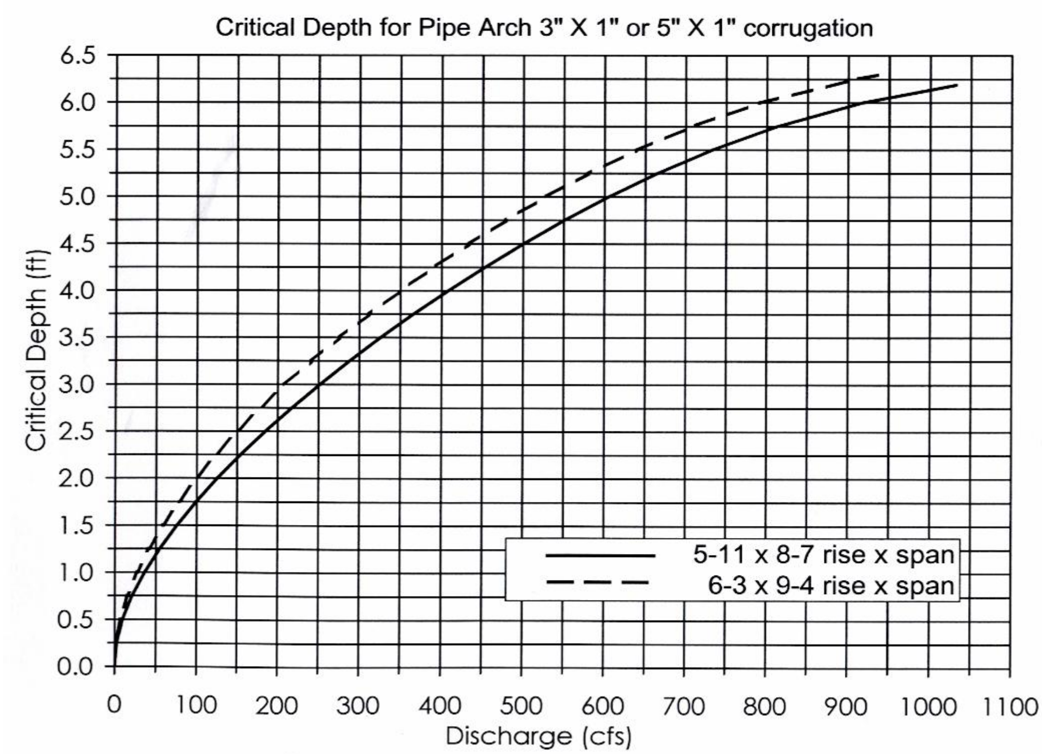


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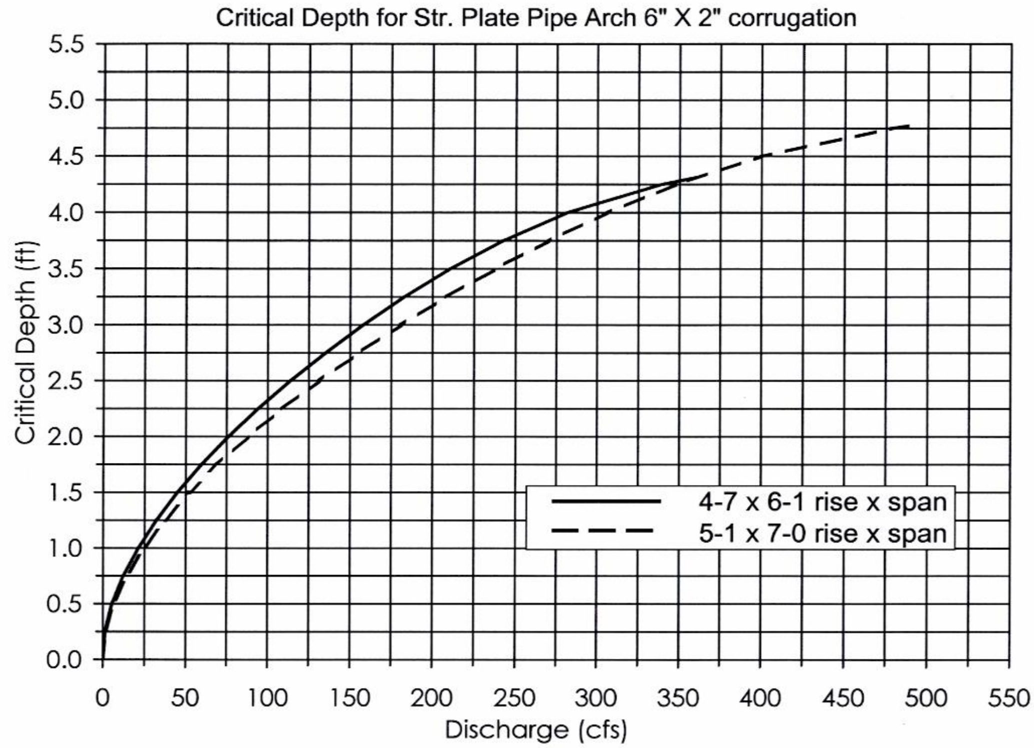
Chart 8-29

Critical Depth Str. Plate Pipe Arch 6 x 2 in. Corrugation



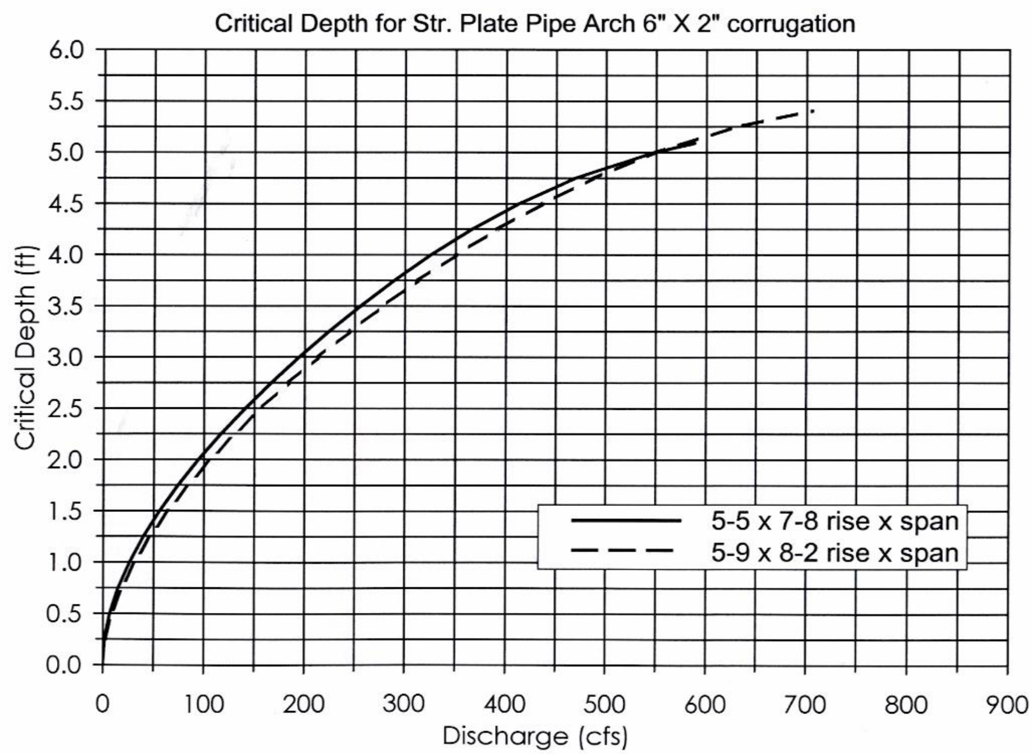


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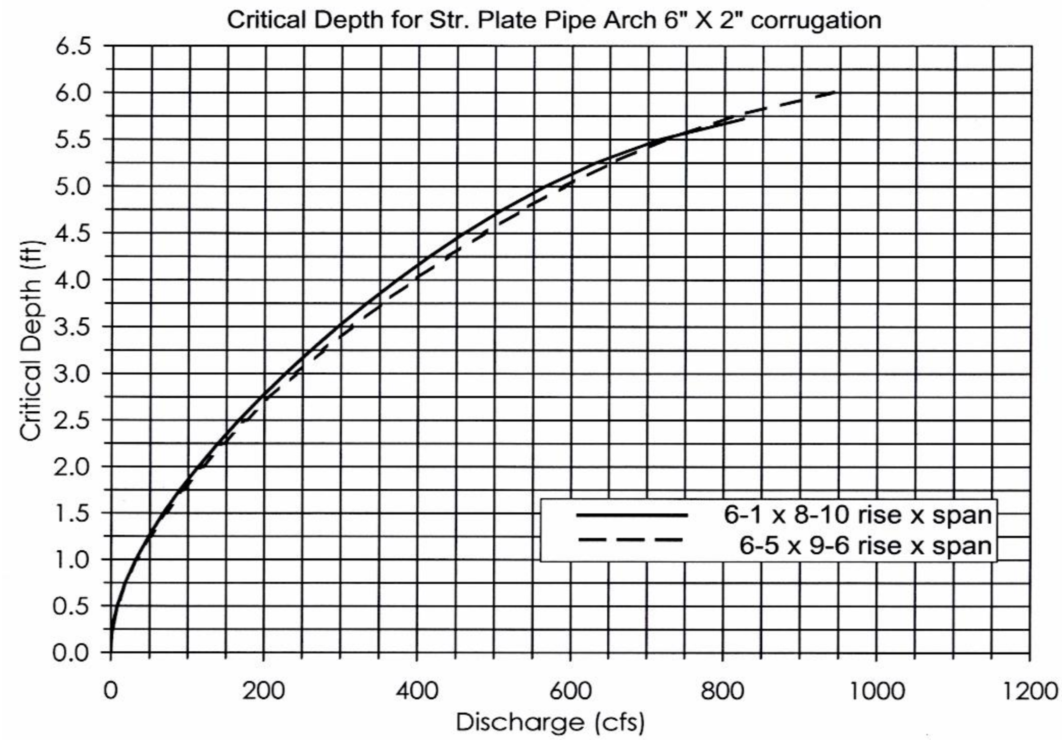
Chart 8-30

Critical Depth Str. Plate Pipe Arch 6 x 2 in. Corrugation



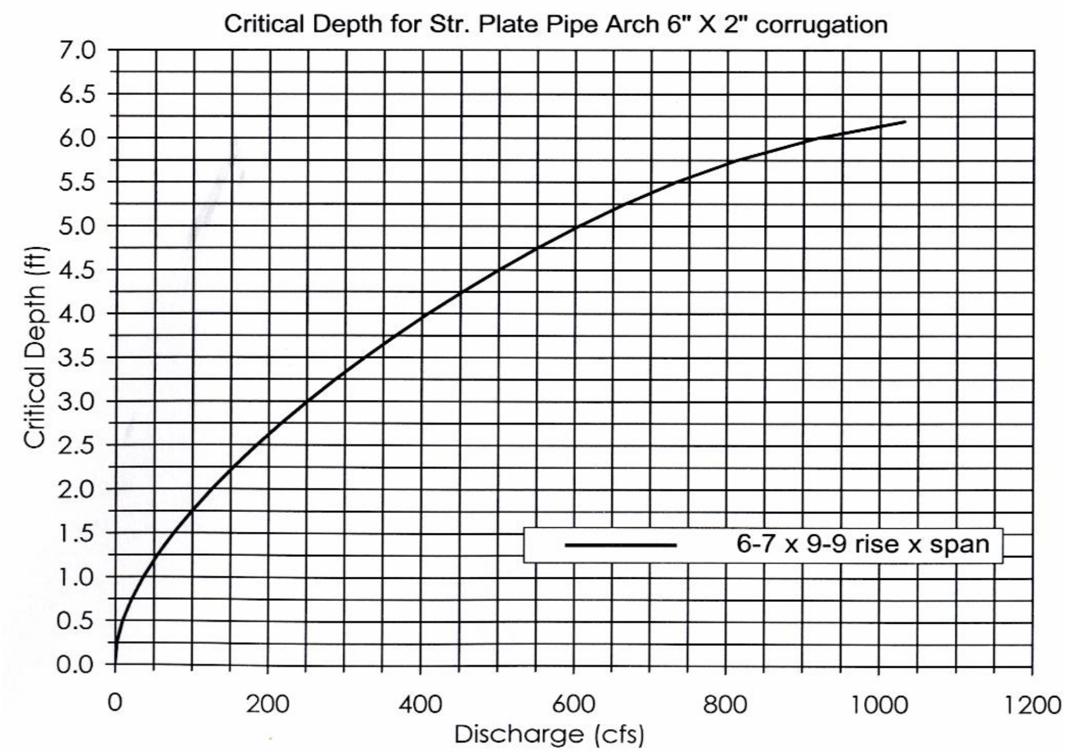


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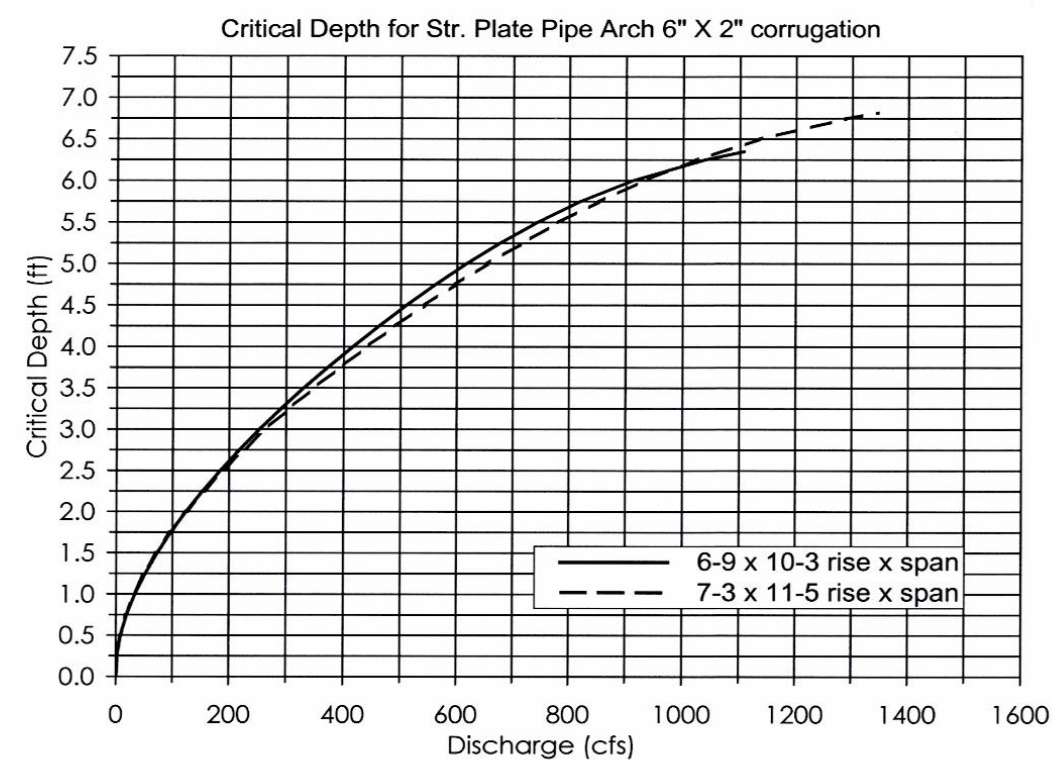
Chart 8-31

Critical Depth Str. Plate Pipe Arch 6 x 2 in. Corrugation



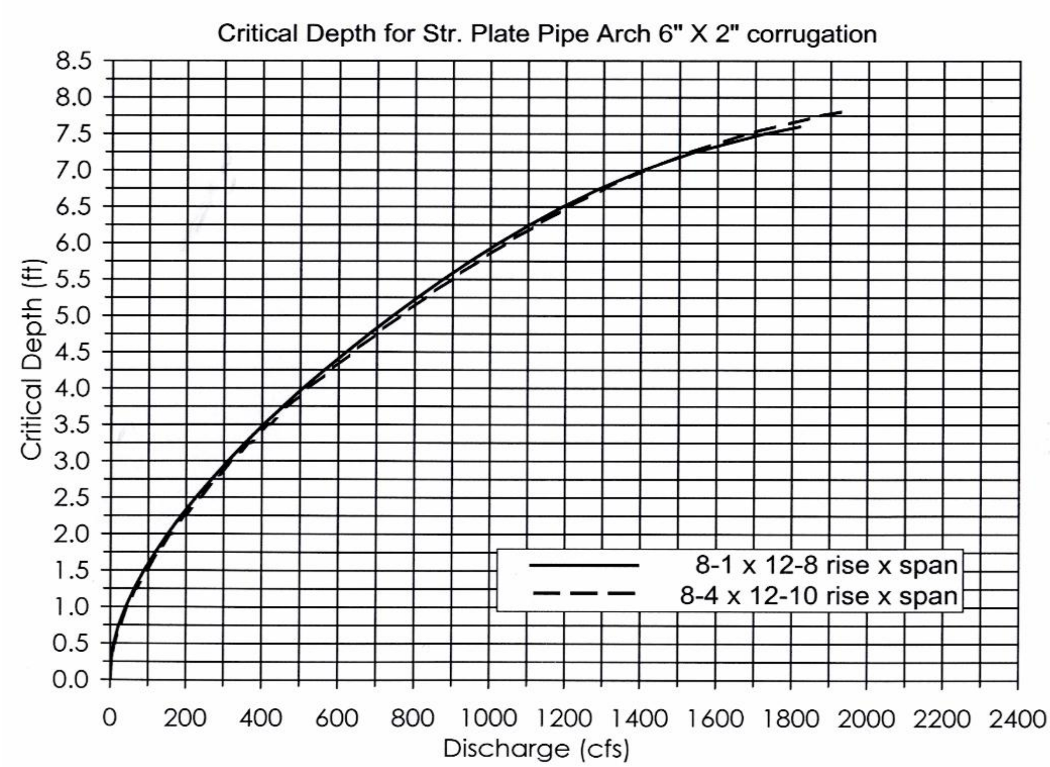


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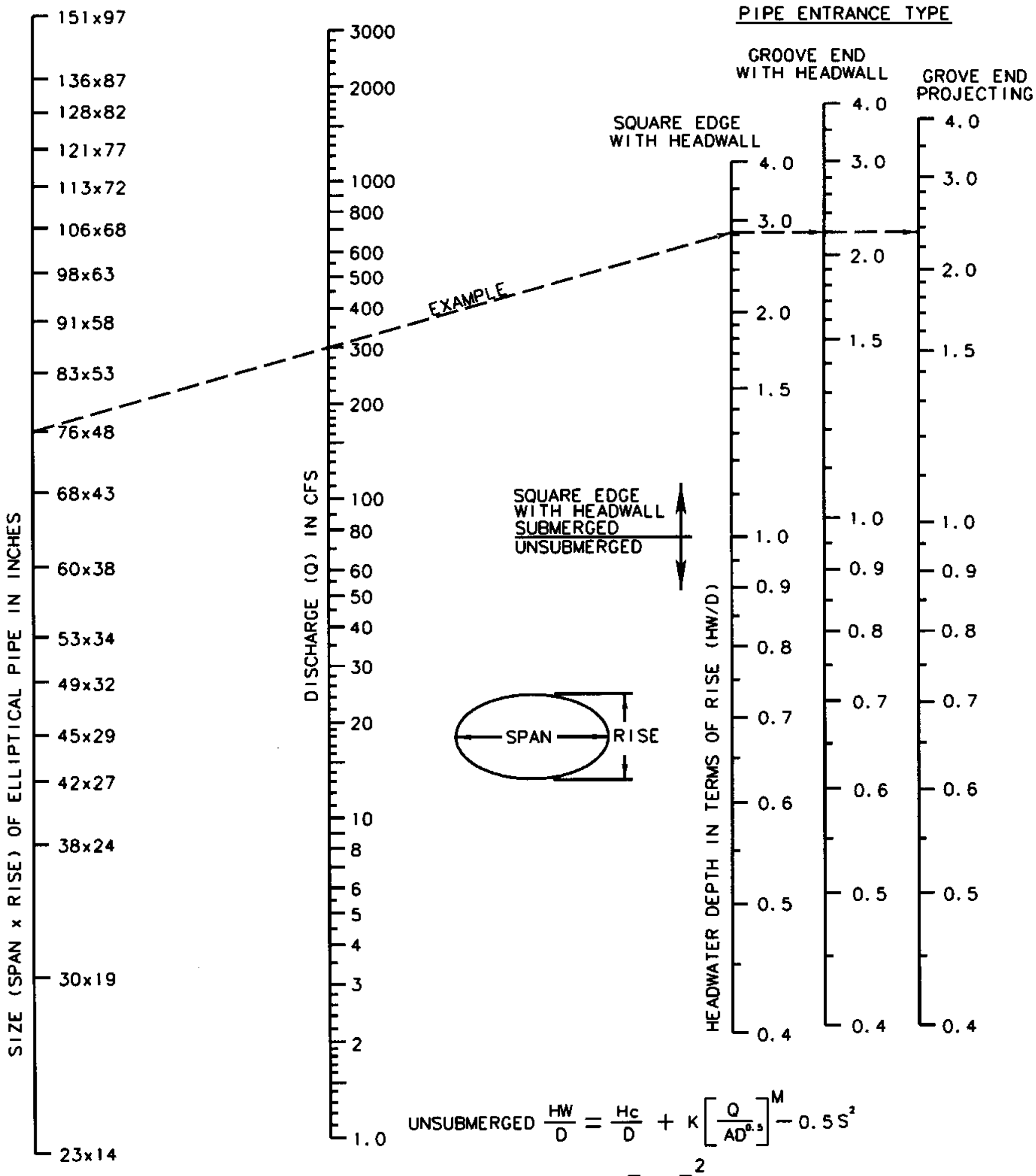
Chart 8-32

Horizontal Elliptical Concrete Pipe with Inlet Control



$$\text{UNSUBMERGED} \quad \frac{HW}{D} = \frac{Hc}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^{M} - 0.5S^{2}$$

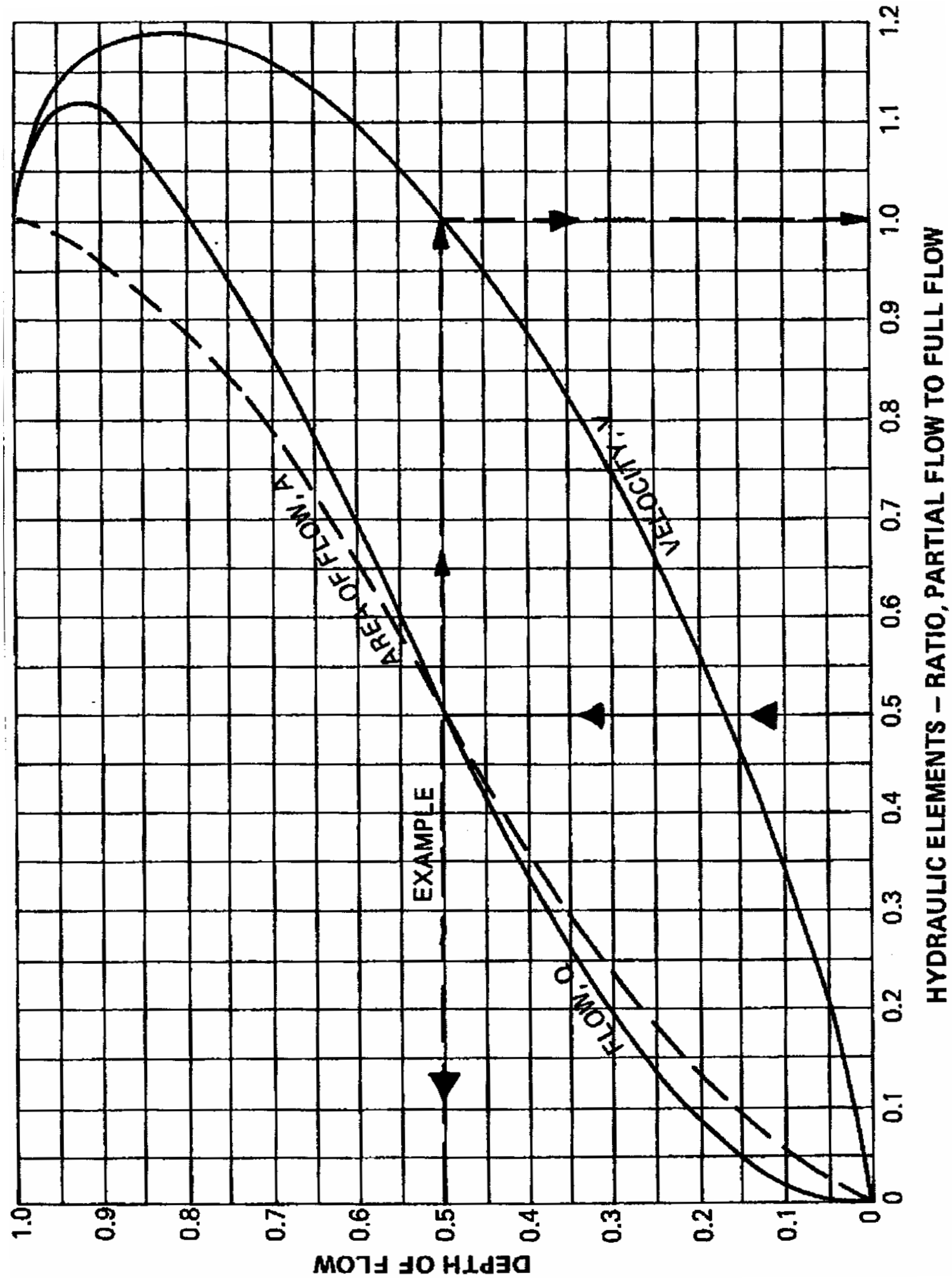
$$\text{SUBMERGED} \quad \frac{HW}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^{2} + Y - 0.5S^{2}$$

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

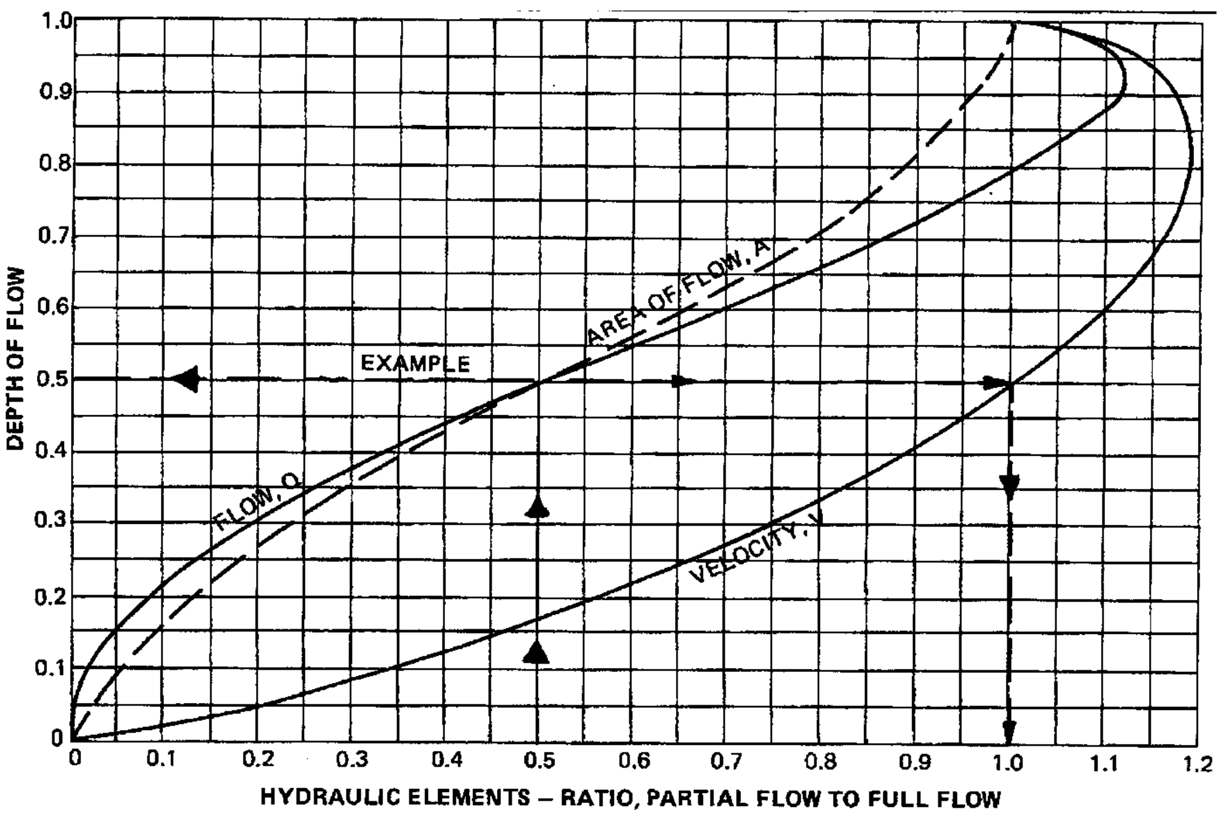
Hydraulic Elements for Partially Full Horizontal Elliptical Concrete Pipe



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

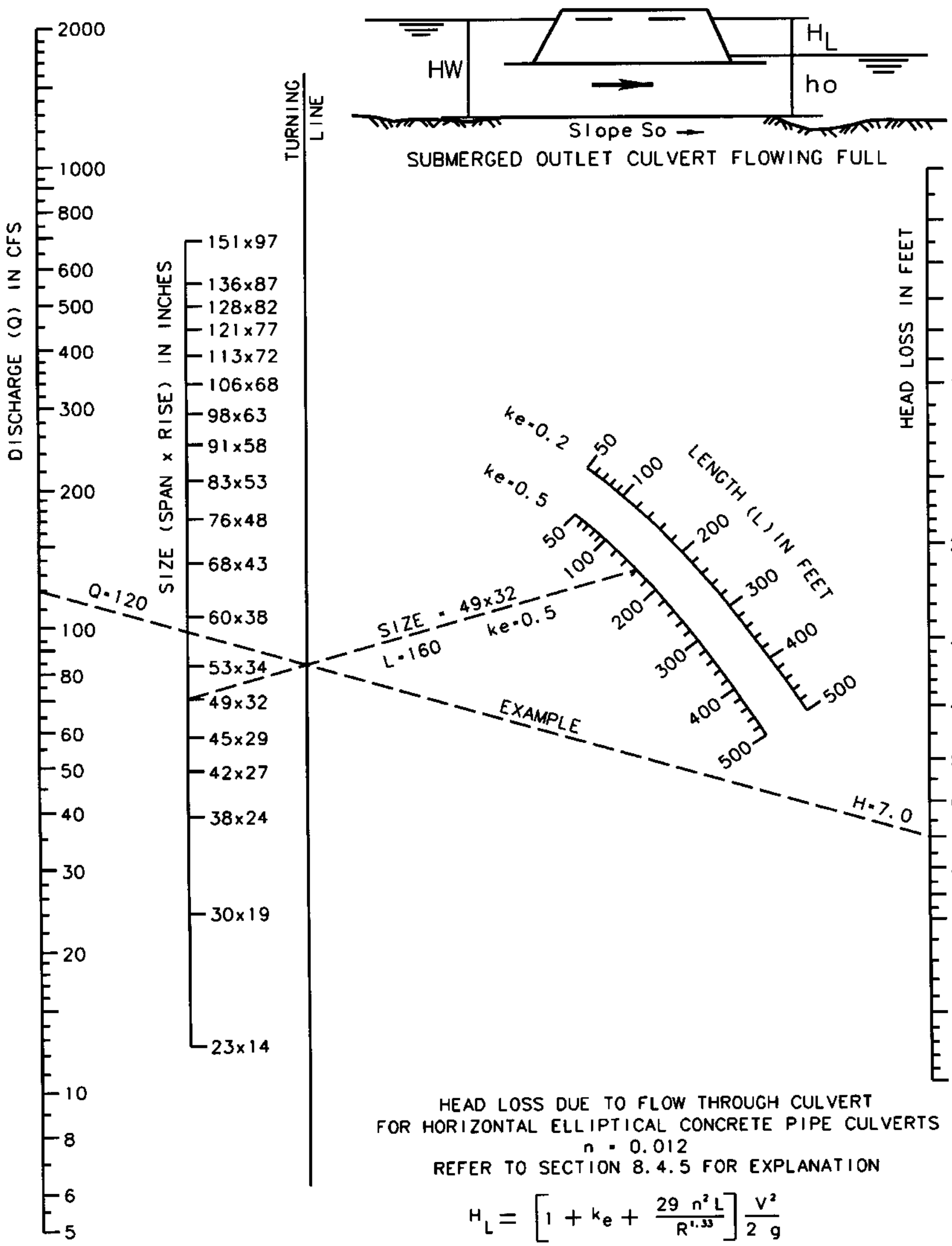
Full Flow Values for Horizontal Elliptical Concrete Pipe

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



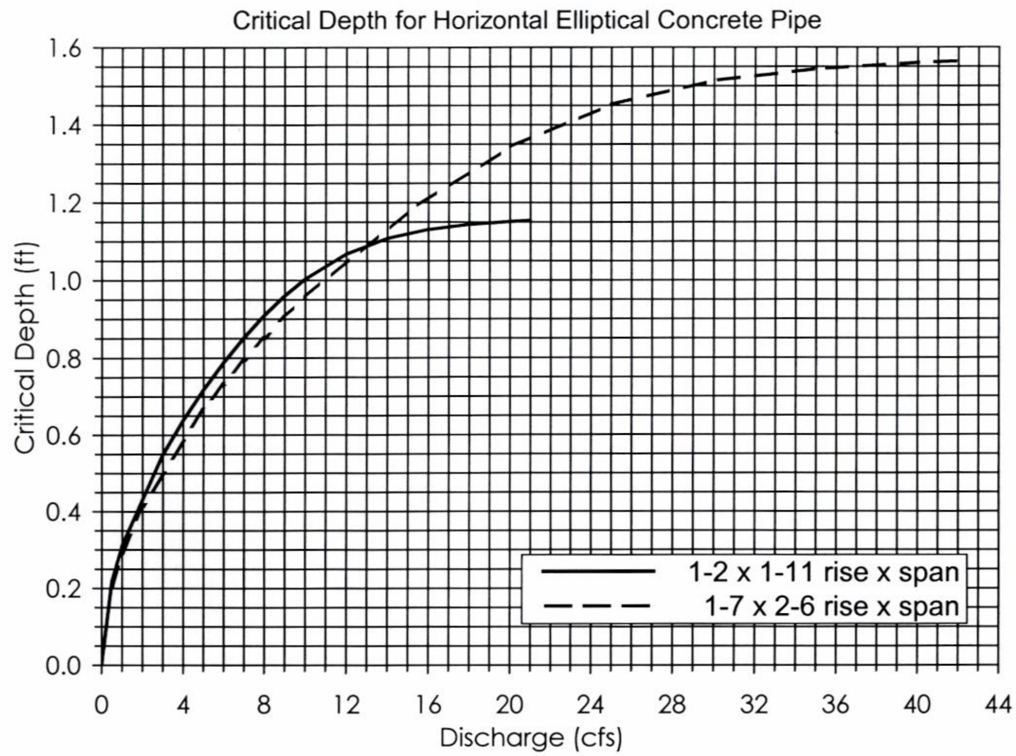
HEAD LOSS DUE TO FLOW THROUGH CULVERT
FOR HORIZONTAL ELLIPTICAL CONCRETE PIPE CULVERTS
n = 0.012
REFER TO SECTION 8.4.5 FOR EXPLANATION

$$H_L = \left[1 + k_e + \frac{29\, n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

Source: Hydraulic Design of Highway Culverts, HDS-5, FHWA, 2005

Chart 8-36

Critical Depth for Horiz. Elliptical Concrete Pipe



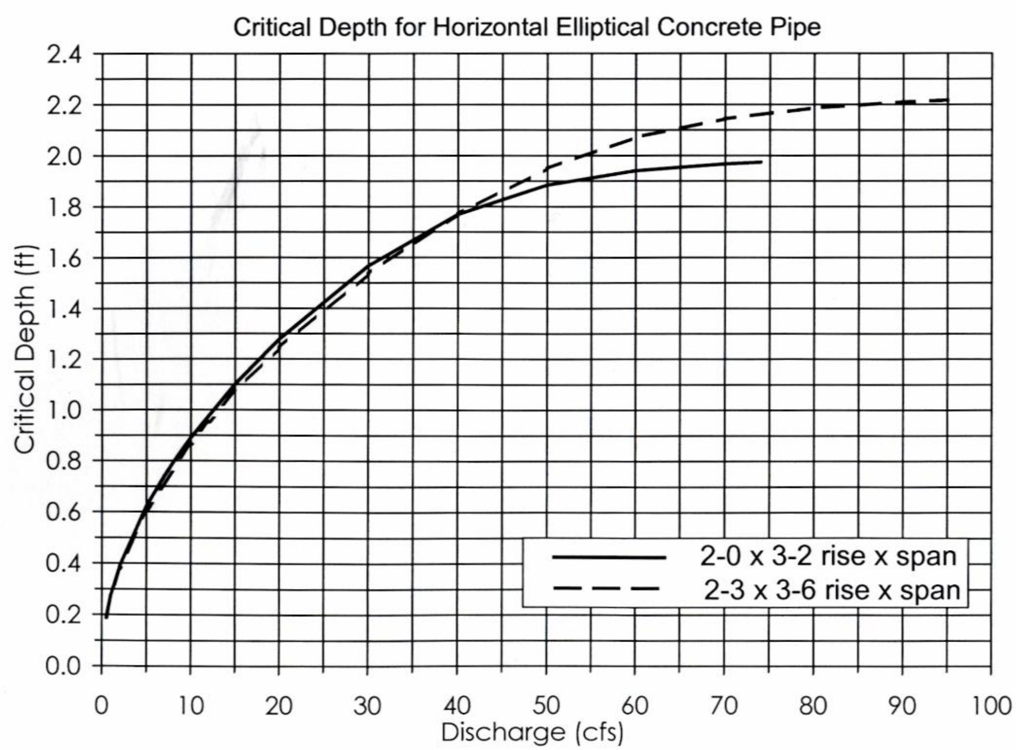


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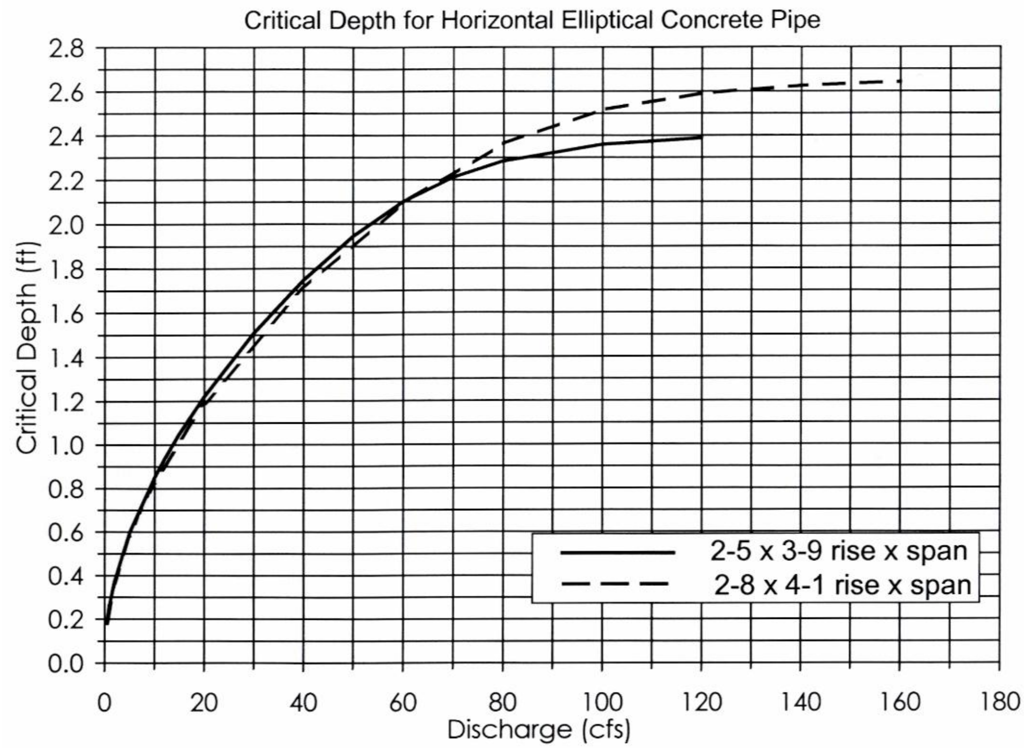
Chart 8-37

Critical Depth for Horiz. Elliptical Concrete Pipe



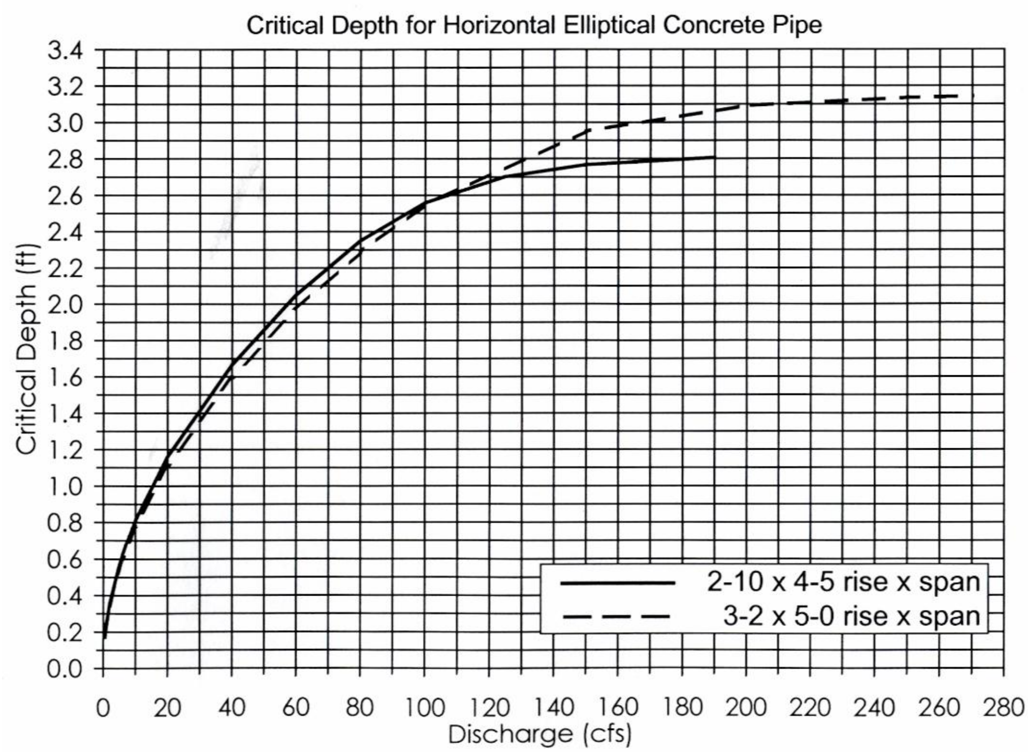


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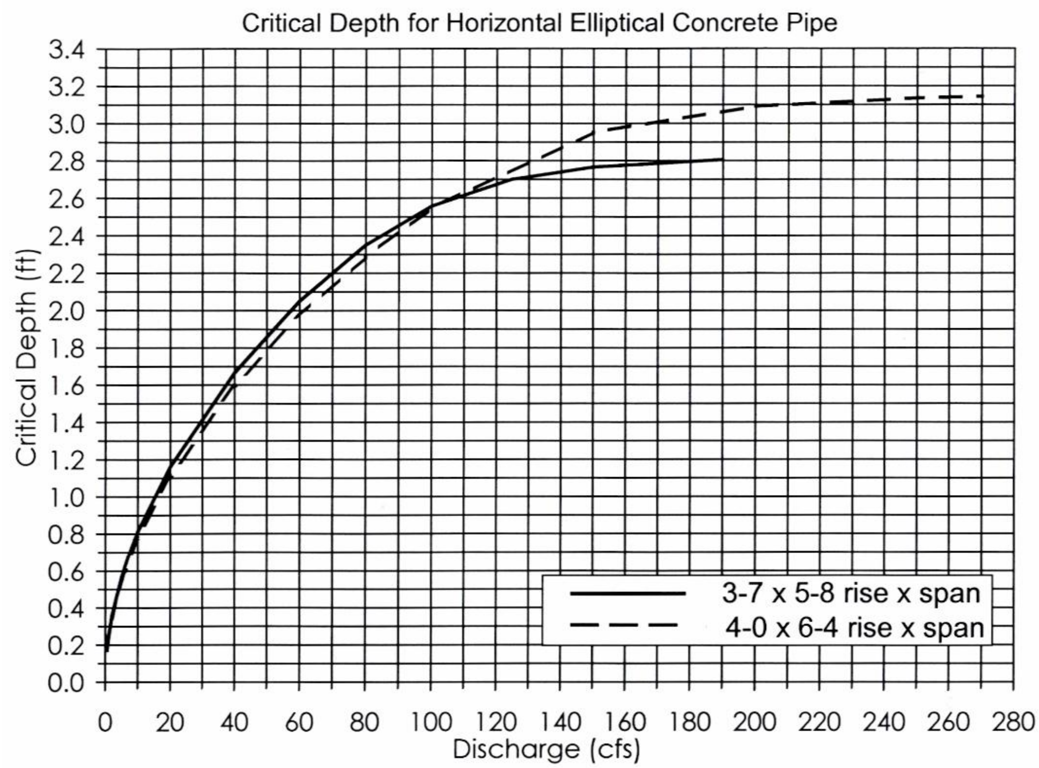
Chart 8-38

Critical Depth for Horiz. Elliptical Concrete Pipe



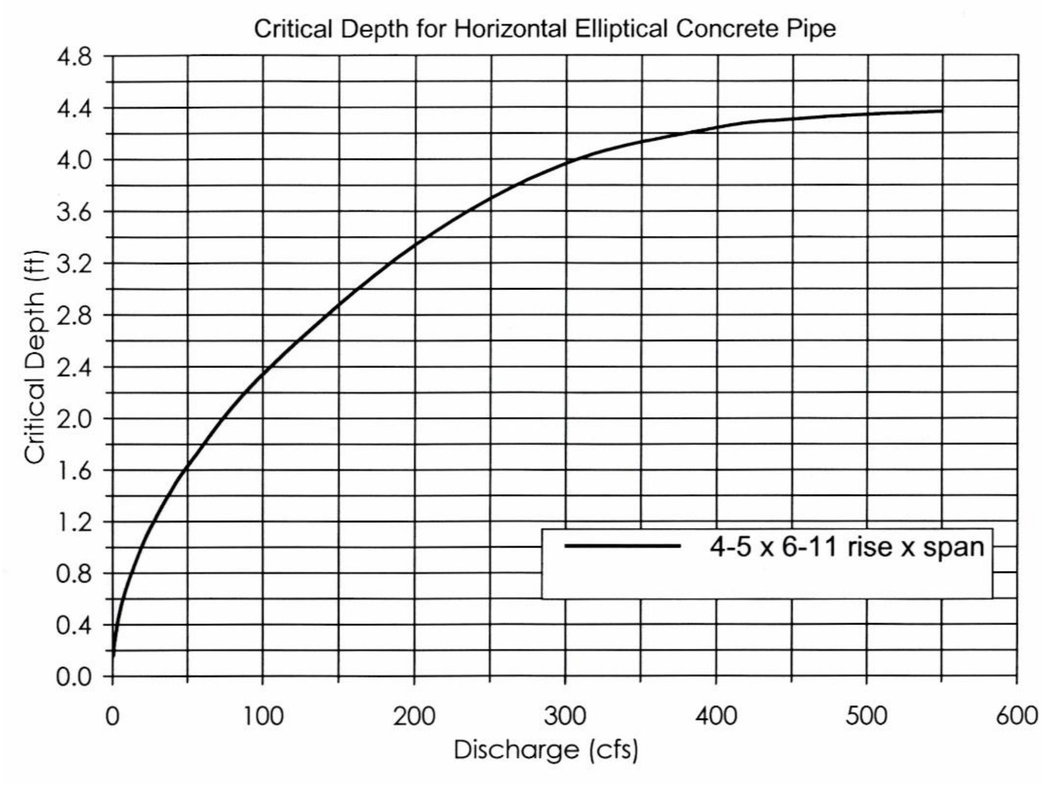


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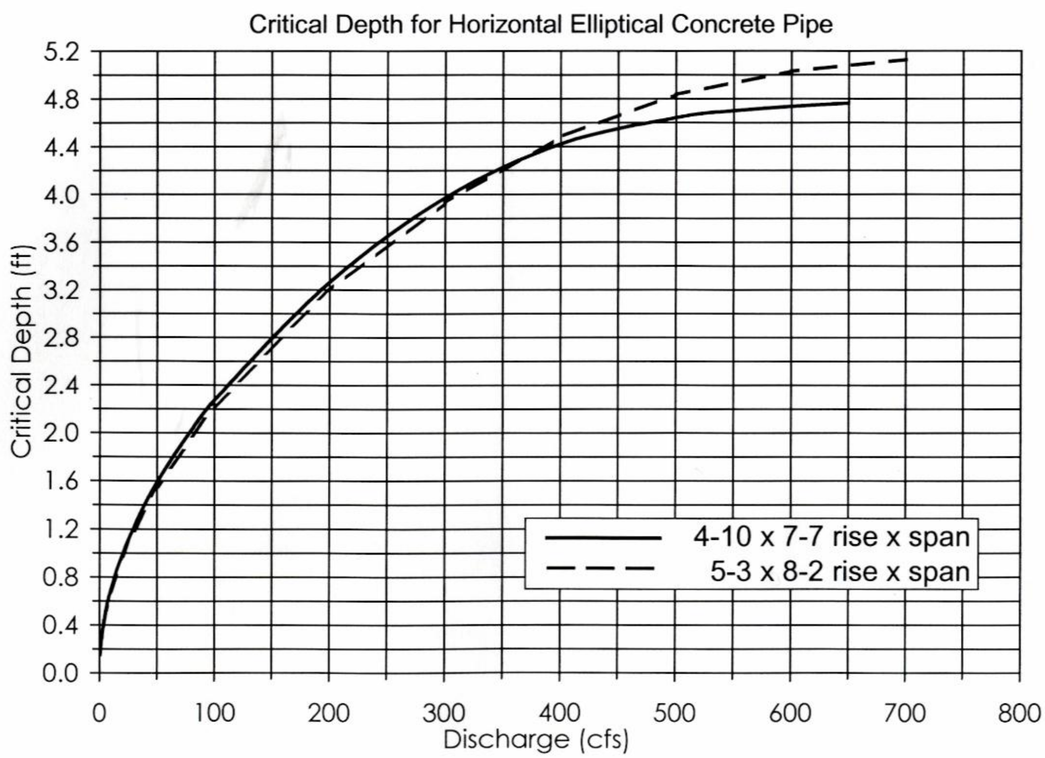
Chart 8-39

Critical Depth for Horiz. Elliptical Concrete Pipe



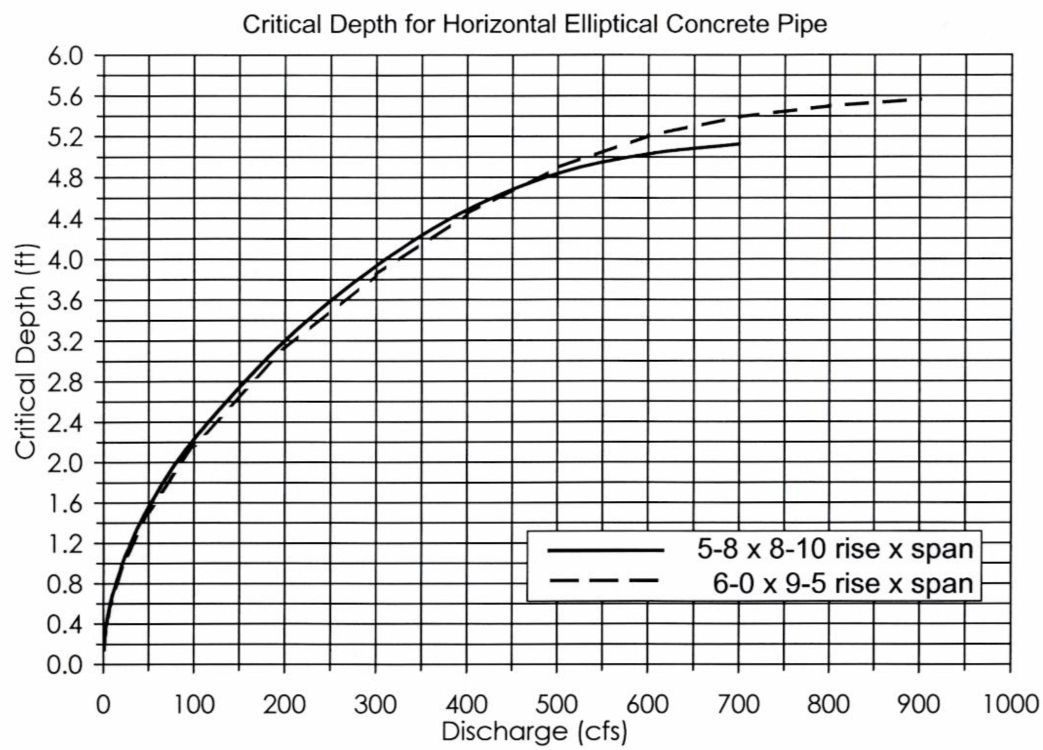


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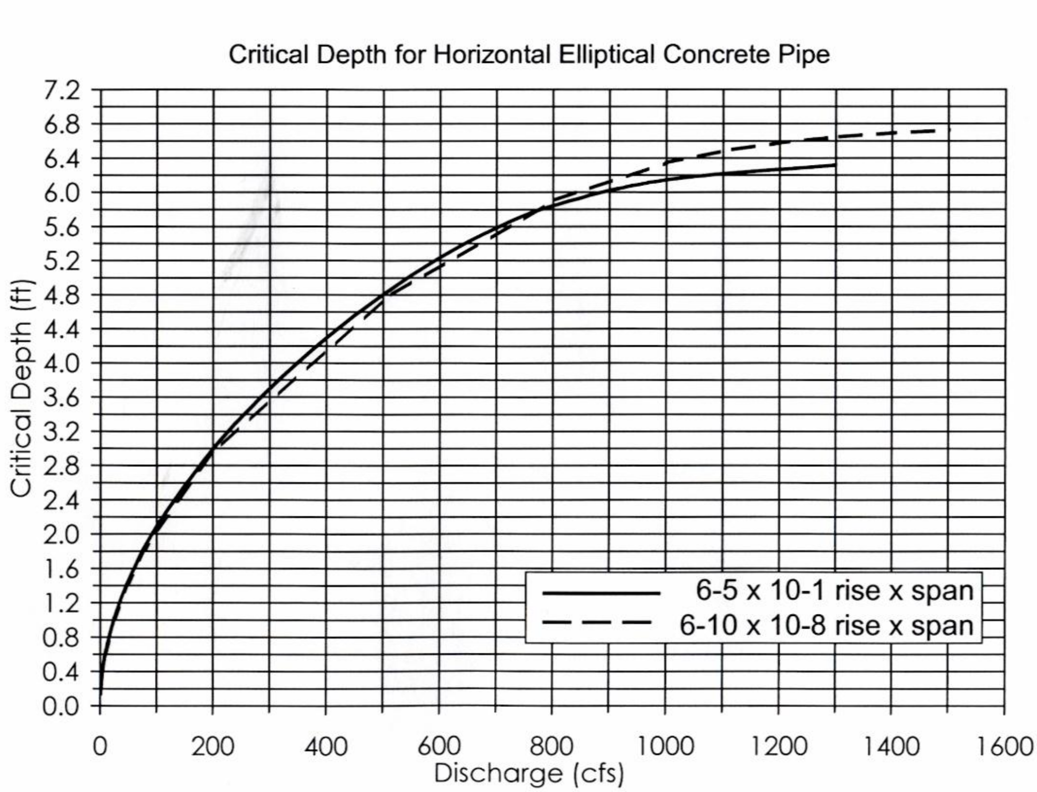
Chart 8-40

Critical Depth for Horiz. Elliptical Concrete Pipe



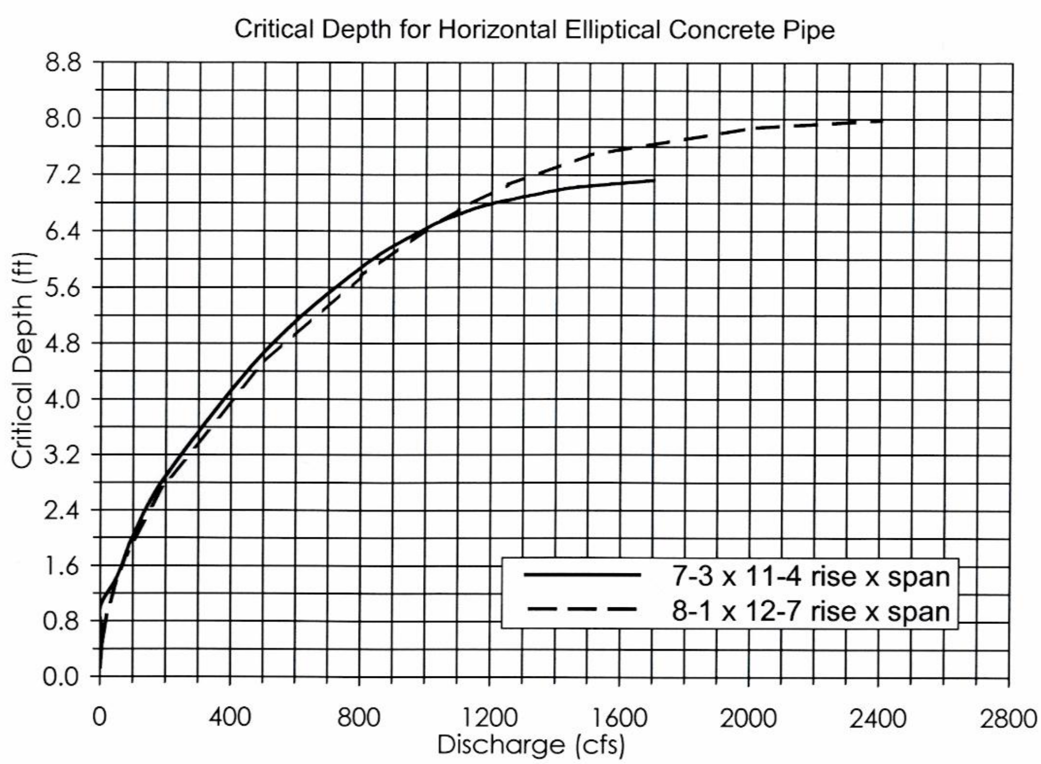


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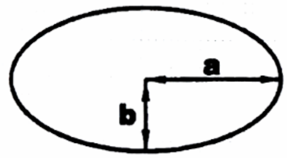
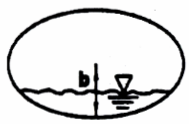
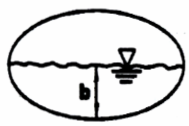
Figure 8–9
Elliptical Pipe Flow Area

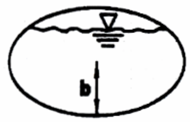




$$b - depth > 0 \quad A = a\,b \arccos\left(\frac{b - depth}{b}\right) - a\left(b - depth\right)\sqrt{1 - \left(\frac{b - depth}{b}\right)^2}$$



$$b - depth = 0 \quad A = \frac{\pi}{2}\,a\,b$$



$$b - depth < 0 \quad A = \pi\,a\,b - \left[a\,b \arccos\left(\frac{depth - b}{b}\right) - a\left(depth - b\right)\sqrt{1 - \left(\frac{depth - b}{b}\right)^2}\right]$$

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

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(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

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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

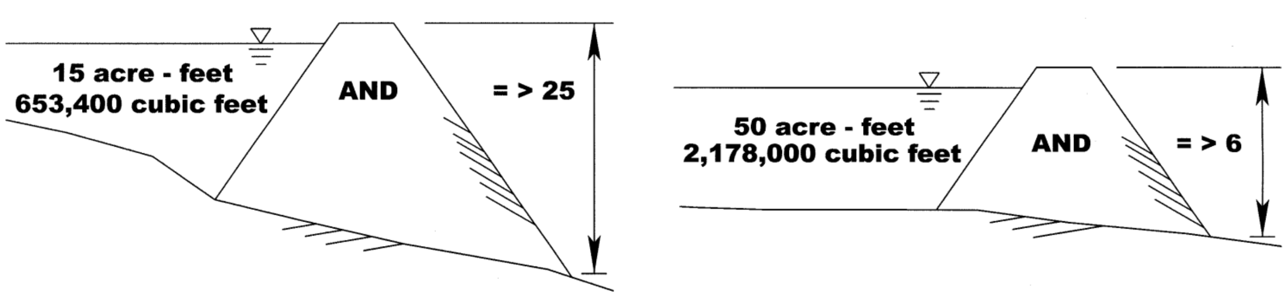
The construction of storage facilities usually requires placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments should be less than 25 feet in height and should have side slopes no steeper than 3H:1V. Side slopes shall be benched at intervals of 5 feet. Riprap-protected 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:

$$V_{1,2} = ((A_1 + A_2)/2)D$$

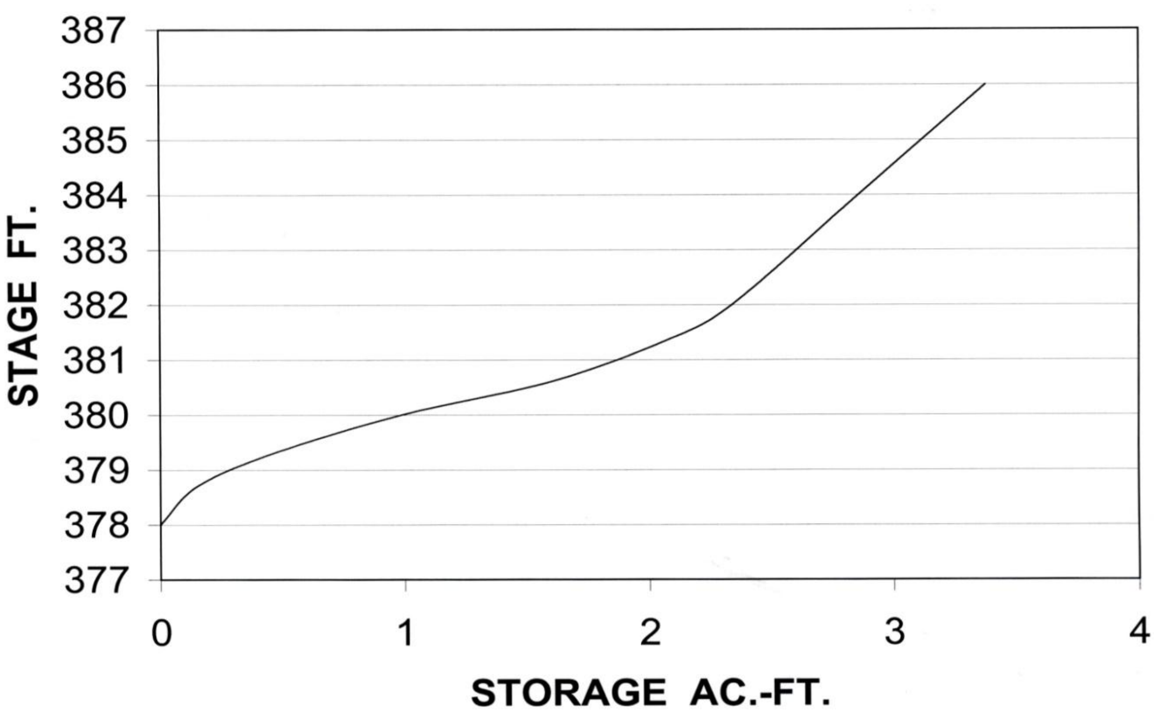
Where: $V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2

$A_{1,2}$ = 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

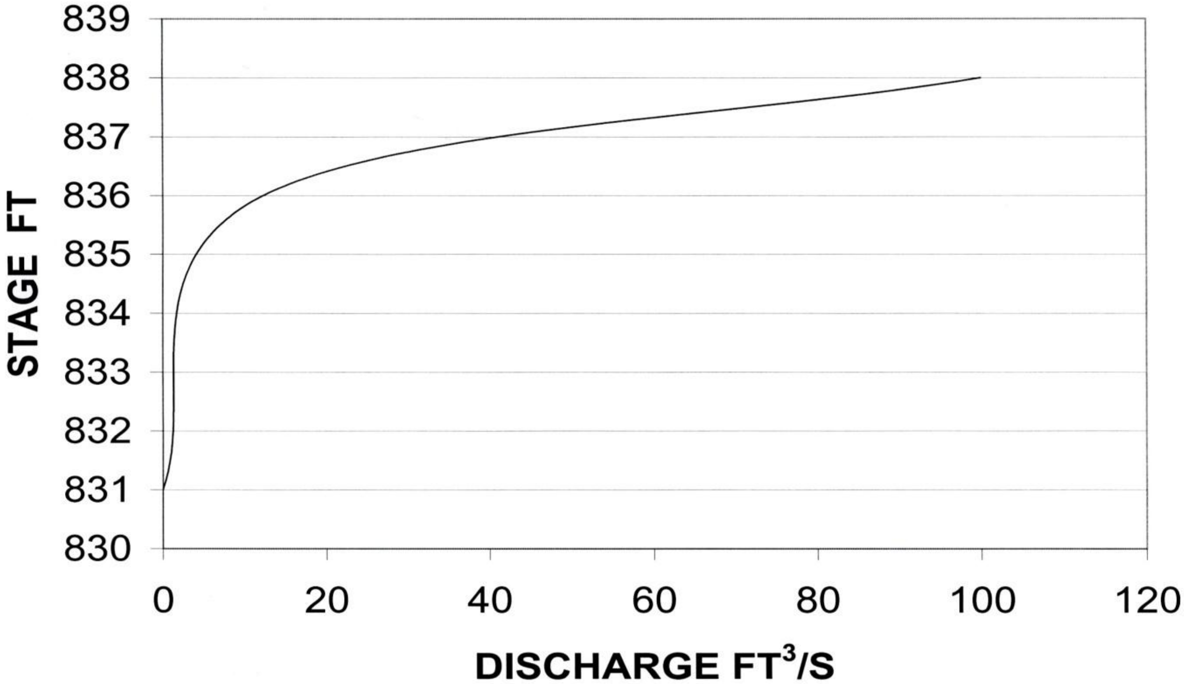


9.3.4 STAGE-DISCHARGE CURVE

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)

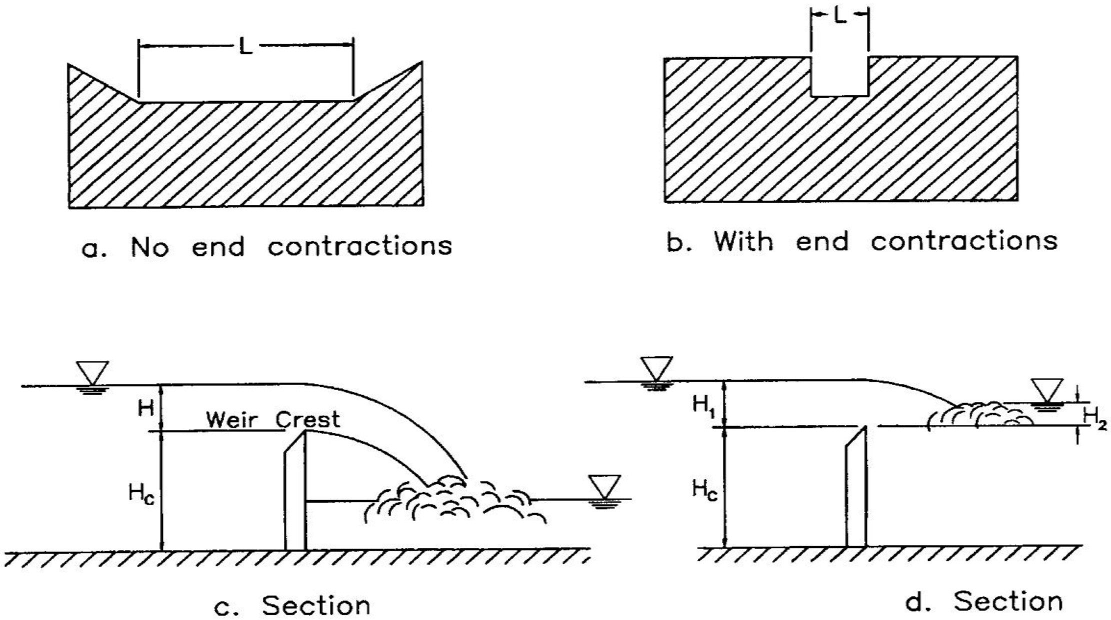
L = horizontal weir length (ft)

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)

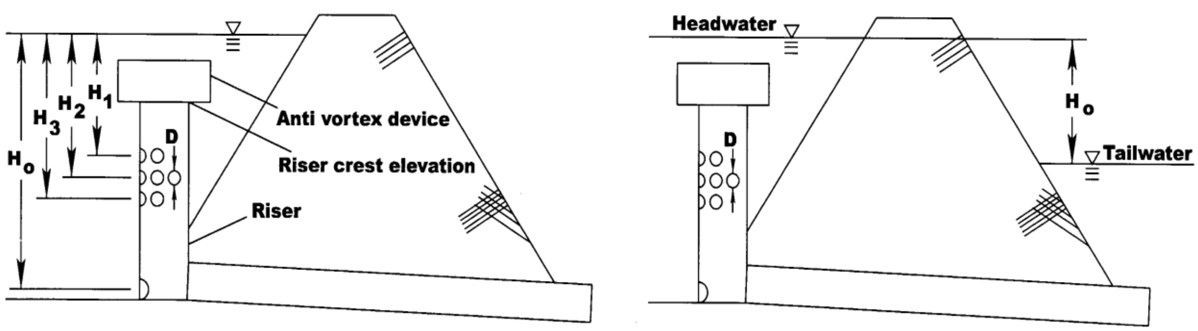
H_1 = upstream head above crest (ft)

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

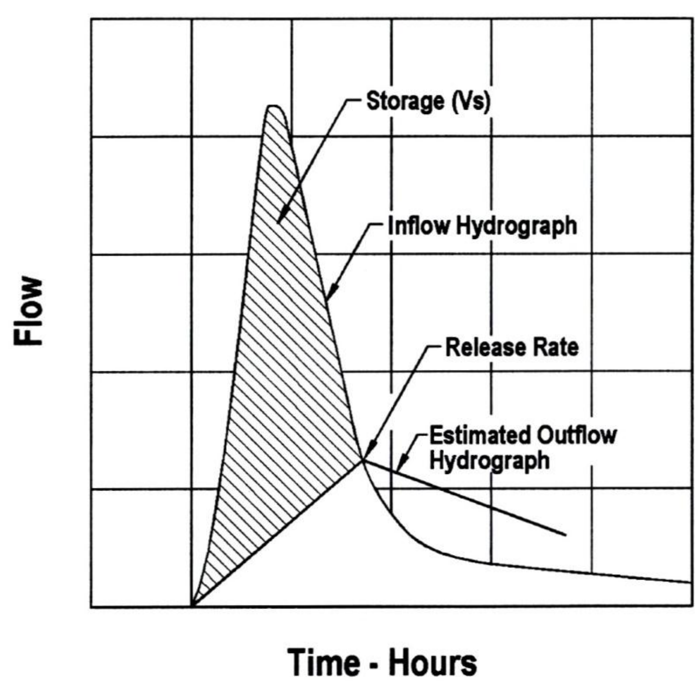
Estimating the required volume of storage to accomplish the necessary peak reduction is an important task since an accurate first estimate will reduce the number of trials involved in the routing procedure. The following sections present three methods for determining an initial estimate of the storage 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

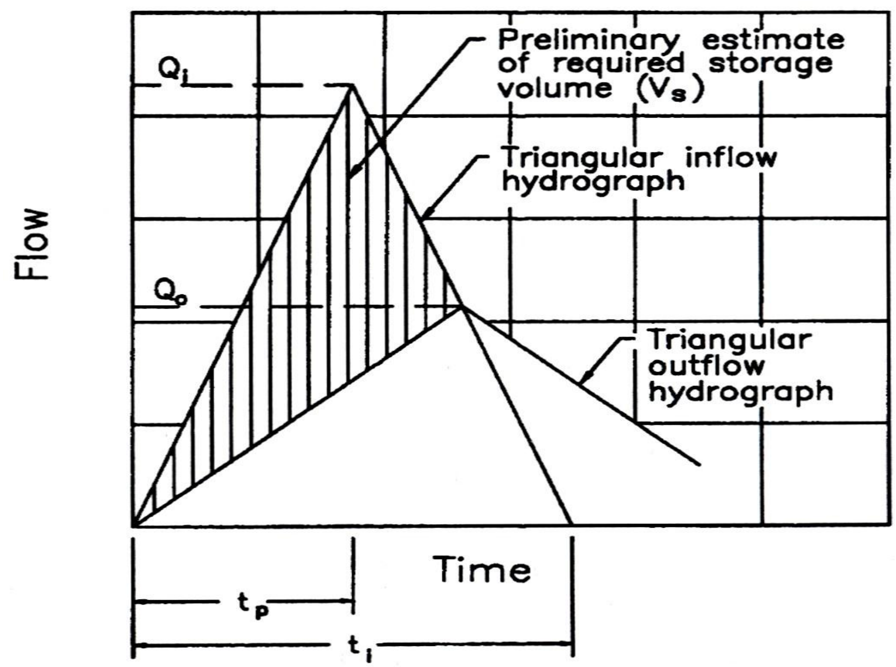
Hydrograph Method



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

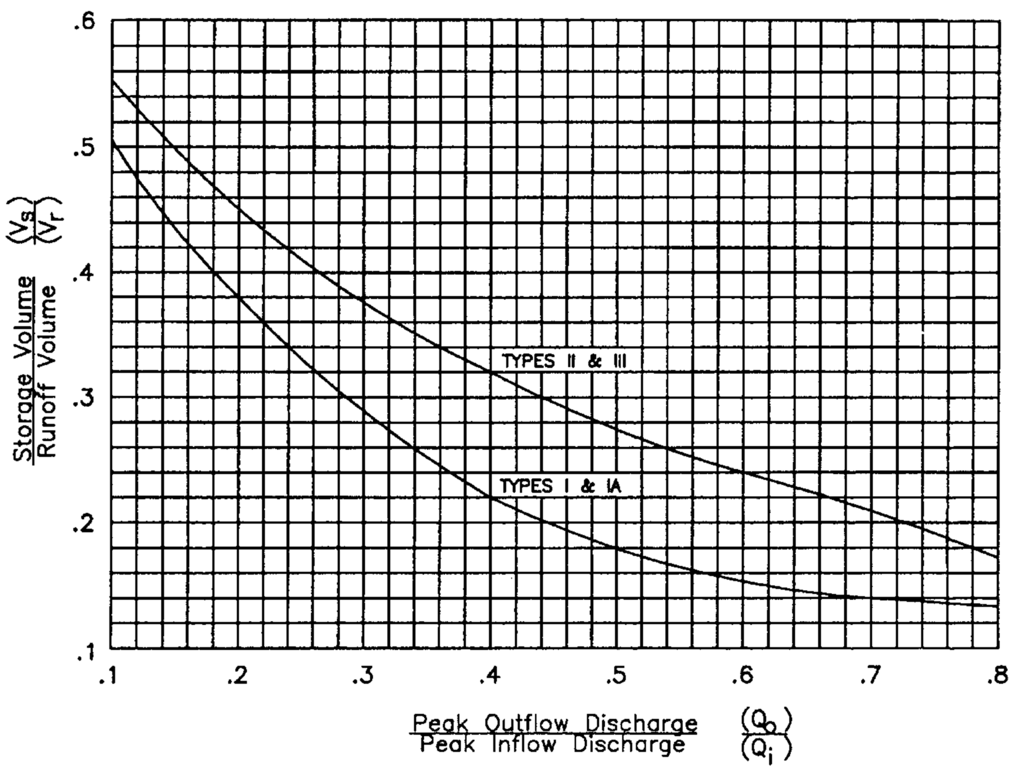
T_i = Duration of basin inflow, s

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 ($I - O = \Delta\Sigma$). 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)

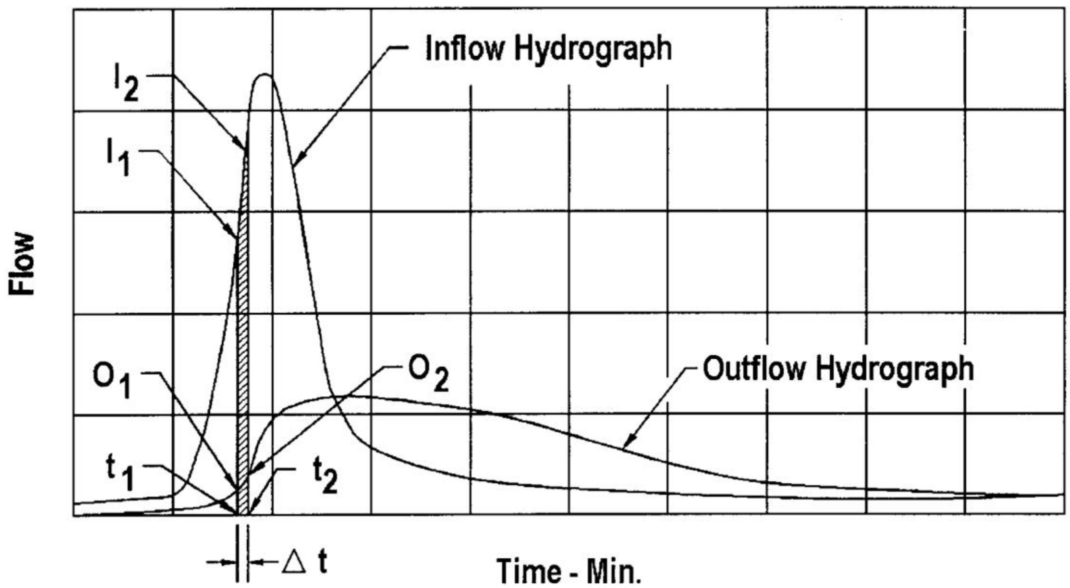
Δt = time interval, min

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.

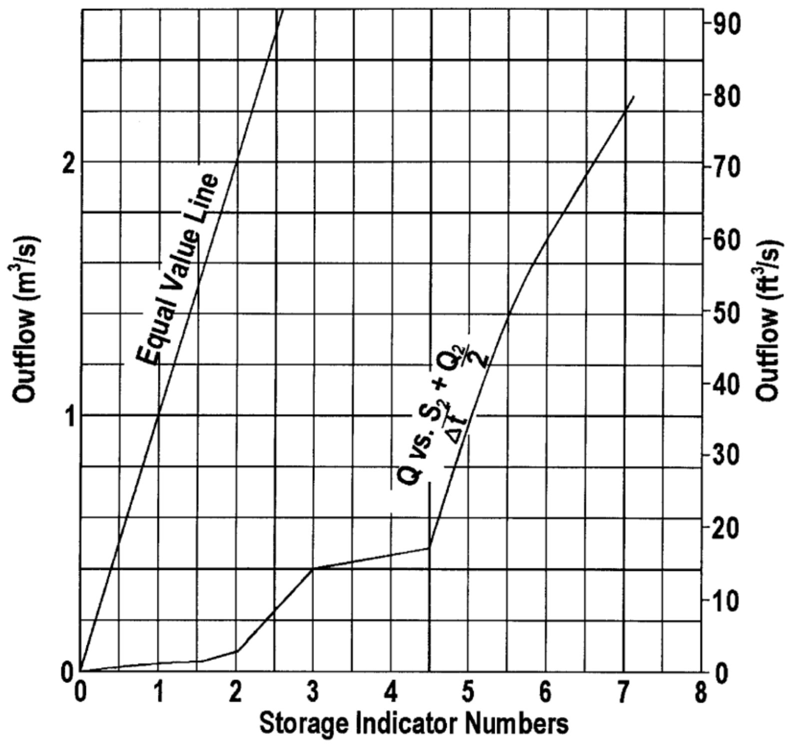
c. The time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.

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

(1) Chow, C. N. Open Channel Hydraulics, New York: McGraw Hill Book Company, 1959

(2) Design of Small Dams. U.S. Department of the Interior, 1977

(3) Model Drainage Manual, AASHTO, 1991

(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

(5) Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22, Second Edition, Federal Highway Administration, August 2001

WEST VIRGINIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

2007
DRAINAGE MANUAL

CHAPTER 10:
BRIDGES

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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

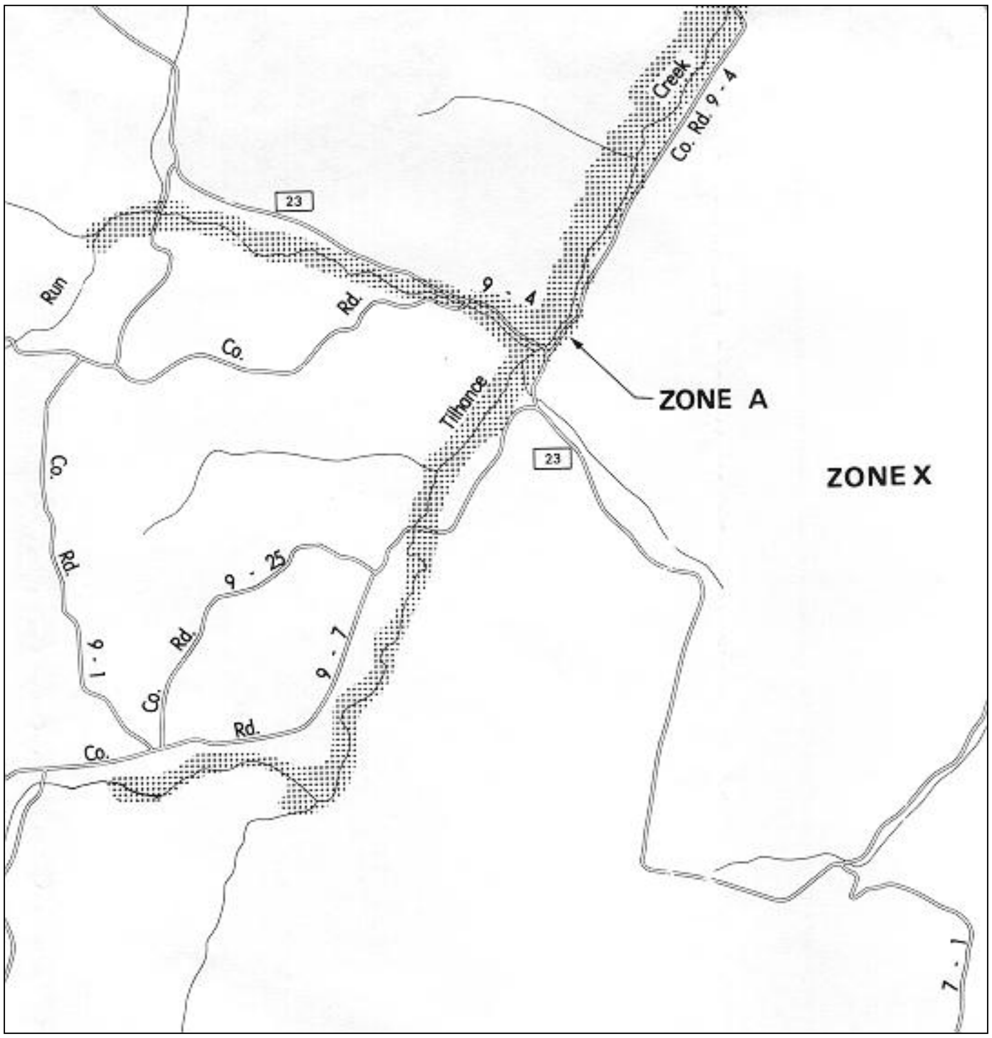
The designer should be familiar with the following common acronyms:

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

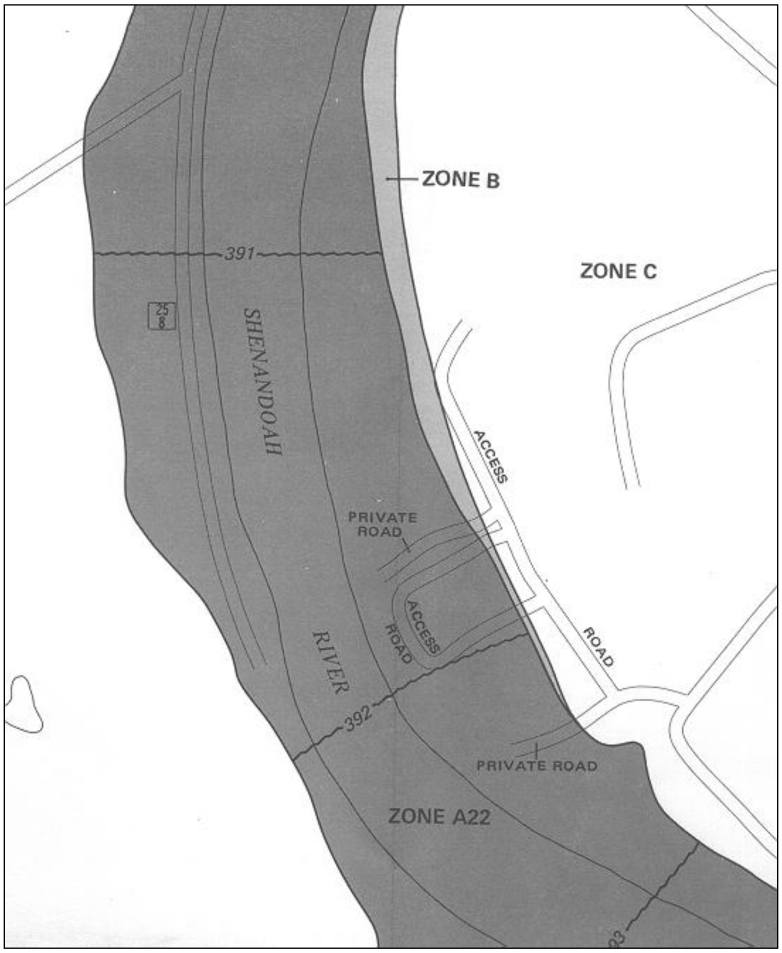
Example FHBM: Berkeley County, WV



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.

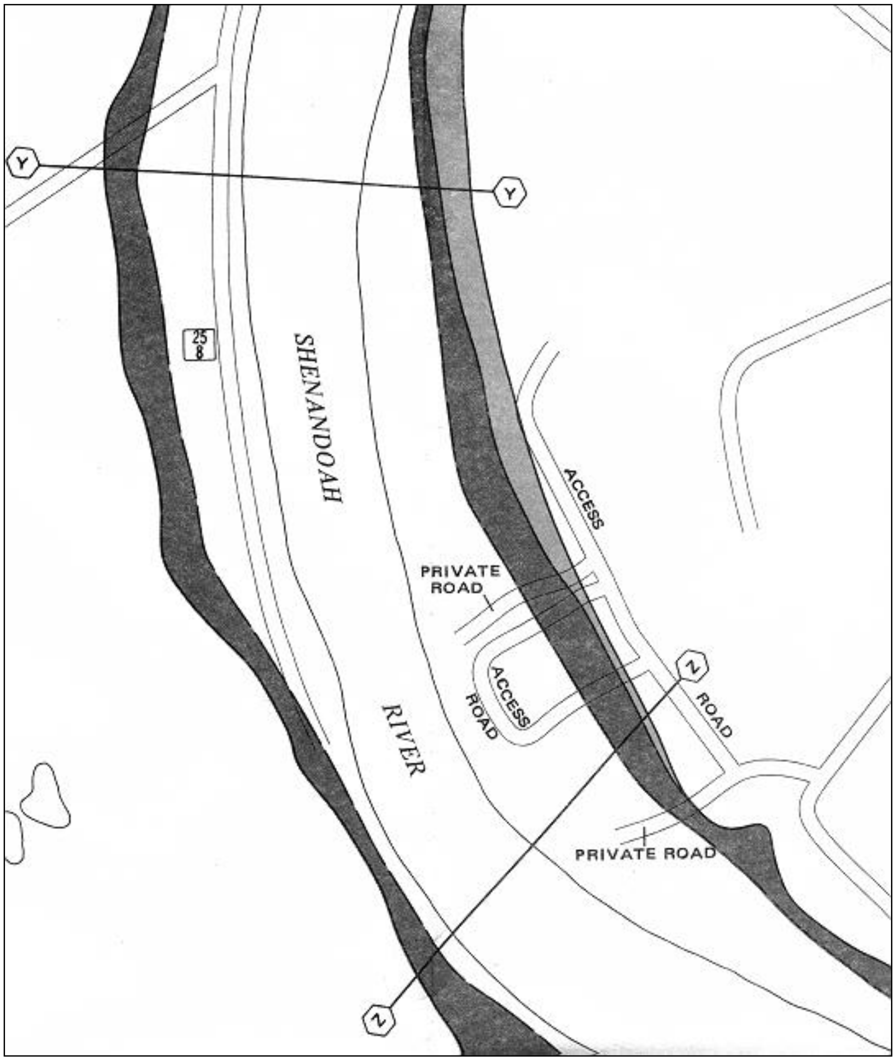
Zone C and Zone X (unshaded): Area of minimal flood hazard, usually depicted on FIRMs as above the 500-year flood level. Zone C may have ponding and local drainage problems that don't warrant a detailed study or designation as base floodplain. Zone X is the area determined to be outside the 500-year 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

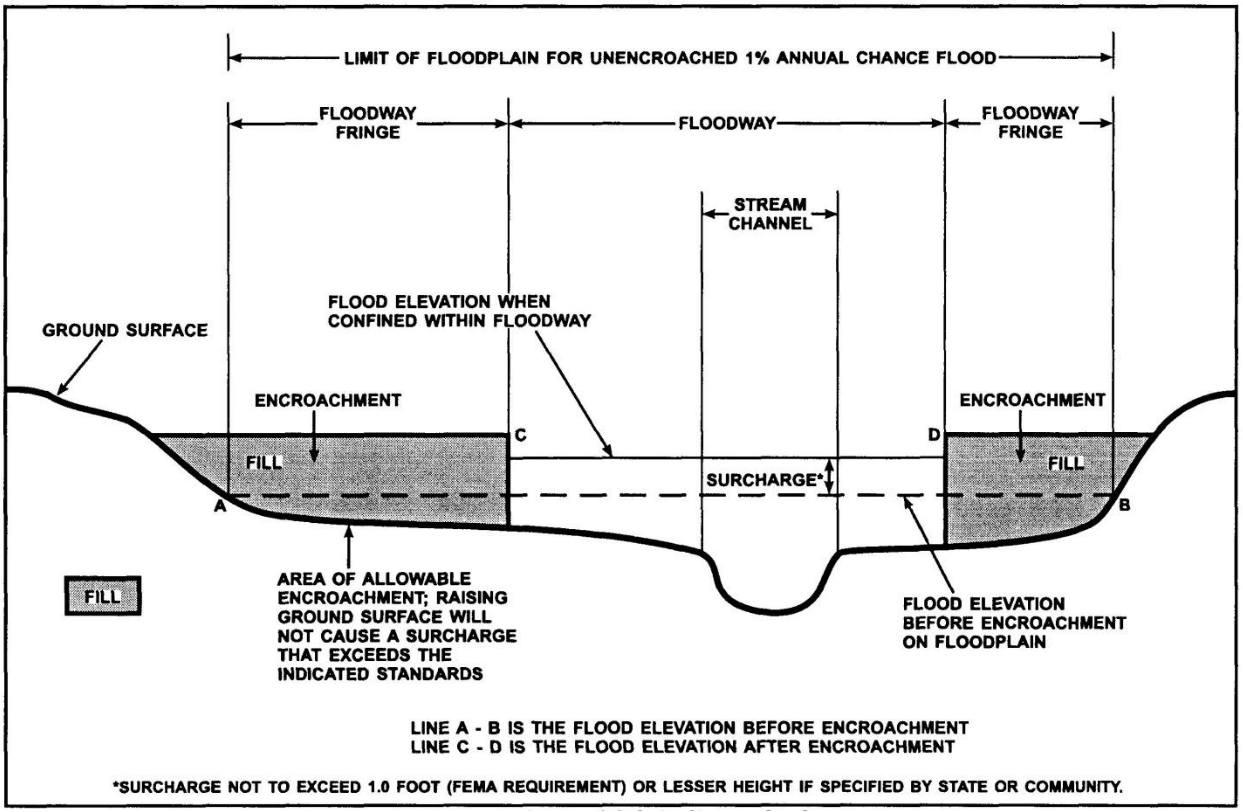
Example FBFM: Jefferson County, WV



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)

<u>Flow Criteria</u>

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.

<u>Risk Criteria</u>

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.

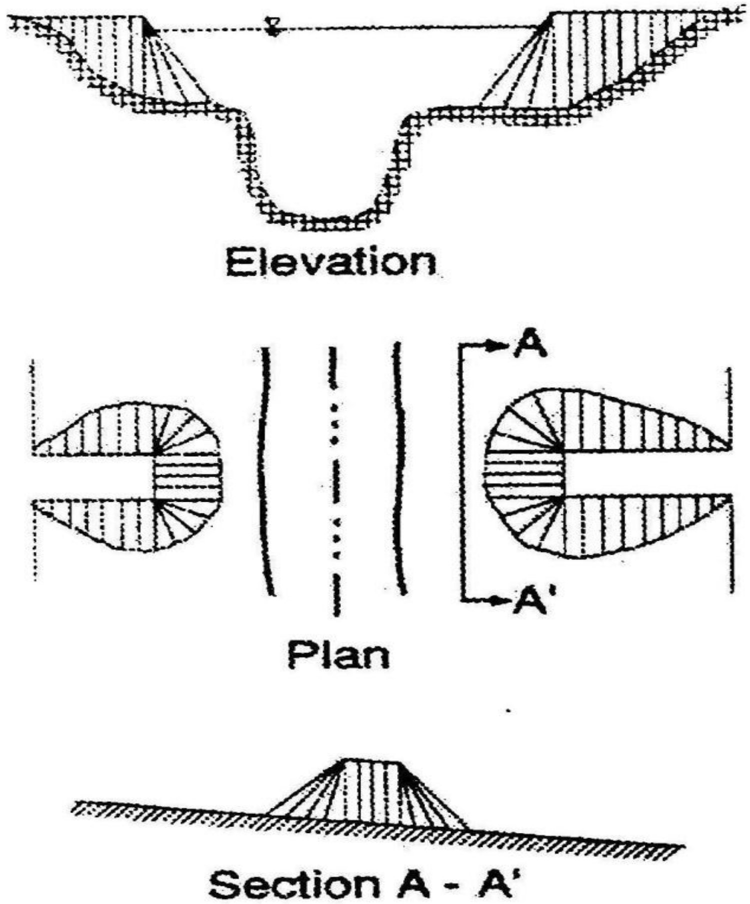
Normal flow is the water flow prevailing during the greater part of the year. Statistically, this is most closely represented by the "mode", which is the value or item occurring most frequently in a series of observations or statistical data. However, for the purpose of meeting the 404 permit requirements, 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.