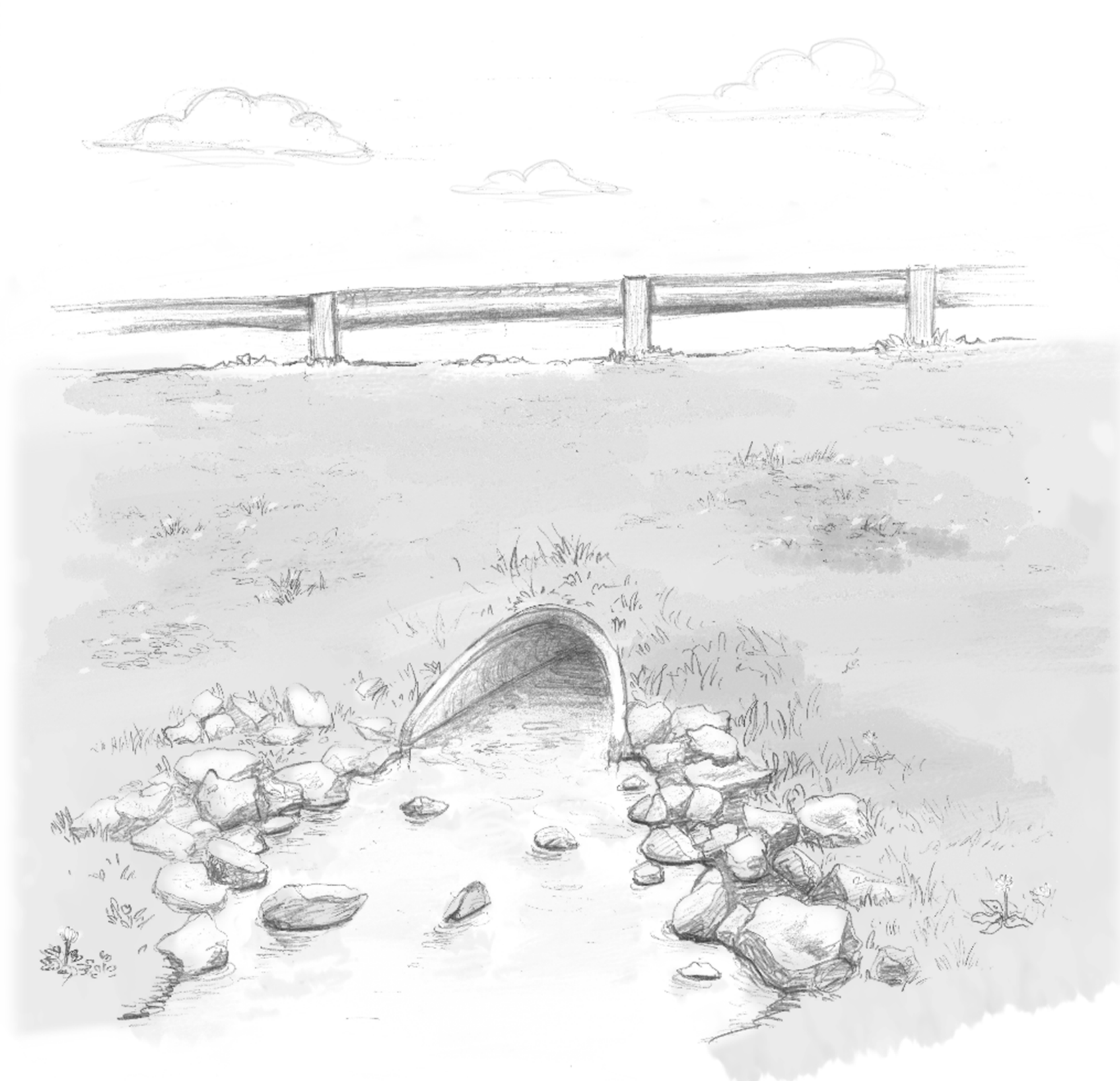
HIGHWAY DRAINAGE MANUAL



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MARYLAND DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION

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1
INTRODUCTION

1.1 Background

The Maryland Department of Transportation State Highway Administration (MDOT SHA) has developed the 2023 Highway Drainage Manual (hereinafter typically referred to as 'this Manual') to document policies and procedures for standard drainage design of MDOT SHA facilities. This document also provides guidance for other MDOT business units such as the Maryland Transportation Authority (MDTA). This edition of the Drainage Manual constitutes a major technical update of the 1981 Highway Drainage Manual. Multiple resources such as the 2014 American Association of State Highway and Transportation Officials' (AASHTO) Drainage Manual were used in the development of this Manual.

The goals of this Manual are to:

- Establish policies and procedures for drainage design
- Educate drainage designers with concise technical information
- Enhance the quality of drainage design submittals

1.2 Scope

This Manual is intended for use by anyone involved in the preparation of MDOT SHA construction plans, analysis of drainage and stormwater infrastructure for MDOT SHA, or design of projects that will interact with MDOT SHA facilities. This Manual includes chapters on the following subjects:

Hydrology: Chapter 2 covers the development of hydrologic analyses including methodology choice, rainfall data selection, and drainage area development.

Open Channel Hydraulics: Chapter 3 covers the analysis and design of roadside channels, including freeboard requirements and stability assessment techniques.

Culverts: Chapter 4 covers the analysis and design of roadway culverts, including design storm requirements, hydraulic design practices, outfall protection, and pipe material guidance.

Storm Drain Systems: Chapter 5 covers the analysis and design of closed storm drain systems including design requirements for surface interception and system conveyance.

1.3 DESIGNER RESPONSIBILITY

The standards in this Manual provide a basis for uniform design practices for typical drainage design situations. Realizing that drainage design is primarily a matter of sound application of good engineering judgment, it is impossible to provide precise rules that may apply to all possible situations and scenarios that may arise. For appropriate design, engineers must preserve a reasonable and practical level of flexibility to account for safety, varying site conditions, regulations, and sustainability. Situations will exist in which the standards and requirements presented herein are inappropriate and do not apply. There will also be situations in which these standards and requirements are only adequate when exceeded. **The inappropriate use of and/or adherence to these standards does not exempt nor absolve the engineer from the professional responsibility of developing an appropriate and safe design.** The engineer is responsible for identifying standards that do not apply to a particular situation and for obtaining approval to deviate from the standards. Authority for project-specific changes from this Manual rests with the Highway Hydraulics Division Chief, and all variations from the standards in this Manual must be approved by the Division Chief or assigned designee prior to implementation or inclusion on a project. Design exception requests must include proper justification and documentation.

1.4 PHILOSOPHY

Drainage system design is more than the application of the technical principles of hydrology and hydraulics. Sound drainage design is a matter of properly balancing technical principles and data with the environment, giving due consideration to other important factors such as safety and economics. A balanced design may only be accomplished by using sound engineering judgment and reason.

Design practicable drainage systems to remove water from transportation infrastructure and to convey surface water without causing adverse impacts to safety, adjacent properties, and environmental resources while perpetuating natural drainage patterns and complying with stormwater management (SWM) and other environmental regulations.

1.5 STATE DRAINAGE LAW

Maryland applies a Civil Law doctrine to the flow of surface water. Owners of higher land are entitled to have surface water flow naturally onto the land of an adjoining lower property owner. If the runoff from a highway has historically been flowing to a particular location, the State has a continuing right to have the water flow to that location and the adjoining property owner has no right to interfere with that drainage of surface water. Likewise, if runoff from an adjoining property has been historically flowing onto lower MDOT SHA property as part of a natural drainage system, the adjoining property owner has a continuing right to have the water flow to that location and the State has no right to interfere with that drainage of surface water. No property owner, including MDOT SHA, has the autonomous right to substantially change the volume or direction of the natural flow of surface water.

Courts have adopted a “reasonableness of use” rule that’s intended to balance the benefits and harms caused by modifications to surface water flow. Essentially this rule implies that a landowner acting in good faith may modify surface water flow if it’s for a legitimate and necessary purpose and does not cause injury or hardship to the downstream property. If an injured party seeks injunctive relief, the court system would ultimately decide the reasonableness and resultant monetary damages, if applicable.

1.6 STANDARDS

The drainage design criteria are proportionate with the relative importance of roadways and other transportation assets, associated risks, and possible damage to adjacent properties. Rather than designing only to meet minimum criteria, it is the responsibility of designers to optimize design considering risk, safety, and function versus cost.

Various MDOT SHA publications play an integral role supporting and supplementing the content of this Manual. These include, but are not limited to the following publications:

Standard Specifications for Construction and Materials
Book of Standards - for Highway & Incidental Structures
Stormwater Management and Sediment & Erosion Control Resources

Many standards outlined in this Manual apply to roadway drainage structures with a drainage area of less than 400 acres. Refer to the Office of Structures Manual for Hydrologic and Hydraulic Design when designing structures and roadway encroachments on floodplains or streams with a drainage area of 400 acres or greater.

While the standards presented in this Manual conform to FHWA requirements, drainage designers can find additional information on Federal policy at the Federal Highway Policy and Guidance Center.

1.7 OBJECTIVES

- Facilitate safe and efficient travel for all users of Maryland's highway system by providing drainage features that adequately remove surface water from highway assets.
- Provide drainage features that convey and discharge surface water in a stable manner that protects public and private infrastructure within, adjacent to, or upstream or downstream of MDOT SHA right-of-way.
- Understand the connectivity of drainage systems and minimize the effects of proposed drainage design on existing drainage structures and systems handling the same flows.
- Perpetuate natural drainage patterns to the extent it is practical.
- Maintain roadway, structure, and embankment integrity by providing adequate subsurface drainage. Groundwater studies must be coordinated with the Office of Materials Technology (OMT) Engineering Geology Division (EGD).
- Avoid or minimize impacts to environmental resources. Aspects of wetland and waterway protection must be coordinated with the Office of Environmental Design (OED) Environmental Programs Division (EPD).
- Provide access for maintenance operations wherever practicable.
- Incorporate resilient design by considering non-stationary land use and climate conditions.

1.8 ECONOMICS

Economic considerations are a significant portion of the decision-making parameters for all aspects of drainage design. Designers should consider the following economic factors in the selection of design alternatives and include justifications that support the chosen alternative in the narrative of the Drainage Design Report. Include computations when necessary and appropriate.

- Initial cost of construction.
- Right-of-way and easements necessary to construct and maintain facilities.
- Utility impacts and relocation costs.
- Maintenance of traffic (MOT), including detours, road closures, and associated costs.
- Service life of materials.
- Proximity of trees and other woody vegetation that pose potential risk to the long-term function and maintainability of the drainage system.
- Accessibility by maintenance personnel and costs for cleanout, repairs, and other pertinent charges that may be incurred during the service life of the system.
- Future needs and compatibility with future projects being planned for the area.
- Flood-related risks to the highway and adjacent properties including potential liabilities for damage.
- Costs to traveling public for delays or extra travel distance due to road closures in the event of a severe weather event or complete system failure.
- The frequency, severity, and economic impact of historic drainage problems in the area.
- Aesthetics and quality-of-life concerns for motorists and neighboring residents.
- Vulnerability to sea level rise, increased rainfall, and other climate change effects over the service life of the asset.

1.9 DOCUMENTATION

Provide documentation in the form of a Drainage Report. The Final Drainage Report must be signed and sealed by a Professional Engineer licensed in the State of Maryland. This Drainage Report is intended to document any supporting computations, justifications, and modeled performances for drainage systems and conveyances. The Drainage Report will be the design record document both for verification and as the basis for future analysis or modifications. The Drainage Report is intended to document drainage assets and is not submitted for regulatory approval.

Refer to the Drainage Report Template for the required documentation and justification. Submit reports to Highway Hydraulics Division at the following milestones:

a. **Preliminary Investigation.** The Drainage Report at the Preliminary Investigation milestone primarily documents the condition of existing drainage systems, known performance issues/complaints, preliminary recommendations for system improvement, outfall conditions, and outfall stabilization recommendations.

b. **Semi-Final Review**. The Drainage Report at the Semi-Final milestone incorporates design computations of the semi-final open and closed drainage systems. Design exceptions will also be requested at this milestone after having been discussed with MDOT SHA Highway Hydraulics Division.

c. **Final Review.** The Final Drainage Report is a complete document. Amend as necessary when design modifications occur that may come in the form of addendums, red line revisions, and green line revisions.

Include approvals of exceptions, with supporting justifications, from the requirements of this Manual in the project Drainage Report. Also include relevant correspondence with any other parties, such as individual property owners, County administrations, or other regulatory agencies, supporting the drainage design and the decisions made therein. Submit an electronic copy of the Final Drainage Report to the Highway Hydraulics Division.

1.10 SOFTWARE

Analysis and design software covering almost every aspect of Hydrology, Hydraulics, Open Channel, Culvert, and Storm Drain Design has been developed over recent decades, with increased usability and functionality as time goes on. These programs include freely licensed software developed by the United States Federal Government and individual states and proprietary software packages developed by various companies. The MDOT SHA allows the use of many up-to-date versions of the government software packages (such as TVHG from MDOT SHA, WinTR-20 from NRCS, Hydraulic Toolbox from FHWA, and HEC-RAS from USACE), as well as many proprietary software packages (such as HydroCAD or Bentley OpenRoads Designer) but does not necessarily approve or endorse the use of all computational methodologies / routines within these software packages.

Software applications used for MDOT SHA must follow methodologies required for design described in this manual (for example, hydrological analysis software using the NRCS methodology), and must be approved for use by MDOT SHA Highway Hydraulics Division (HHD). See Table 1-1 for additional information about software approved by the MDOT SHA HHD. Applications not listed in Table 1-1 may be used when warranted with approval from Highway Hydraulics Division.

Designers must take care to fully understand the methodologies employed by software used for MDOT SHA projects. The designer is responsible for the correct and appropriate use of all software, including computation options, input data, input sensitivity, and interpretation of results.

Table 1-1: Computer Software

USE	SOFTWARE	VERSION*	SOURCE	NOTES
Hydrology	Win TR-20	v 3.30.1 (2022)	NRCS	
	WMS	v. 11.0	AQUAVEO	
	Win TR-55	v. 2.00.0 (2022)	NRCS	
	HydroCAD	v. 10.0 or later	HydroCAD	
	Hydraulic Toolbox	v. 5.2.0 (2022)	FHWA	
	GISHYDRONXT		Univ. of MD	Typically for drainage areas ½ mi^2 and larger
Rainfall Data	Atlas 14	Vol. 2 v.3.0	NOAA	Atlas 14 rainfall data for MD counties compiled by the MDE or NRCS may be substituted for point precipitation frequency estimates
Culvert Analysis and Design	HY-8	v. 7.80.2 (2022)	FHWA	
	HEC-RAS	v. 6.4.1 (2023)	USACE	
Outfall Protection	Hydraulic Toolbox	v. 5.2.0 (2022)	FHWA	At minimum, software used for outfall protection design should follow HEC-14 methodology
	HY-8	v. 7.80.2 (2022)	FHWA	
Culvert Service Life Estimator	CSLE 2022	v. 2.0 (2022)	FDOT	Tool does not currently account for abrasion
Storm Drains	TVHG	(1994)	MDOT SHA	Inquire with HHD for software
	Inroads / OpenRoads Designer		Bentley	
Stream Modeling	HEC-RAS	v. 6.4.1 (2023)	USACE	
Ditch Lining Inlet Analysis Channel/Weir Analysis	Hydraulic Toolbox	v. 5.2.0 (2022)	FHWA	
Riser	SHARISER	(2008)	MDOT SHA	
	HydroCAD	v. 10	HydroCAD	

*Software versions typically represent the latest version at the time of publication. Designers should use the latest software version available. Please contact the Highway Hydraulics Division prior to using different versions for MDOT SHA projects.

1.11 Source Material

This Manual provides basic hydrologic and hydraulic engineering analysis methods common to the engineering industry and indicates methods preferred by MDOT SHA.

Various sources of information have been used to compile this manual, including:

- AASHTO Drainage Manual;
- FHWA Hydraulic Design Series (HDS) and Hydraulic Engineering Circulars (HECs);
- Title 23, Code of Federal Regulations (CFR), Part 650, Subpart A;
- Federal-Aid Policy Guide;
- NCHRP and other TRB reports;
- Numerous hydrology and hydraulics reports and texts;
- Design, construction, and contract administration experience.

Where a conflict of information, procedures, or methodologies may arise, the engineer is expected to examine all pertinent parameters and use best judgment and sound reasoning to determine which approach is the most consistent with the design intent of the principles herein. Specific references are included in each chapter or section as applicable.

1.12 Acknowledgments

The MDOT SHA graciously acknowledges all the individuals and organizations that contributed to the development of the 2023 update of the Highway Drainage Manual. Principal authors include Ryan Doheny and his predecessor Stephen Buckley, in addition to consultant contributors Daniel Plantholt, Steve Phillips, Charles Wallace, Joan Wang, Matt Abel, and Michelle Moir. The authors also thank Dana Havlik for her support and the numerous reviewers who helped shape the publication.

1.13 Updates

The Office of Highway Development will issue periodic updates to this Manual. Users are encouraged to subscribe to updates here: https://roads.maryland.gov/mdotsha/pages/Index.aspx?PageId=38.

Additional information, assistance or questions may be obtained from:

Maryland Department of Transportation (MDOT)
State Highway Administration (SHA)
Office of Highway Development
Highway Hydraulics Division
707 North Calvert Street
Mailstop C-128
Baltimore MD 21202
888-320-9346
hhd@mdot.maryland.gov

1.14 DESIGN AIDS

Table 1-2: Abbreviations & Acronyms

ABBREVIATION/ ACRONYM	MEANING
AASCD	Anne Arundel Soil Conservation District
AASHTO	American Association of State Highway and Transportation Officials
BMP	Best Management Practice
CBCA	Chesapeake Bay Critical Area
CEM	Coastal Engineering Manual
CFS	Cubic Feet per Second
CFR	Code of Federal Regulations
COMAR	Code of Maryland Regulations
CP_V	Channel Protection Volume
DNR	Department of Natural Resources
EO	Executive Orders
EPA	Environmental Protection Agency
ESC	Erosion and Sediment Control
ESD	Environmental Site Design
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FWPCA	Federal Water Pollution Control Act
FWS	Fish and Wildlife Service
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
HHD	Highway Hydraulics Division
HIRE	Highways in the River Environment
HW	Headwater
HYG	Hydrograph
IDF	Intensity Duration Frequency
LDP	Land Development Project
LID	Low Impact Development
LOD	Limit of Disturbance
MDE	Maryland Department of Environment
MDOT	Maryland Department of Transportation or the “Department”
MDTA	Maryland Transportation Authority
MHW	Mean High Water
MLW	Mean Low Water
MS4	Municipal Separate Storm Sewer System
MSL	Mean Sea Level
NAS	National Academy of Sciences

Table 1-2: Abbreviations & Acronyms (continued)

ABBREVIATION/ ACRONYM	MEANING
NAVD	North America Vertical Datum
NCHRP	National Cooperative Highway Research Program
NEH	National Engineering Handbook
NEPA	National Environmental Protection Act
NFIA	National Flood Insurance Act
NFIP	National Flood Insurance Program
NHS	National Highway System
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
NTIS	National Technical Information Service
OHD	Office of Highway Development
OLF	Overland Flow
OMT	Office of Materials Technology
OOC	Office of Construction
OOS	Office of Structures
PRD	Plan Review Division
RFP	Request for Proposal
ROW	Right-of-Way
SCD	Soil Conservation District
SCS	Soil Conservation Service (former name of the NRCS)
SHA	State Highway Administration
SSM	Soil Stabilization Matting
SWM	Stormwater Management
SWMR	Stormwater Management Regulations
SWPPP	Stormwater Pollution Prevention Plan
TMDL	Total Maximum Daily Load
TR	Technical Release
TRB	Transportation Research Board
TW	Tailwater
USBR	United States Bureau of Reclamation
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WQ_V	Water Quality Volume

Table 1-3: Symbols

SYMBOL	DEFINITION	UNITS
A	Area of cross section of flow	ft^2
A	Watershed area	ft^2 or ac
A_g	Clear opening area of grate	ft^2
a	Depth of depression	in
AHW	Allowable HW	ft
B	Barrel width (pipe)	in or ft
B, b	Bottom width (channel)	ft
C	Runoff coefficient or coefficient	—
C_o	Orifice coefficient	—
C_W	Weir coefficient	—
D	Culvert diameter or barrel height	in or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
D_{50}, d_{50}	Median diameter of riprap, or median grain size	in
E	Specific energy	ft
E	Efficiency of an inlet	percent (%)
E_o	Ratio of frontal flow to total gutter flow (Q_w/Q)	—
Fr	Froude number	—
g	Acceleration due to gravity	ft/s^2
H	Sum of $H_E + H_f + H_o$	ft
H	Head loss	ft
H_b	Bend headloss	ft
H_E	Entrance headloss	ft
h	Height of curb-opening inlet	ft
h	Stage (water surface height)	ft
hl	Head loss	ft
H_f	Friction headloss	ft
H_L	Total energy losses	ft
H_o	Outlet or exit headloss	ft
H_v	Velocity headloss	ft
h_o	Hydraulic grade line height above outlet invert	ft
HW	Headwater depth (subscript indicates section)	ft
i	Rainfall intensity	in/h
K	Coefficient	—
K_M	Adjusted loss coefficient	—
K	Conveyance capacity	cfs (ft^3/s)
k_m	Contraction or Expansion loss coefficient	—
K_E	Entrance loss coefficient	—
L	Length of culvert	ft
L	Channel reach length	ft
*L	Length of curb-opening inlet or grate inlet	ft

Table 1-3: Symbols (continued)

SYMBOL	DEFINITION	UNITS
L	Pipe length	ft
L	Length of runoff travel	ft
n	Manning's roughness coefficient	—
P	Wetted perimeter	ft
P	Perimeter of grate opening, neglecting bars and side against curb	ft
Q, q	Rate of discharge	cfs (ft^3/s)
Q_b	Bypass flow	cfs (ft^3/s)
Q_i	Intercepted flow	cfs (ft^3/s)
Q_s	Gutter capacity above the depressed section	cfs (ft^3/s)
Q_T	Total flow	cfs (ft^3/s)
Q_w	Gutter capacity in the depressed section	cfs (ft^3/s)
R	Hydraulic radius (A/P)	ft
R_f	Ratio of frontal flow intercepted to total frontal flow	—
R_s	Ratio of side flow intercepted to total side flow	—
S	Slope of culvert	ft/ft
S or S_x	Pavement cross slope	ft/ft
S	Energy gradeline slope	ft/ft
S or S_L	Longitudinal slope of pavement	ft/ft
S_e	Equivalent cross slope	ft/ft
S_w	Depressed section slope	ft/ft
T	Channel top width	ft
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
v_c	Critical velocity of flow	ft/s
V_d	Mean velocity of flow in downstream channel	ft/s
V_o	Mean velocity of flow at culvert outlet	ft/s
V_u	Mean velocity of flow in upstream channel	ft/s
Y, y	Depth of flow	ft
y_c	Critical depth	ft
z	Elevation head	ft
z	Horizontal distance	ft
γ	Unit weight of water	lb/ft^3
τ	Shear stress (Tractive force)	lb/ft^2
τ_p	Permissible shear stress	lb/ft^2
α	Velocity distribution coefficient	—
θ	Channel slope angle	° (degrees)

2

HYDROLOGY

2.1 OVERVIEW

2.1.1 Introduction

Hydrology is generally defined as a science that addresses the interrelationship between water on and under the surface of earth and in the atmosphere. For this Manual, hydrology will address estimating flow magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in ft^3/s and hydrographs as discharge per time. For structures that are designed to control the volume of runoff (e.g., detention storage facilities) or where flow routing through culverts is used, the entire discharge hydrograph will be of interest. Beyond the peak flow rate, drainage considerations often require the need for information such as volumetric runoff, flow duration, and low flow analyses.

The design of drainage related facilities will typically conform to criteria in this manual. Roadways and other projects in undeveloped areas should meet all criteria. However, for projects in more urbanized areas, meeting the criteria may not be practicable due to existing constraints such as utilities, right-of-way limitations, budgetary limits, and situations where the project would be connected to drainage facilities not meeting current criteria. The design should be appropriate to the type of project and surroundings. When the design standards are not met, a design exception request must be approved by the Highway Hydraulics Division Chief.

Hydrologic methodologies generate models that approximate real-world systems and the design guidance in this chapter ensure that drainage systems and facilities are neither under-designed nor over-designed. The highway project hydrologic and hydraulic design will emerge after evaluating drainage basin characteristics, obtaining rainfall data, and utilizing acceptable methodologies.

The practices described here define the framework in which designers can operate effectively, taking into consideration different aspects of hydrologic and hydraulic design.

2.1.2 Symbols and Definitions

Symbols and variable definitions are provided where an equation or procedure is introduced. No attempt has been made to standardize terms in equations; e.g. USGS variables and terms are used for their procedures and NRCS variables and terms are used in their procedures.

2.1.3 Definitions

Following are discussions of concepts that will be important in a hydrologic analysis. These concepts will be used throughout this chapter to address different aspects of hydrologic studies.

Antecedent Moisture Conditions. Antecedent moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event. Notably, they affect the peak discharge only in the lower range of flood magnitudes, below approximately the 15-year event threshold. As storm magnitude increases, antecedent moisture has a rapidly decreasing influence on runoff.

Depression Storage. Depression storage is the natural depressions within a watershed that store runoff. Generally, after the depression storage is filled, runoff will commence.

Drainage Area. The drainage area to a point will include all the land area which drains to that point due to natural topography or man-made conduits and structures.

Frequency. Frequency is the number of times a flood of a given magnitude can be expected to occur on average over a long period of time. Frequency analysis is then the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in any year. Exceedance probability is “one” divided by the return interval, expressed as a percent. This is further discussed in Section 2.2.6.

Hydraulic Roughness. Hydraulic roughness is a composite of the physical characteristics that influence the flow of water across the earth’s surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel and the channel storage characteristics.

Hydrograph. The hydrograph is a graph of the time distribution of runoff from a watershed.

Hyetographs. The hyetograph is a graph of the time distribution of rainfall over a watershed.

Infiltration. Infiltration is a complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.

Interception. Storage of rainfall on foliage and other intercepting surfaces during a rainfall event is called interception storage.

Lag Time. The time from the centroid of rainfall to the peak of the runoff hydrograph.

Peak Discharge. The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.

Rainfall Excess. The rainfall excess is the water available for surface runoff (overland flow) after interception, depression storage, and infiltration have been satisfied.

Stage. The stage of a river is the elevation of the water surface above some elevation datum.

Stationarity. An assumption indicating statistical parameters are independent of time. Non-stationary hydrology implies that the average precipitation and resultant flood magnitudes change over time.

Time of Concentration. The time of concentration is defined as the interval of time required for the flow at the point of investigation to become a maximum. For homogeneous drainage areas, the time of concentration is the time it takes the drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the outlet. For heterogeneous drainage areas, the flow path needs to be selected to be the most hydrologically representative of the drainage area. Time of concentration is further discussed in Section 2.5.

Unit Hydrograph. A unit hydrograph is the direct runoff hydrograph resulting from a rainfall event that has a specific temporal and spatial distribution, lasts for a specific duration, and has unit volume (or results from a unit depth of rainfall). The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area. When a unit hydrograph is shown with units of ft^3/s, it is implied that the ordinates are cubic feet per second per inch of direct runoff. Hydrographs are further discussed in Section 2.8.

For a more complete discussion of these concepts and others related to hydrologic analysis, the reader is referred to the NRCS National Engineering Handbook Part 630 - Hydrology *(NRCS, 2020)* and the FHWA HDS-2 *(FHWA, 2002)*.

2.2 GENERAL CHARACTERISTICS

2.2.1 Factors Affecting Floods

The hydrologic analysis for a certain location or drainage conveyance must recognize the many variable factors that affect floods. Factors that must be considered for every hydrologic analysis include:

- rainfall total,
- rainfall temporal distribution,
- drainage area size, shape, and orientation,
- ground cover,
- soil type,
- slopes of terrain,
- characteristics of drainage conveyances within the drainage area,
- antecedent moisture condition,
- storage potential (e.g., overbank, ponds, wetlands, reservoirs, embankments, and channel),
- watershed development potential,
- potential hazard to life or infrastructure (e.g. from dams), and
- non-stationarity (climate change and land use/cover changes).

2.2.2 Sources of Information

The information available and appropriate for use in hydrologic analyses will vary from project to project, dependent on location, project scope, and the needs / purpose of the hydrologic analysis. The designer is responsible for obtaining up-to-date information for use in their hydrologic analysis. Suggested sources of information are provided in Sections 2.4 and 2.5 of this Manual.

2.2.3 Site Data

Hydrologic considerations can influence the selection of a highway corridor and the alternative routes within the corridor. Therefore, good hydrological practice should undertake studies and investigations, including consideration of the environmental and ecological impact of the project. Also, sensitive locations may require special studies and investigations. The magnitude and complexity of these studies should be proportionate with the importance and magnitude of the project and the problems encountered. Typical data to be included in surveys or studies are topographic maps, aerial photographs, precipitation records, streamflow records, historical high-water elevations, historical flood discharges, and locations of hydraulic features (e.g., reservoirs, water resource projects, designated or regulatory floodplain areas).

2.2.4 Evaluation of Runoff Factors

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

- Drainage basin characteristics including size, shape, slope, urbanization, land use, geology, soil type, surface infiltration, and storage.
- Stream channel characteristics including geometry and configuration, natural and artificial controls, channel roughness, channel modification, aggradation/degradation, and ice/debris.
- Floodplain characteristics.
- Meteorological characteristics such as precipitation amounts and temporal distribution.

2.2.5 Flood History

All hydrologic analyses should consider the flood history of the area. Designers should evaluate drainage complaints and documented nuisance flooding and may contact the Highway Hydraulics Division for historical drainage investigation records.

2.2.6 Storm Return Frequency

Hydrology analysis is based around the estimates of rainfall that can be expected for individual rain events. These events are categorized based on a statistical probability of their occurrence within each year. This “Annual Exceedance Probability” is the likelihood that a storm event of that magnitude will happen in any given year and is expressed in a percentage. Therefore, a 4% storm means that there is a 4% chance that a storm with at least that much rainfall will happen within a 12-month window.

These Annual Exceedance Probabilities are more frequently referred to by their inverse, in which case the categorization is named the “Return Frequency” or “Return Period”. In this case, that same 4% storm would be referred to as a “25-year storm”. With this nomenclature, the meaning is still the same:

There is a 4% chance that a storm with at least that much rainfall will happen in any year. The Return Frequency is not a guarantee that a storm will happen (that there will be a 25-year storm once every 25 years), nor is it intended to indicate that there is a period of safety after a storm (that if a 25-year storm happened 2 years ago, there won't be another one for 23 years). It is solely another way to describe the likelihood that a storm will happen within a given year. This Manual, and MDOT SHA in general, uses the Return Frequency categorization.

2.2.7 Economics

Hydrologic analyses for MDOT SHA projects will include the use of multiple storm return frequencies for use in the hydraulic design of various aspects of the project. These return frequencies are used to size different drainage facilities considering both risk of damage and construction cost.

2.3 DESIGN CRITERIA

2.3.1 Analysis Methodologies

The appropriate hydrologic analysis methodology depends on the application. For MDOT SHA projects, the following methodology are considered appropriate:

- Rational Method for storm drain analysis and design.
- Rational Method or NRCS methodologies are acceptable for roadside channels / ditches.
- NRCS Methodologies for natural (e.g., stream) channels.
- NRCS Methodologies for stormwater management analysis and design.
- NRCS Methodologies for culvert analysis and design.
- Structures with drainage areas greater than 400 acres and small structures per Section 4.1.2 should be coordinated with the Office of Structures, Structure Hydrology and Hydraulics Division. *(MDOT SHA OOS, 2020)*.

There may be reasons for the use of other methodologies in a particular case. Use of methodologies outside of these conditions must be justified in the project documentation. Generally, analysis methodologies should not be mixed in the same application, but in some cases this may be the most appropriate or only option. In such cases, the designer should use sound engineering judgment to ensure that the methodologies are compatible and thoroughly documented.

2.3.2 Design Return Period

- Channels: See Chapter 3 Channels
- Culverts: See Chapter 4 Culverts
- Storm Drains: See Chapter 5 Storm Drain Systems
- Stormwater Management: See Maryland Stormwater Design Manual *(MDE, 2009)*
- Erosion and Sediment Controls: See Maryland Standards and Specifications for Soil Erosion and Sediment Control *(MDE, 2011)*

- Dam Breach: See MDE Dam Safety Program's Guidance for Completing a Dam Breach Analysis for Small Ponds and Dams in Maryland *(MDE, 2018)*
- Waterway Construction: See Maryland Waterway Construction Guidelines *(MDE, 2000)*
- FEMA Flood Risk Mapping: See FEMA's Guidelines and Standards for Flood Risk Analysis and Mapping Activities under the Risk Mapping, Assessment, and Planning Program

2.3.3 Rainfall Data

MDOT SHA uses the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 Point Precipitation Frequency Estimates for development of project hydrology *(NOAA, 2006)*. Atlas 14 rainfall data is available for the entire state of Maryland from the NOAA Precipitation Frequency Data Server (PFDS) and is available in both Precipitation Intensity and Precipitation Depth values. It is therefore usable with both the Rational Method and NRCS Method discussed in this chapter. Designers should obtain both sets of data for use in project development.

Atlas 14 rainfall data should be obtained for a location within the project limits or a representative location within the appropriate Maryland county. This location should be documented by address or latitude / longitude within the Drainage Report for verification purposes.

On some projects, it may be necessary to account for additional rainfall amounts to enhance the resiliency of certain MDOT SHA facilities. See Section 2.9 of this Manual for guidance on the application of Projected Rainfall Ratios for these facilities.

2.4 DRAINAGE AREA DEVELOPMENT

2.4.1 Drainage Area / Points of Investigation

A drainage area, also known as a watershed, is all the land area that drains to a chosen point. This chosen point is sometimes referred to as the Point of Investigation. The drainage area is determined by delineating drainage divides (often ridge lines) which separate drainage areas. Delineation is typically performed using Computer-Aided Design (CAD) or Geographic Information System (GIS) software.

Drainage areas are frequently divided into subareas or Hydrologic Units. Subareas are used to divide a larger drainage area into areas of similar land use, or to investigate the area to an existing or proposed hydraulic control feature such as a bridge, culvert, or pond, or to divide areas at the junction of two or more waterways. Ideally, subareas should be delineated so that they contain relatively homogeneous characteristics such as land use, land cover, and soil type.

2.4.2 Hydrologic Soil Groups

Soil types for hydrologic analysis are divided into four Hydrologic Soil Groups:

Group A

Soils having high infiltration rates even when thoroughly wetted, consisting chiefly of deep, well to excessively drained sands and/or gravels. These soils have a high rate of water transmission and would result in a low runoff potential.

Group B

Soils having moderate infiltration rates when thoroughly wetted, consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These Soils have a moderate rate of water transmission and a moderate runoff potential.

Group C

Soils having a slow infiltration rate when thoroughly wetted, consisting of (1) soils with a layer that impedes the downward movement of water, or (2) soils with moderately fine to fine texture and a slow infiltration rate. These soils have a slow rate of water transmission and a high runoff potential.

Group D

Soils having very slow infiltration rates when thoroughly wetted, consisting chiefly of (1) clay soils with a high swelling potential; (2) soils with a high permanent water table; (3) soils with claypan or clay layer near the surface; and (4) shallow soils over nearly impervious materials. These soils have a very slow rate of water transmission and a very high runoff potential.

Soil group information must be obtained for the full extents of the drainage area. Up to date soil group information may be downloaded from the NRCS Web Soil Survey: https://websoilsurvey.nrcs.usda.gov. In some situations, the NRCS Soil Survey may indicate that an area is a mix of soil groups, such as “B/D”. In these cases, designers should use the soil group that represents a higher runoff potential. Soil data may also be obtained from other sources but should be checked against the current Web Soil Survey results. More detailed definitions of the Hydrologic Soil Groups are provided in the NRCS National Engineering Handbook, Part 630 Hydrology, Chapter 7 *(NRCS, 2009)*.

2.4.3 Land Use

The type of vegetation or development on an area of land is the key factor in the potential for runoff from that area. Land use delineation must be completed for the full extents of the drainage area.

For MDOT SHA projects, land use delineation into single cover types, such as Impervious, Open Space, Woods, or Meadow, is required within MDOT SHA right-of-way, and recommended everywhere else within the project drainage areas. Composite land use categories may be used for areas of very consistent off-site development, comprised of only impervious and open areas, which are not to be changed by the project, but the designer should ensure that the assumed composite land use type is appropriate for the development of the area by checking the impervious percentage of a representative section of the area. Composite land uses should not be used for areas that include significant areas of other cover, such as woods or farmland. All undeveloped areas within a project’s limit of disturbance (LOD) that are outside of the existing maintained roadway areas must be considered to be Woods (if wooded) or Meadow (all other undeveloped cover) in existing conditions.

Delineation of existing land use should use the most current information available. Sources such as aerial photos, up-to-date GIS information, and survey information should be combined and verified by field inspections. Sources such as online street view mapping can also be a good tool for selection of existing cover. Proposed condition land use delineation should reflect all changes made as a result of the project but should otherwise remain unchanged from existing conditions.

Ultimate land use analysis may be required for specific purposes in project development. Ultimate land use will frequently use zoning maps from Counties and municipalities and other local jurisdictions, supplemented by designer judgment about existing development patterns, and be delineated with composite land use categories.

2.4.4 Drainage Area-Land Use-Soil Group Breakdown

The goal of the delineations of drainage areas, Hydrologic Soil Group, and land use is to allow the breakdown of the drainage area by areas of common land use and soil group which will be used to determine the runoff potential of the area via C factor or Runoff Curve Number (RCN). See Sections 2.7.5 and 2.8.2.

2.5 TIME OF CONCENTRATION

2.5.1 General

The Time of Concentration (Tc) is the time required for runoff from all parts of a drainage area / watershed to contribute to the peak discharge at the POI for the drainage area. For a homogeneous drainage area, this represents the time taken for runoff from the hydraulically most distant point in the watershed to the outlet, the point with the longest travel time. This point may not necessarily be the point with longest flow distance. This time of concentration path must be representative of the majority of drainage area. The minimum time of concentration is 5 minutes for rational method calculations and 0.1 hours (6 minutes) for NRCS methodology.

2.5.2 Methodology

MDOT SHA projects should use the NRCS Velocity Method for determining Tc values for both Rational Method and NRCS methodology computations. While there are many methodologies that have been developed for estimating the Time of Concentration, MDOT SHA wants to encourage consistency within computation methodology. The Velocity Method is described in full detail in the NRCS National Engineering Handbook, Part 630 Hydrology, Chapter 15 – Time of Concentration *(NRCS, 2010)*, as well as in the NRCS TR-55, Urban Hydrology for Small Watersheds, Chapter 3 *(NRCS, 1986)*. This methodology consists of breaking the Tc flow path into segments of similar flow characteristics within three regimes: Sheet Flow, Shallow Concentrated Flow, and Channel Flow. Travel time through each of these segments is computed, and the sum total of all the travel time of these segments is the Time of Concentration. Time of concentration paths for subareas will end within the subarea and should not continue to any downstream point. If translation of the hydrograph is necessary to a downstream POI, it must be computed with Reach Routing (see Section 2.8.6)

2.5.3 Sheet Flow

Sheet flow is defined by NRCS as “flow over plane surfaces”. Sheet flow occurs at the upper areas of a watershed, near boundaries. Sheet flow segments of the time of concentration path for a drainage area should be representative of the drainage area. Since sheet flow typically occurs for lengths no more than 100 feet *(USDA/NRCS, 2001)*, MDOT SHA projects should limit sheet flow to no more than 100 feet.

For additional guidance in determining the lengths of sheet flow segments, Equation 2-1 from NEH Chapter 15 can provide a limiting length of sheet flow (*l*) based on slope (*S*) and Manning's roughness coefficient (*n*) for sheet flow. The use of Equation 2.1 is intended to assist in design but is not required.

$$\boxed{l = \frac{100\sqrt{S}}{n}}$$

(Eq. 2.1)
Sheet Flow Length

Manning's Roughness Coefficients for sheet flow include both the effects of roughness but also obstacles applicable to flow depths of less than 0.1 feet. These roughness coefficients are not interchangeable with coefficients for channel flow. Roughness coefficients for sheet flow can be found in Table 15-1 of NEH Chapter 15.

Travel time for Sheet Flow segments is computed using a simplified version of the Manning's kinematic equation:

$$\boxed{T_t = \frac{0.007(nl)^{0.8}}{(P_2)^{0.5}S^{0.4}}}$$

(Eq. 2.2)
Kinematic Wave Equation
for Sheet Flow Travel Time

Where:

T_t = travel time (hr)

n = Manning's roughness coefficient for sheet flow

l = sheet flow length (ft)

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

S = slope of land surface (ft/ft)

2.5.4 Shallow Concentrated Flow

Shallow Concentrated Flow is flow in swales, rills, and gullies that are assumed to not have a well-defined / regular channel shape. Flow depths for shallow concentrated flow range between 0.1 and 0.5 feet. Gutter flow is also frequently considered shallow concentrated flow. Velocities for shallow concentrated flow are developed using Figure 15-4 and Table 15-3 of the NEH Part 630 Chapter 15 *(NRCS, 2010)*. Designers should be consistent with velocities on similar slopes and ground types.

Travel time for shallow concentrated flow segments is computed with the basic travel time equation:

$$\boxed{T_t = \frac{l}{3600V}}$$

(Eq. 2.3)
Travel Time

Where:

T_t = travel time (hr)

l = flow segment length (ft)

V = average velocity of flow (ft/s)

2.5.5 Channel Flow

Beyond the point where flow depths exceed the 0.5 ft depth of shallow concentrated flow, channel flow is assumed to occur. Channel flow segments should be selected based on similar flow characteristics, and separate segments should be established for changes in flow. These changes can include switching from open channel to pipe flow, or along the length of a long channel if the channel section changes. For channel flow segments, Manning's equation (Eq. 3.5) or water surface profile information can be used to estimate average flow velocities. Manning's "n" values for channel flow should be developed per Section 3.8.3. When the average velocity of flow is obtained, Equation 2.3 will be used to determine the travel time of the segment.

2.6 SUMMARY OF HYDROLOGIC ANALYSIS

2.6.1 General

Many hydrologic analysis methods are available, though only two, the Rational Method and NRCS Unit Hydrograph Method, are generally applicable to the watershed sizes covered by this manual. Procedures for applying the methods are found in Sections 2.7 and 2.8. If possible, the method should be calibrated to local conditions and tested for accuracy and reliability.

These hydrologic methods can be divided in two broad types of deterministic and statistical methods:

- Deterministic. Deterministic methods are based on fundamental concepts such as runoff potential, drainage area size, and time of concentration. These methods often require significant judgment and experience to be used effectively but leave room for adjustments that are often necessary when analyzing small watersheds. The Rational and Unit Hydrograph methods are examples of deterministic methods.

- Statistical. These methods are usually well-documented mathematical procedures that are applied to measured or observed data and typically do not require as much judgment and experience to apply as deterministic methods. The accuracy ranges of statistical methods can also be measured quantitatively. However, statistical methods may not be well understood and, as a result, answers may be misinterpreted. These methodologies are also less accurate over the relatively small drainage areas analyzed for MDOT SHA projects. Regression equations and gaged data analysis are examples of statistical methods.

2.6.2 Overview

Selection of Rational Method versus Unit Hydrograph method typically considers watershed size, type of hydraulic structure, and availability of data, among others. Section 2.3.1 describes the appropriate methodology for use in most situations found in MDOT SHA projects.

- See Section 2.7 for further information on the Rational Method.
- See Section 2.8 for further information on the NRCS and other unit hydrograph methods.

The following methods are more appropriate for drainage areas greater than 400 acres, and therefore outside the scope of this manual. Refer to the Office of Structures (OOS) “Manual for Hydrologic and Hydraulic Design” *(MDOT SHA OOS, 2020)* and the MD Hydrology Panel Report *(MD Hydrology Panel, 2020)* for additional information:

- State watershed regression equations are range specified in the regression analysis study and in accordance with the other limitations of the regression equations, unless there are stream gage data or historical evidence suggesting other alternatives. Regression equation confidence limits can be used to resolve significant differences between other methods.
- Flow distribution statistical methods (e.g., log-Pearson Type III analyses) are desirable for designs on drainage basins at or near streamflow gaging stations, provided that there is at least 10 years of a continuous or synthesized record. Estimates should be limited to twice the years of record (e.g., 25 years of data are needed for 50-year discharge estimate).

2.6.3 Software for Hydrologic Analysis

While most of these hydrologic analysis methods were developed prior to the widespread use of computers, and therefore are based on hand calculation or graphical analysis procedures, computer programs have been developed and are considered the standard for analysis. See Section 1.10 Software for more information.

2.6.4 Hydrologic Accuracy

The accuracy of the hydrologic estimates will have a major effect on the design of drainage or flood control facilities. The sensitivity of results to various parameters should be tested as part of any analysis. Although it might be argued that one hydrologic procedure is more accurate than another, practice has shown that all of the methods discussed in this chapter can, when used with good engineering judgment, produce acceptable results consistent with observed or measured events. Designers should take care to ensure that the results of their analysis are consistent with expectations and historical observations.

2.6.5 Calibration

Calibration of a hydrologic model may occasionally be necessary in watershed analysis, such as considering available historic or anecdotal data to achieve a more realistic peak-discharge estimate.

2.7 RATIONAL METHOD

2.7.1 Introduction

The Rational Method is generally useable for estimating the design storm peak runoff for areas up to 200 acres, however for the purposes of MDOT SHA project, the use of Rational Method for estimation of runoff shall be limited to drainage areas less than 50 acres, consistent with the provisions of Sections 2.3.1 and 2.6.2. The procedures presented in this section are based on HDS-2 *(FHWA, 2002)*. The various tables and charts referenced can be found in Section 2.11.

2.7.2 Application

Some precautions should be considered when applying the Rational Method:

- The Rational Method analysis will result in a single peak runoff amount in cubic feet per second (cfs). The designer should be aware of this limitation when adding multiple drainage areas which may have different times of concentration or a travel time downstream of the point of investigation.
- The first step in applying the Rational Method is to define the boundaries of the drainage area(s) in question. See Section 2.4 Drainage Area Development. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.
- In determining surface characteristics for the drainage area, consider any future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system.
- Restrictions to the natural flow (e.g., highway crossings and dams that exist in the drainage area) should be investigated to determine how they might affect the design flows.
- The charts, graphs, and tables included in this section are not intended to replace reasonable and prudent engineering judgment that should permeate each step in the design process.

2.7.3 Characteristics

Characteristics of the Rational Method that limit its effective use to 200 acres include:

a. <u>Rainfall Intensity vs. Time of Concentration</u>. 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.

b. <u>Uniform Rainfall Intensity</u>. The Rational Method assumes that rainfall intensity is uniform over the entire watershed.

c. <u>Peak Discharge Frequency</u>. The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

d. <u>Runoff</u>. The fraction of rainfall that becomes runoff (*C*) is independent of rainfall intensity or volume.

e. Peak Rate Only. The Rational Method will only provide a peak rate of runoff, as opposed to runoff over time.

2.7.4 Equation

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

$$\boxed{Q = CiA}$$

(Eq. 2.4)
Rational Method Equation

Where:

Q = maximum rate of runoff, ft^3/s

C = runoff coefficient representing a ratio of runoff to rainfall

i = average rainfall intensity for a duration equal to the time of concentration for a selected return period, in/h

A = drainage area tributary to the design location, acres

The results of using the Rational Method to estimate peak discharges are very sensitive to the parameters used, especially time of concentration and runoff coefficient. The hydraulics engineer should use good engineering judgment in estimating values that are used in the Rational Method. Following is a discussion of the variables used in the Rational Method.

2.7.5 Runoff Coefficient

The runoff coefficient (C) is the variable of the Rational Method least amenable to precise determination and requires the judgment and understanding of the hydraulic designer. Although engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters. See Table 2-1 or coefficients for use with the Rational Method.

The runoff coefficient shall be selected for portions of the drainage area on the basis of the land use and soil type, as determined via the methodology described in Section 2.4.4 Drainage Area – Land use – Soil Group Breakdown. Where heterogeneous areas or surfaces are encountered, a weighted value of the runoff coefficient shall be used. In the determination of the weighted "C" value, each parcel of area shall be assigned a “C”; value based upon soil type, slope, and ground cover / land use.

Justifications and assumptions for the hydrologic condition of land uses should be consistently applied throughout a project. The designer may choose to use a single slope condition for a project analysis (for instance, all 0-2% slopes for a project on the Eastern Shore or all 2%-6% slopes for a project in Central Maryland), provided that this is consistently applied for all analyzed areas and conditions (existing and proposed).

2.7.6 Rainfall Intensity

The rainfall intensity (i) is the average rainfall rate (in/hr) for a duration equal to the time of concentration (subject to the minimum time of concentration) for a selected return period. Once the return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Intensity-Duration-Frequency (IDF) curves.

To determine the maximum discharge from a watershed, for a given storm frequency and duration, the designer should use the rainfall intensity for which the drainage area will yield the greatest peak discharge. This is assumed to occur when the duration of the storm equals the time of concentration. The basic intensity for this storm is estimated using NOAA Atlas 14 data.

NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: MD

See Section 2.3.3 for guidance on selecting and documenting the location of rainfall data.

When the time of concentration is between points on the rainfall data tables, interpolate the rainfall intensity value from the two adjacent data table points. Alternatively, a graphical representation of the IDF curve may be used to select values. The designer should take care to use consistent rainfall intensity values and precision whether determined by interpolation or by curves.

Further background in the use of the data server is available in NOAA Atlas 14 Vol. 2 https://www.weather.gov/media/owp/oh/hdsc/docs/Atlas14_Volume2.pdf *(NOAA, 2006).*

2.8 NRCS HYDROGRAPH METHODOLOGIES

For the Natural Resources Conservation Service (NRCS) formerly known as the Soil Conservation Service (SCS) methods - The basic technical references for the SCS methods are the National Engineering Handbook - Section 4 - Hydrology, and the SCS Engineering Field Manual.

The detailed procedures for using TR-20 may be found in the NRCS Technical Release No. 20 "Computer Program for Project Formulation – Hydrology" *(NRCS TR-20, 1992)*. The standard program for TR-20 analysis is WinTR-20 available directly from NRCS. All TR-20 analysis should use the latest version of WinTR-20 and should not use older versions of WinTR-20 nor DOS versions.

While the 1986 DOS version of TR-55 used regression equations to determine peak discharges, WinTR-55 is actually an input/output interface which runs WinTR-20 in the background to generate, route, and add hydrographs. It is suggested that designers use WinTR-20 rather than WinTR-55 for more complex analyses.

The NRCS Hydrograph Methodologies have been developed to provide an estimation of the rainfall over time throughout the entirety of a storm, called a hydrograph. These hydrographs provide the ability to add multiple watersheds and account for travel time delays and storage effects, as well as split flows to multiple points of investigation.

2.8.1 Rainfall Distributions

NOAA Atlas 14 Precipitation values must be used with NRCS Regional Rainfall Distributions for the Ohio Valley and Neighboring States. These rainfall distributions will be NOAA Type B, Type C, or Type

D, depending on the location within Maryland. TR-55 / TR-20 Type II Rainfall Distributions are not compatible with NOAA Atlas 14 rainfall.

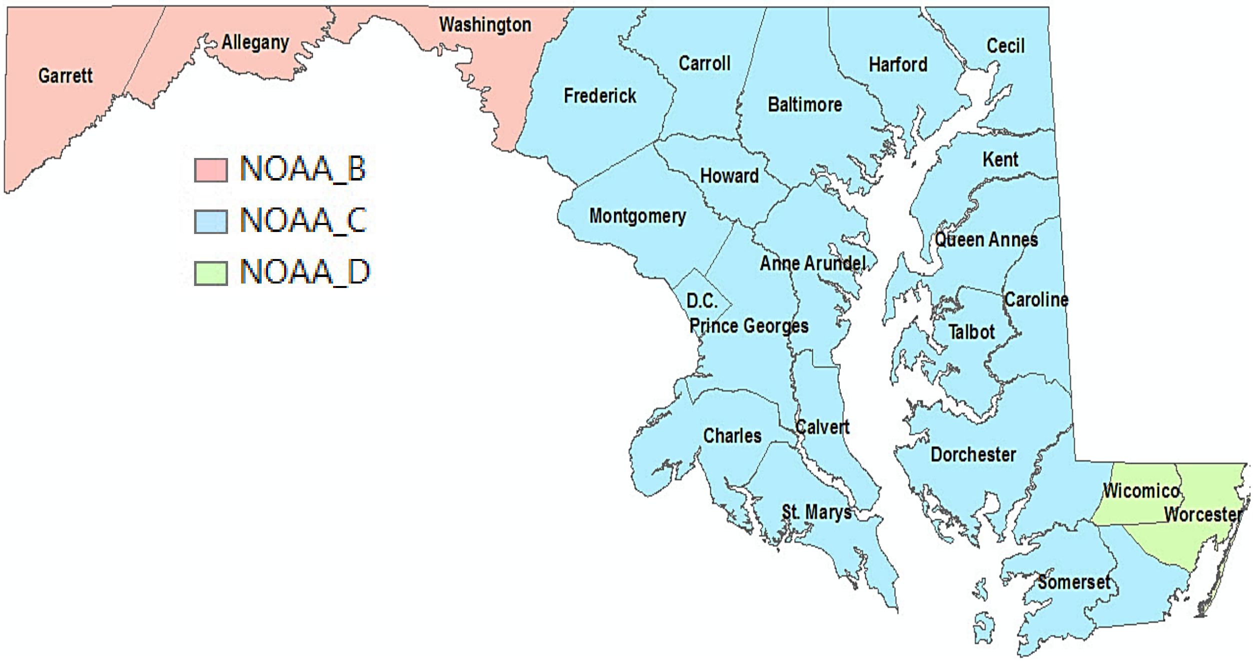

Figure 2-1: Regional Rainfall Distributions in Maryland

2.8.2 Dimensionless Unit Hydrograph

The dimensionless unit hydrograph, as defined in Section 2.1.3, is one of several watershed-related parameters that are incorporated into the NRCS hydrologic modeling process. The Standard Unit Hydrograph applies to all watersheds in Maryland outside of the Eastern Shore. The Delmarva Unit Hydrograph was developed for watersheds that are characterized by flat topography and applies to all watersheds on Maryland's Eastern Shore.

2.8.3 Runoff Depth Estimation

The volume of storm runoff can depend on a number of factors. The volume of rainfall will be the most important factor. Additionally, the volume of runoff from one storm event is affected by rainfall that occurred during previous storm events. This is accounted for by the use of the Antecedent Moisture Condition (AMC), also referred to as the Antecedent Runoff Condition (ARC). AMC Type II (average conditions) should be used for all projects in Maryland.

A common assumption in hydrologic modeling is that the rainfall available for runoff is separated into three parts – initial abstraction, direct (or storm) runoff, and losses. Initial abstraction is the initial fraction of the storm rainfall prior to the beginning of runoff which is lost to processes such as infiltration, interception, evaporation, and surface depression storage. Factors that affect the split between losses and direct runoff include the volume of rainfall, land cover and use, soil type, and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage.

2.8.4 Cover Complex Classification and Runoff Curve Number

Tables 2-2 and 2-3 show the NRCS CN values for the different land uses, treatments, and hydrologic condition; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. Justifications for the hydrologic condition of land cover should be consistently applied throughout a project.

2.8.5 Estimation of CN Values for Urban Land Uses

Table 2-2 includes CN values for a number of composite urban land uses. For each of these, the CN is based on a specific percentage of imperviousness, with the remaining area considered as open space. For example, the CN values for commercial land use are based on an imperviousness of 85 percent. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus, pervious curve number, $CN_{p,}$ values of 39, 61, 74, and 80 are used for hydrologic soil groups A, B, C, and D, respectively. The following equation can be used to compute a weighted curve number, CN_w:

$$\boxed{CN_w = CN_p\,(1 - f) + f(98)}$$

(Eq. 2.5)
Weighted CN

in which *f* is the fraction (not percentage) of imperviousness. To show the use of Equation 2.5, the CN values for commercial land use with 85 percent imperviousness are:

A soil: 39(0.15) + 0.85(98) = 89

B soil: 61(0.15) + 0.85(98) = 92

C soil: 74(0.15) + 0.85(98) = 94

D soil: 80(0.15) + 0.85(98) = 95

These are the same values shown in Table 2-2.

2.8.6 Urban Impervious Area Modifications

Although the NRCS hydrograph methodologies include adjustment factors for use with connected or unconnected impervious areas, these adjustment factors are not used for MDOT SHA projects.

2.8.7 Rainfall-Runoff Equation

A relationship between accumulated rainfall and accumulated runoff was derived by NRCS from experimental plots for numerous soils and vegetative cover conditions. Data for land-treatment measures (e.g., contouring, terracing) from experimental watersheds were included.

The equation was developed mainly for small watersheds for which only daily rainfall and watershed data 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 NRCS runoff equation is therefore a method of estimating direct runoff from a 24-h or 1-d storm rainfall.

The equation is:

$$Q = (P - I_a)^2 / ((P - I_a) + S)$$

(Eq. 2.6)
Rainfall-Runoff Equation

Where:

Q	=	accumulated direct runoff, in.
P	=	accumulated rainfall (potential maximum runoff), in.
I_a	=	initial abstraction including surface storage, interception and infiltration prior to runoff, in.
S	=	potential maximum retention, in., S = (1000/CN) - 10

The relationship between I_a and S was developed from experimental watershed data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the NRCS runoff equation is:

$$I_a = 0.2S$$

(Eq. 2.7)
Initial Abstraction Equation

Substituting 0.2S for I_a in Eq. 2.6, the NRCS rainfall-runoff equation becomes:

$$Q = (P - 0.2S)^2 \;/\; (P + 0.8S)$$

(Eq. 2.8)
NRCS Simplified Rainfall-Runoff

2.8.8 Additional Possible Computations in NRCS Methodology

The runoff-over-time nature of the NRCS hydrograph methodologies allow for the inclusion of computations for the manipulation of hydrographs for the purposes of modeling specific situations encountered within the watershed. The designer should include these computations as necessary to accurately model the watershed.

Reach Routing – Reach routing allows for a hydrograph to be translated downstream to the point where it can be added to another hydrograph. A representative cross section is used to model the flow characteristics of a channel, and multiple reaches should be used if the cross section changes as it progresses through the watershed. The designer should use reach routing downstream of a subarea when the travel time from the analysis point of the subarea to the point of addition to another hydrograph will affect the resulting hydrograph addition.

Storage / Basin Routing – Storage routing allows for a hydrograph to be attenuated by a storage area with a release, such as a pond or a culvert. Storage routing requires the development of stage (elevation) – storage and stage – discharge ratings for the storage area.

Split Routing – Split routing allows for the splitting of a hydrograph into two hydrographs, which can be used to model a flow condition where runoff may go to two different locations, such as an undersized roadway culvert where upstream overflow will continue along the roadway to another culvert, or a pond where a principal spillway is directed to one outfall while an emergency spillway flows to another outfall.

2.9 Projecting Rainfall Data for Climate Change

2.9.1 Overview

With the changing climate, historical rainfall patterns may not always be a reasonable assumption to analyze drainage infrastructure that will service roadways well into the future. As our local climate changes, new risks may arise, including increases in precipitation intensity and frequency *(IPCC, 2014).* Engineers should take these risks into consideration when designing drainage infrastructure to mitigate risks of future flooding and other drainage-related issues on Maryland's roadways.

One means to account for these risks on a project is to use projected rainfall data when performing hydrological analyses. Future rainfall data can be estimated by applying projected rainfall ratios to existing rainfall datasets. The ratios represent the change between modeled future rainfalls and the historical baseline precipitation data. These ratios are often generated using multiple global climate models. A wide array of models are used in developing the ratios for a particular emission scenario as it generally provides a more accurate representation than a single model would on its own. Using models with varying degrees of sensitivity ensures that the projections reflect the uncertainty of the climate's response to external factors *(Kilgore, 2019).*

The use of multiple models in the development of these ratios is similar to using multiple models in projecting hurricane paths. A single model may be more sensitive to a particular contributing factor, such as water temperature when compared to other models and that increased sensitivity may affect the model projections. However, when multiple models are evaluated collectively, they generally provide a more accurate representation of the hurricane path as each individual model's sensitivity helps create the confidence limits for the collective projection. For a more detailed discussion on how these ratios are developed, see references "Applying Climate Change Information to Hydrologic and Coastal Design of Transportation Infrastructure" *(Kilgore, 2019)* and "Projected Intensity-Duration-Frequency (IDF) Curve Tool for the Chesapeake Bay Watershed and Virginia *(Miro, 2021)*.

2.9.2 Projected Rainfall Ratios

A. Projected IDF Curve Data Tool

Projected rainfall ratios (also referred to as change factors) can be obtained using the Mid-Atlantic Projected IDF Curve Data Tool and are also presented in Table 2-4. This resource is based on multiple downscaled global climate model datasets and was created by a group of researchers from Carnegie Mellon University, the Northeast Regional Climate Center at Cornell University, and the RAND Corporation. Historical and projected IDF curves were derived for each dataset and then projected rainfall ratios were calculated using the derived curves. This tool presents those projected rainfall ratios on a county-wide basis for different return periods, emissions scenarios, and projection time periods.

Emissions scenarios attempt to anticipate human behavior, socio-economic conditions, and public policy decisions. Several emissions scenarios are identified in the Intergovernmental Panel on Climate Change (IPCC) Fifth Assessment Report (AR5) Synthesis Report. These emissions scenarios include Representative Concentration Pathway (RCP) 2.6, RCP 4.5, RCP 6.0, and RCP 8.5, with the lower numbers corresponding to lower emissions and greenhouse gas concentrations. The RCP 2.6 scenario requires emissions to start declining around 2020 until a net zero is reached by 2100, RCP 4.5 and 6.0

are intermediate scenarios, and RCP 8.5 assumes emissions continue to rise throughout the century. For MDOT SHA projects described in Section 2.9.3, the Representative Concentration Pathway (RCP) 8.5 emissions scenario should be used.

Many of the drainage systems installed within MDOT SHA right-of-way have an expected service life that can well exceed 50 years. For example, a new drainage system installed in 2020 with a 50-year service life is intended to last until 2070 before rehabilitation or replacement is needed. To ensure that the drainage networks have adequate capacity for the entirety of their service life, a late century projection time period should be used. The 2050 – 2100 projection time period should be used for MDOT SHA projects to ensure that critical drainage networks continue to function well into the future.

Because projected rainfall ratios are developed by using multiple models, the final projections often result in the engineer having a median value ratio with a range of possible deviation. The median value ratios should be used on MDOT SHA projects.

B. Application of Ratios in Design

Once obtained the projected rainfall ratios can be applied to a particular storms rainfall by multiplying the projection ratio with the storm events rainfall *(Miro, 2021).*

$$\boxed{Rainfall\ Depth * Projected\ Rainfall\ Ratio = Projected\ Rainfall\ Depth}$$

(Eq. 2.9) Projected Rainfall Ratio

The ratios may also be used with NOAA Atlas 14 rainfall intensities. Projected rainfall ratios are storm event dependent, ratios for the 10-year storm should not be applied to rainfalls from the 50-year storm.

Once obtained, the projected rainfalls can be used in the same manner as Atlas 14 data is in other sections of this manual. Note that where full IDF curves are used in the design, the projected rainfall ratio should be applied to the entire curve and not only the 24-hr rainfalls.

C. Limitations and Future Developments

It’s recommended that the projected rainfall ratios only be applied to rainfall data that was developed using the same historical time period *(Miro, 2021)*. NOAA Atlas 14 Volume 2 generally uses a 1950 – 2000 historical time period which is similar to that used by the Mid-Atlantic Projected IDF Curve Data Tool recommended in Section 2.9.2.A. Therefore, the projected rainfall ratios developed using this tool should only be applied to the NOAA Atlas 14 Volume 2 data.

Climate change science is evolving at a rapid rate and the models used to project rainfall will become more accurate over time. NOAA Atlas 15 is under development and is anticipated to account for non-stationarity and include future precipitation information. Historical precipitation data will also eventually be updated for Maryland to include an expanded time series in NOAA Atlas 14 Volume 13. MDOT SHA will continue to monitor the latest developments and update its guidance on the subject accordingly.

2.9.3 Project Application

The use of projected rainfall data in the design of permanent drainage infrastructure is only required along FEMA hurricane evacuation routes and recommended in areas with vulnerable assets or limited adaptive capacity. Vulnerable assets include those susceptible to sea level rise, riverine flooding, scour, or other disruptive flooding events. The MDOT SHA Climate Change Vulnerability Viewer is an ArcGIS Online web application that showcases a variety of climate related data layers to help transportation professionals assess roadway vulnerability. Adaptive capacity is a system's ability to cope with existing climate variability and future climate impacts. Areas with limited adaptive capacity include roadways that are the only means of emergency vehicle access for communities and those that would result in a significant detour length if closed due to flooding. Permanent drainage infrastructure includes features such as open channels, roadway culverts, and storm drains. This process does not apply to the design of Stormwater Management Facilities or Sediment Control practices. Stormwater Management Facilities should continue to be designed in accordance with guiding principles set forth by the Maryland Department of the Environment and the Plan Review Division of MDOT SHA.

The application of projected rainfall data should typically be consistent on a project. When projected rainfall data is used on a project, it should be used to design new drainage infrastructure and evaluate existing drainage infrastructure.

Existing drainage infrastructure will often be undersized when analyzed with projected rainfall data. Situations will also arise where increasing the capacity of existing drainage infrastructure results in challenges such as utility conflicts or stormwater management requirements. Designers must consider the project's objective statement (or purpose and need) in addition to short and long-term system performance, mobility, and safety goals when justifying their design decisions in the Drainage Report (Section 1.9). For example, quantitative performance measures may be identified early on in project development that require drainage infrastructure to provide a certain service life or target a specific flooding problem. To meet these performance measures, the designer may consider replacing and redesigning the existing drainage infrastructure using projected rainfall data. Alternatively, if the existing drainage infrastructure is in good condition and there are no flooding issues identified during planning or design, it may be prudent to maintain the existing infrastructure, even if there would be nominal hydraulic performance gains when analyzed using projected rainfall data. In these situations, future projects would presumably address the performance improvements when the drainage infrastructure approaches or exceeds the end of its service life.

Only projects located along FEMA hurricane evacuation routes require a design exception approved by the Highway Hydraulics Division Chief when projected rainfall data is not used. Projects located elsewhere in the state should only document design decisions and justifications in the Drainage Report.

2.10 REFERENCES

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2.11 DESIGN AIDS

Chapter 2 Design Aids include the following Tables, Figures and Charts

- Table 2-1 Rational Formula Coefficients for SCS Hydrologic Soil Groups (A, B, C, D)
- Table 2-2 NRCS Runoff Curve Numbers - Agricultural Land Uses
- Table 2-3 NRCS Runoff Curve Numbers - Fully Developed Urban Area (vegetation established)
- Table 2-4 NOAA Atlas 14 Projected Rainfall Ratios/Change Factors

Table 2-1: Rational Formula Coefficients for SCS Hydrologic Soil Groups (A, B, C, D) – Formerly SHA 6.1.1.410.0

PART 1 RURAL LAND USES
Top Box - Storm Frequencies of Less Than 25 Years
Bottom Box - Storm Frequencies of 25 Years or Greater

LAND USE	TREATMENT/ PRACTICE	HYDROLOGIC CONDITION	A			B			C			D		
			0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%
Pasture or range		poor	0.23	0.25	0.26	0.31	0.33	0.34	0.37	0.38	0.39	0.40	0.41	0.42
			0.27	0.29	0.31	0.36	0.37	0.39	0.42	0.43	0.44	0.45	0.46	0.47
		fair	0.12	0.13	0.15	0.24	0.25	0.27	0.31	0.33	0.34	0.36	0.37	0.38
			0.15	0.17	0.19	0.28	0.30	0.32	0.36	0.37	0.39	0.40	0.41	0.43
		good	0.07	0.09	0.10	0.18	0.20	0.22	0.27	0.29	0.31	0.32	0.34	0.35
			0.09	0.11	0.13	0.22	0.24	0.26	0.32	0.33	0.35	0.37	0.38	0.40
	Contoured	poor	0.11	0.12	0.14	0.22	0.24	0.26	0.33	0.34	0.36	0.39	0.40	0.41
			0.13	0.16	0.18	0.26	0.28	0.30	0.37	0.39	0.40	0.44	0.45	0.46
		fair	0.06	0.07	0.08	0.17	0.19	0.21	0.28	0.30	0.31	0.35	0.36	0.37
			0.07	0.08	0.10	0.21	0.23	0.25	0.32	0.34	0.36	0.39	0.41	0.42
		good	0.03	0.04	0.06	0.11	0.12	0.14	0.24	0.26	0.28	0.31	0.33	0.34
			0.05	0.06	0.08	0.13	0.14	0.15	0.28	0.30	0.32	0.36	0.37	0.39
Meadow		-	0.06	0.08	0.10	0.10	0.14	0.19	0.12	0.17	0.22	0.15	0.20	0.25
			0.08	0.11	0.14	0.13	0.18	0.22	0.16	0.20	0.26	0.21	0.25	0.32
Wooded		poor	0.10	0.11	0.13	0.13	0.15	0.20	0.16	0.18	0.25	0.18	0.22	0.26
			0.12	0.14	0.16	0.16	0.19	0.23	0.19	0.23	0.28	0.22	0.27	0.33
		fair	0.06	0.08	0.09	0.10	0.13	0.18	0.11	0.15	0.20	0.13	0.18	0.23
			0.08	0.10	0.12	0.13	0.17	0.21	0.15	0.18	0.24	0.18	0.22	0.29
		good	0.05	0.07	0.08	0.08	0.11	0.15	0.10	0.13	0.17	0.12	0.15	0.21
			0.06	0.09	0.11	0.11	0.15	0.18	0.13	0.17	0.21	0.15	0.19	0.25

PART 2 AGRICULTURAL LAND USES Top Box - Storm Frequencies of Less Than 25 Years Bottom Box - Storm Frequencies of 25 Years or Greater														
LAND USE	**TREATMENT/ PRACTICE**	**HYDROLOGIC CONDITION**	**A**			**B**			**C**			**D**		
			0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%
Fallow		-	0.41	0.48	0.53	0.60	0.66	0.71	0.72	0.78	0.82	0.84	0.88	0.91
			0.57	0.64	0.69	0.70	0.76	0.80	0.83	0.88	0.91	0.95	0.97	0.98
Row Crops	Straight Row	poor	0.31	0.36	0.39	0.54	0.58	0.62	0.70	0.74	0.77	0.75	0.78	0.80
			0.45	0.50	0.54	0.65	0.70	0.73	0.82	0.86	0.88	0.86	0.88	0.89
		good	0.24	0.30	0.35	0.43	0.48	0.52	0.61	0.65	0.68	0.73	0.76	0.78
			0.38	0.44	0.49	0.60	0.64	0.67	0.75	0.77	0.79	0.83	0.85	0.86
	Contoured	poor	0.28	0.34	0.39	0.51	0.55	0.59	0.61	0.65	0.68	0.70	0.74	0.77
			0.43	0.48	0.52	0.64	0.68	0.71	0.73	0.76	0.78	0.84	0.86	0.88
		good	0.21	0.26	0.30	0.41	0.45	0.49	0.55	0.59	0.63	0.63	0.66	0.68
			0.33	0.38	0.42	0.56	0.60	0.64	0.69	0.72	0.74	0.74	0.76	0.77
	Contoured and Terraced	poor	0.26	0.30	0.34	0.38	0.42	0.46	0.50	0.54	0.57	0.56	0.59	0.61
			0.38	0.42	0.46	0.52	0.57	0.62	0.66	0.70	0.74	0.69	0.72	0.74
		good	0.20	0.24	0.27	0.31	0.35	0.39	0.45	0.48	0.51	0.55	0.58	0.60
			0.34	0.37	0.40	0.45	0.49	0.53	0.61	0.64	0.67	0.68	0.70	0.72
Small Grain	Straight Row	poor	0.24	0.28	0,32	0.43	0.47	0.51	0.62	0.65	0.68	0.72	0.74	0.76
			0.37	0.40	0.43	0.59	0.63	0,66	0.73	0.76	0.78	0.84	0.86	0.87
		good	0.23	0.26	0.29	0.42	0.45	0.48	0.57	0.60	0.62	0.71	0.73	0.75
			0.35	0,38	0.41	0.57	0.60	0.63	0.70	0.73	0.75	0.83	0.85	0.86

PART 2 AGRICULTURAL LAND USES Top Box - Storm Frequencies of Less Than 25 Years Bottom Box - Storm Frequencies of 25 Years or Greater														
LAND USE	**TREATMENT/ PRACTICE**	**HYDROLOGIC CONDITION**	**A**			**B**			**C**			**D**		
			0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%
Small Grain (cont.)	Contoured	poor	0.21	0.26	0.30	0.38	0.42	0.46	0.55	0.59	0.62	0.63	0.65	0.67
			0.33	0.38	0.42	0.53	0.57	0.61	0.69	0.72	0.75	0.75	0.77	0.78
		good	0.17	0.22	0.27	0.33	0.38	0.42	0.54	0.58	0.61	0.62	0.65	0.67
			0.29	0.34	0.38	0.50	0.54	0.58	0.67	0.70	0.73	0.74	0.76	0.77
	Contoured and Terraced	poor	0.18	0.22	0.26	0.32	0.36	0.40	0.52	0.55	0.58	0.56	0.59	0.61
			0.30	0.34	0.37	0.46	0.50	0.53	0.65	0.68	0.71	0.70	0.72	0.73
		good	0.16	0.20	0.24	0.31	0.35	0.38	0.45	0.48	0.50	0.55	0.58	0.60
			0.28	0.32	0.35	0.44	0.48	0.51	0.62	0.64	0.66	0.68	0.70	0.71
Close seeded legumes or Rotation Meadow	Straight Row	poor	0.25	0.30	0.35	0.44	0.48	0.52	0.62	0.65	0.68	0.73	0.76	0.78
			0.37	0.42	0.46	0.60	0.64	0.67	0.74	0.77	0.80	0.83	0.85	0.86
		good	0.15	0.19	0.23	0.31	0.35	0.38	0.55	0.58	0.60	0.63	0.65	0.66
			0.20	0.24	0.28	0.47	0.50	0.53	0.67	0.70	0.72	0.75	0.77	0.78
	Contoured	poor	0.23	0.28	0.32	0.41	0.45	0.49	0.57	0.60	0.63	0.62	0.65	0.67
			0.35	0.40	0.44	0.56	0.60	0.63	0.70	0.73	0.76	0.74	0.77	0.79
		good	0.14	0.18	0.21	0.30	0.34	0.37	0.45	0.48	0.51	0.58	0.60	0.61
			0.24	0.28	0.31	0.42	0.46	0.49	0.61	0.64	0.66	0.71	0.73	0.74
	Contoured and Terraced	poor	0.21	0.26	0.30	0.34	0.38	0.42	0.51	0.54	0.57	0.58	0.60	0.61
			0.33	0.38	0.42	0.50	0.54	0.57	0.67	0.70	0.72	0.71	0.73	0.74
		good	0.07	0.10	0.13	0.28	0.32	0.35	0.44	0.47	0.49	0.52	0.54	0.56
			0.20	0.24	0.28	0.40	0.44	0.47	0.61	0.63	0.65	0.68	0.70	0.71

PART 3 URBAN LAND USE Top Box - Storm Frequencies of Less Than 25 Years Bottom Box - Storm Frequencies of 25 Years or Greater												
LAND USE	**A**			**B**			**C**			**D**		
	0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%	0-2%	2-6%	>6%
Paved Areas & Impervious Surfaces	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97
Open Space, Lawns, Etc.	0.08	0.12	0.15	0.11	0.16	0.21	0.14	0.19	0.24	0.20	0.24	0.28
	0.11	0.15	0.19	0.15	0.20	0.26	0.19	0.24	0.32	0.25	0.29	0.37
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
RESIDENTIAL												
Lot Size 1/8 acre	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Lot Size 1/4 acre	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Lot Size 1/3 acre	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Lot Size 1/2 acre	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Lot Size 1 acre	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46

Table 2-2: NRCS Runoff Curve Numbers – Fully Developed Urban Area (vegetation established)[1]
(Adapted from TR-55, Table 2-2a)

COVER DESCRIPTION			CURVE NUMBERS FOR HYDROLOGIC SOIL GROUPS			
COVER TYPE		AVERAGE PERCENT IMPERVIOUS AREA[1]	A	B	C	D
Open Space (Lawns, Parks, Golf Courses, Cemeteries, etc.)[3]:	Poor Condition (Grass Cover <50%)		68	79	86	89
	Fair Condition (Grass Cover 50% to 75%)		49	56	79	84
	Good Condition (Grass Cover > 75%)		39	61	74	80
Impervious Areas:	Paved Parking Lots, Roofs, Driveways, etc. (Excluding Right-of-Way)		98	98	98	98
Streets and Roads:	Paved; Curbs and Storm Drains (Excluding Right-of-Way)		98	98	98	98
	Paved; Open Ditches (Including Right-of-Way)		83	89	92	93
	Gravel (Including Right-of-Way)		76	85	89	91
	Dirt (Including Right-of-Way)		72	82	87	89
Urban Districts:	Commercial and Business	85%	89	92	94	95
	Industrial	72%	81	88	91	93
Residential Districts by Average Lot Size:	1/8 Acre or Less (Town Houses)	65%	77	85	90	92
	1/4 Acre	38%	61	75	83	87
	1/3 Acre	30%	57	72	81	86
	1/2 Acre	25%	54	70	80	85
	1 Acre	20%	51	68	79	84
	2 Acres	12%	46	65	77	82
Newly Graded Areas (Pervious Areas Only, No Vegetation)			77	86	91	94

[1]Average runoff condition, Ia = 0.2S
[2]The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.
[3]CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Table 2-3: NRCS Runoff Curve Numbers – Agriculture Land Uses[1]
(Adapted from TR-55, Tables 2-2b and 2-2c)

COVER DESCRIPTION			CURVE NUMBERS FOR HYDROLOGIC SOIL GROUPS			
COVER TYPE	TREATMENT[2]	HYDROLOGIC CONDITION[3]	A	B	C	D
Fallow	Bare soil	-	77	86	91	94
	Crop residue cover (CR	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight Row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small Grain	Straight Row (SR)	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured (C)	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured & terraced (C&T)	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadows	Straight Row (SR)	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured (C)	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured & terraced (C&T)	Poor	63	73	80	83
		Good	51	67	76	80

COVER DESCRIPTION			CURVE NUMBERS FOR HYDROLOGIC SOIL GROUPS			
COVER TYPE	**TREATMENT[2]**	**HYDROLOGIC CONDITION[3]**	**A**	**B**	**C**	**D**
Pasture or range - native or improved grassland reserved for grazing[4]		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
Meadow - continuous grass, protected from grazing and generally mowed for hay		-	30	58	71	78
Brush – brush-weed-grass mixture with brush as the major element[4]		Poor	48	67	77	83
		Fair	35	56	70	77
		Good	30	48	65	73
Woods[5]		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	30	55	70	77
Farmsteads – buildings, lanes, driveways, and surrounding lots		-	59	74	82	86

[1]Average runoff condition, Ia = 0.2S

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

[3]Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.
Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

[4]Poor: <50% ground cover
Fair: 50 to 75% ground cover
Good: > 75% ground cover

[5]Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 2-4: NOAA Atlas 14 Projected Rainfall Ratios/Change Factors
(Adapted from the Projected IDF Curve Data Tool for the Chesapeake Bay Watershed and Virginia)

County	Projected Rainfall Ratio (Median Value, 2050-2100 Time Period)											
	2-year		5-year		10-year		25-year		50-year		100-year	
	RCP 4.5	RCP 8.5	RCP 4.5	RCP 8.5	RCP 4.5	RCP 8.5	RCP 4.5	RCP 8.5	RCP 4.5	RCP 8.5	RCP 4.5	RCP 8.5
Allegany	1.14	1.19	1.15	1.21	1.17	1.21	1.21	1.22	1.24	1.22	1.13	1.21
Anne Arundel	1.14	1.15	1.16	1.14	1.17	1.14	1.17	1.15	1.17	1.18	1.09	1.19
Baltimore	1.15	1.15	1.15	1.14	1.16	1.14	1.19	1.15	1.21	1.17	1.10	1.19
Calvert	1.11	1.17	1.11	1.17	1.12	1.17	1.12	1.18	1.12	1.19	1.05	1.21
Caroline	1.13	1.18	1.13	1.18	1.12	1.19	1.14	1.19	1.20	1.19	1.12	1.18
Carroll	1.13	1.21	1.16	1.18	1.18	1.17	1.19	1.16	1.21	1.17	1.15	1.18
Cecil	1.16	1.20	1.13	1.18	1.11	1.17	1.09	1.17	1.07	1.18	1.08	1.16
Charles	1.12	1.16	1.12	1.18	1.12	1.18	1.13	1.19	1.15	1.20	1.10	1.21
Dorchester	1.12	1.14	1.11	1.14	1.10	1.15	1.11	1.14	1.14	1.14	1.07	1.13
Frederick	1.13	1.20	1.15	1.20	1.17	1.18	1.18	1.16	1.18	1.16	1.11	1.15
Garrett	1.15	1.15	1.16	1.15	1.16	1.15	1.17	1.14	1.15	1.13	1.12	1.13
Harford	1.16	1.17	1.16	1.16	1.16	1.16	1.15	1.17	1.14	1.17	1.10	1.18
Howard	1.15	1.17	1.18	1.16	1.19	1.15	1.20	1.14	1.19	1.14	1.09	1.13
Kent	1.14	1.18	1.11	1.18	1.08	1.18	1.09	1.18	1.08	1.19	1.07	1.18
Montgomery	1.15	1.15	1.17	1.15	1.19	1.15	1.19	1.15	1.20	1.17	1.10	1.17
Prince George's	1.12	1.15	1.14	1.15	1.15	1.16	1.16	1.17	1.18	1.18	1.11	1.19
Queen Anne's	1.13	1.19	1.11	1.18	1.09	1.18	1.10	1.18	1.12	1.18	1.08	1.17
Somerset	1.11	1.17	1.14	1.17	1.15	1.17	1.17	1.15	1.17	1.15	1.03	1.14
St. Mary's	1.09	1.19	1.09	1.18	1.08	1.17	1.10	1.16	1.10	1.15	1.04	1.15
Talbot	1.11	1.18	1.12	1.16	1.09	1.15	1.09	1.14	1.12	1.14	1.09	1.14
Washington	1.12	1.21	1.13	1.20	1.14	1.19	1.18	1.17	1.19	1.15	1.13	1.13
Wicomico	1.11	1.16	1.14	1.15	1.14	1.14	1.15	1.15	1.16	1.15	1.05	1.15
Worcester	1.12	1.17	1.12	1.15	1.12	1.16	1.11	1.17	1.11	1.17	1.06	1.18

3 CHANNELS

3.1 OVERVIEW

3.1.1 Introduction

The function of ditches and open channels is to safely convey stormwater runoff from, through, or around roadway rights-of-way without damage to the highway, to the channel itself, to adjacent property, or to other components of the highway system such as structures or utilities.

This chapter provides guidance for open channel design and is intended to be applied primarily to roadside and roadway facility channels carrying stormwater runoff. It is not a complete source of information for all channels, especially natural channels or those subject to base flow. Guidance for in-depth analysis and design of natural channels is found in other publications, such as MDOT SHA OOS Manual for Hydrologic and Hydraulic Design, Chapter 14 "Stream Morphology". Regulatory guidance for natural channels is provided by the Maryland Department of the Environment's Wetlands and Waterways Protection Program.

3.1.2 Channel Types

A. Roadside Channels

A roadside channel is defined as an open channel usually paralleling the highway embankment and within the limits of the highway right-of-way. It is normally trapezoidal (with a flat bottom width of 2 to 8 feet) or V-shaped in cross section and lined with grass or other protective linings. The alignment, cross section, and grade of roadside channels are usually constrained to a large extent by the geometric and safety standards applicable to the project.

The primary functions of roadside channels are to intercept offsite runoff before it reaches the highway, collect roadway surface runoff from the highway, and convey the accumulated runoff to acceptable outfall points in a safe and efficient manner. A secondary function of a roadside channel is to collect and drain subsurface water from the base of the roadway to prevent saturation and loss of support for the pavement or to provide a positive outlet for subsurface drainage systems (e.g., pipe underdrains). An additional function for some roadside channels is for stormwater management purposes. Roadside channel requirements must still be met in linear stormwater management facilities.

Examples of roadside channels are listed below:

- Side Ditches are provided to control runoff along the toe of fill. Side ditches may receive flow from the roadway facility, off-site areas, or both.
- Surface Drain Ditches are normally V-shaped ditches incorporated in the typical section through cut areas used to collect runoff from the roadway and slopes. However, when drainage from side ditches is carried into and through a cut, a flat bottom ditch must be used.
- Median Ditches are located in the center of divided highways and are often incorporated into the typical section.
- Outlet and Inlet Ditches either convey flow to or away from a culvert or storm drain system.
- Bench or Interceptor Ditches are provided to control runoff on slopes, by either collecting runoff from offsite areas before flowing down a cut slope (Interceptor), or collecting runoff generated on the slope before it creates significant erosion (Bench).

Flow characteristics (e.g. depth, velocity) in roadside channels are frequently controlled by other features, such as roadway culverts, driveway culverts, or stormwater management facilities. Water surface profiles may be developed by the designer to facilitate the review of complex or costly roadside channels and are often required for linear stormwater management facilities that also function as conveyance channels.

B. Stream Channels

In addition to providing runoff conveyance, natural stream channels are also sensitive ecological systems that require care to prevent disruption. As such, impacts and modifications to natural channels should be avoided when possible, and will require coordination with Local, State, and Federal agencies when any impacts or modifications are necessary. This can include additional discharge to a floodplain that then may create a new channel inflow point.

When analysis of natural channels is necessary, care must be taken to accurately model the natural channel shape and roughness coefficients. Variations in the channel characteristics must be incorporated into the analysis.

C. Dikes and Levees

Dikes and levees are highly regulated by State and Federal agencies. Refer to the appropriate agency for current requirements.

3.2 General Considerations

Channels must provide positive drainage and sufficient capacity to meet freeboard requirements for the design storm. Channel stability must be demonstrated for all channels that are either located within MDOT SHA right-of-way, impact MDOT SHA facilities, or convey flow from MDOT SHA facilities. Channel designs and the design of highway facilities that impact channels should satisfy all applicable FHWA policies on Federal-aid projects, including, but not necessarily limited to, those regarding wetland preservation/protection, floodplain management, and water quality preservation.

3.3 DESIGN CONSIDERATIONS

3.3.1 Location and Geometry

The location and geometry of roadside channels will typically be determined through close coordination between the roadway and water resources designer. The channel grade for roadside ditches need not follow the grade of the adjacent road, particularly if the road is flat. Avoid major breaks in channel longitudinal grade that may cause unnecessary scour or sediment deposition. Not only can the depth and width of the channel be varied to meet different quantities of runoff, longitudinal slopes, types of lining, and the distance between discharge points, but the lateral distance between the channel and the edge of pavement may also be varied.

The designer should ensure that the depths of roadside channels are compatible with the adjacent roadway. Coordination with the roadway or pavement designer may be necessary to determine the appropriate depth to adequately drain the subsurface pavement section or if other measures are necessary, such as longitudinal underdrain.

Roadside channels should be considered along curb and gutter sections where significant off-site area drains toward the roadway. This can help minimize the size and extent of on-site storm drain system that is required.

3.3.2 Clear Zones

A Clear Zone is an unobstructed, traversable roadside area that allows for the recovery of errant vehicles. Clear Zone criteria should be factored into the design and location of channels to minimize hazards to public safety. The MDOT SHA Guidelines for Traffic Barrier Placement and End Treatment Design provides additional information on the Clear Zone concept.

Safety of the general public is an important consideration in the selection of the cross-sectional geometries of drainage channels. Ensure that channel slope combinations are appropriate for the location/proximity to the roadway.

Transverse berms, such as check dams, within the clear zone should be limited to 6 inches in height and have 6H:1V or flatter slopes. Their effect on freeboard should be accounted for, as freeboard checks should determine flow depth/water surface elevation over the check dam.

Where practicable on high-speed roadways, roadside channels should be located so that the peak water surface elevation during passage of the design flow is outside the clear zone, unless a roadside barrier is provided.

3.3.3 Neighboring Properties

Channel design must avoid adverse impacts to adjacent properties. Roadside channels should be designed to have a conveyance capacity that is sufficient to ensure that they cause no increase in depth or frequency of flooding to properties outside the right-of-way without all applicable concurrence, right-of-way, or easements for the impact. In areas where new discharge points are created, right to discharge on downstream properties must be obtained during project development.

3.3.4 Channel Linings

All channels require some form of lining to promote vegetative establishment and/or to protect against erosion. Design channel linings according to Section 3.5, which references the FHWA Hydraulic Engineering Circular 15 (HEC-15) *(FHWA, 2005)* permissible tractive force (shear stress) approach. At a minimum, all newly graded channels will utilize type 'A' soil stabilization matting (SSM). One or more SSM will be specified by the designer for installation in channels, unless another material is necessary (e.g., riprap). The designer will calculate channel velocities and shear stress using the procedures of Section 3.5 to determine the most appropriate SSM. Consider the use of Type B or C SSM only where turfgrass can be established and the use of riprap or other hard channel linings are not desirable. Locations requiring immediate stabilization may use turfgrass sod establishment (sod), but channels that are subject to substantial flows during the establishment period should be protected by diversions or check dams to slow flow velocity.

Channel side slopes should not exceed the angle of repose of the soil or lining material, or both, and should be 2H:1V or flatter in the case of riprap lining. Concrete channel linings are discouraged as they require stormwater management treatment due to their imperviousness and are prone to failure, requiring extensive repairs.

Additional channel lining considerations are needed in areas featuring Karst topography. In these areas, dissolved carbonate bedrock can create sinkholes, sinking streams, caves, springs, and other types of groundwater flow which can be sensitive to changes in surface drainage patterns. The use of synthetic liners is a common way to mitigate the risk of sinkholes developing in Karst areas. Designers should coordinate with the Office of Materials Technology Engineering Geology Division to ensure properly engineered channels in Karst topography.

3.3.5 Stream Channels

Wherever possible, encroachment into streams should be avoided and encroachment onto floodplains should be minimized due to the regulatory challenges that coincide with stream impacts. Whenever stream channels must be relocated or otherwise modified, the extent of channel reach and degree of modification should be the minimum necessary to provide compatibility of the channel and roadway. Modifications to stream channels may need to incorporate aspects of natural channel design and/or stream restoration techniques. Stream restorations require a thorough analysis of the stream's morphology and environment.

3.3.6 Cut and Fill Slopes

Steep slopes are susceptible to rill and gully erosion. Provide channels along the tops of cuts or embankments exceeding 20 feet in height in order to reduce risk of slope erosion. Benching with additional channels is required for slopes whenever the height of the slope exceeds 10*h*, where the slope is expressed as *h*:1, consistent with MDE *Standards and Specifications for Soil Erosion and Sediment Control* Detail B-3-1**.** For embankments/cut slopes steeper than 2:1 or exceeding 10 feet in height, contact the Office of Materials Technology Engineering Geology Division to coordinate an evaluation of slope stability. In areas where the groundwater table or perched water level is above the toe of cut slope, drainage blankets are recommended. Drainage blanket details can be obtained from the Office of Materials Technology Engineering Geology Division.

3.3.7 Maintenance

The design of channels should consider the frequency and type of maintenance expected and make allowance for maintenance access. Vegetation management is an important part of channel maintenance. Boom mowers are often utilized along roadside slopes. In some areas, other equipment such as riding mowers may be necessary. Slopes 4:1 or flatter are generally easier for most mowing equipment to traverse. Standing water should be avoided within channels to promote healthy vegetation and facilitate maintenance efforts.

Sedimentation, or siltation, is another potential maintenance concern. Flat or sluggish channels can result in the settling of soils that can accumulate and result in blockages of a channel over time. Where practicable, roadside drainage channels should be designed to have a self-cleaning velocity for the design storm frequencies outlined in Section 3.4.

3.4 DESIGN FREQUENCY

Design open channels to collect and convey without damage, and to confine within the ditch, stormwater flow with standard design frequencies as follows:

Table 3-1: Design Storm Frequencies of Open Channels

TYPE OF CHANNEL	FREQUENCY	MINIMUM FREEBOARD
Roadside, Median, and Interceptor ditches or swales	10-year	9 inches below edge of shoulder in open section or curb flow line in closed section
Roadside Channels that function as outfalls (e.g., pipe or SWM outfalls)	Match culvert or SWM Facility (10-year minimum)	9 inches below edge of shoulder
Temporary roadside and median ditches or swales	2-year	9 inches below edge of shoulder
Channels Away from / Not Adjacent to Roadways	Match upstream pipe or channel (10-year minimum)	6 inches from the top of channel embankment

Site-specific factors may warrant the use of an atypical design frequency. Increasing discharges, water surface elevations, or introducing new concentrated discharges onto adjacent property owners will require the acquisition of easements, right-of-way, or right of discharge and must be coordinated with MDOT SHA Office of Real Estate in project development.

3.5 CHANNEL LININGS

3.5.1 Overview

All MDOT SHA channels must be stable and design storm velocities must be non-erosive. Hydraulic conditions in a drainage channel can become erosive even at mild highway grades. As a result, these channels often require additional stabilization against erosion. HEC-15 *(FHWA, 2005)* is the basic reference for selection and design of linings for roadside channels. The riprap design procedures in HEC-15 are appropriate for constructed channels that have a uniform cross section.

The HEC-15 design methodology for the evaluation of channel linings is based on shear stress and considers permissible shear stresses on the lining and the erodibility of the underlying soil. Shear stress is the tractive force caused by water flowing in the channel. Shear stress is normally analyzed for a 10-year flood event. A channel is unstable where the flow-induced shear stress exceeds the permissible shear stress of the channel lining material. Table 3-2 provides permissible shear stress values for design. Velocities have been included in Table 3-2 but are provided for comparative reference purposes only. Design should be performed using shear stress.

The FHWA Hydraulic Toolbox software package includes a channel lining analysis module and can be used to compute shear stress in channels and develop appropriate linings. Care must be taken to use appropriate input values to ensure the resulting lining design is stable.

3.5.2 Vegetative Linings

Vegetative channel linings work in interaction with the underlying soil to resist erosion. As such, the properties of the soil are a key factor in determining the permissible shear stress. Designers should ensure that soil characteristics assumed for design approximate the actual soils in the project area. Either assume non-cohesive soils or perform testing on available topsoil during design to determine cohesive or non-cohesive soils, Plasticity Index, and Soil Classification (e.g. GC, SC).

3.5.3 Stone Linings

Stone linings include riprap and gabion baskets. Well-graded stone is placed on filter fabric to form a mass of stone with a minimum of voids. Riprap stone is generally angular with shapes that allow stones to interlock, hard, and durable, resistant to disintegration from chemical and physical weathering. The underlying filter fabric should have adequate permeability to prevent uplift pressures.

3.5.4 Rigid Linings

Rigid linings, such as concrete, can be useful in non-uniform flow conditions and situations where shear stress exceeds the permissible values of common linings in Table 3-2. However, rigid linings are highly susceptible to foundation instability, leading to failure of the lining. Areas where designers believe a rigid lining to be appropriate should be discussed with MDOT SHA HHD.

Table 3-2: Permissible Shear Stress and Recommended Allowable Velocity for Channels

TYPE OF LINING	SHEAR STRESS - τ_p (lb/ft^2)	RECOMMENDED ALLOWABLE VELOCITY (ft/s)
Established Vegetation (Class C Vegetation)	1.00	5-6
Type A SSM (Curled Wood Matting)	1.55	5.0
Type B SSM (Synthetic Matting)	2.00	7.0
Type C SSM (Turf Reinforcement Matting)	4.00	8.5
Type D SSM (Jute Net)	0.45 (prior to establishment) 1.00 (after establishment)	5.0 (after establishment)
Type E SSM (Degradable straw/coconut blend fibers)	1.5	5.0

TYPE OF LINING	SHEAR STRESS - τ_p (lb/ft^2)	RECOMMENDED ALLOWABLE VELOCITY (ft/s)
Class I Riprap	3.0	8
Class II Riprap	4.0	10.5
Gabions	4.0	10+
Gravel (D_{50}=2 in)	0.8	6
Bedrock	No maximum established	No maximum established

Source: Shear Stress adapted from FHWA-RD-89-199, 1989.

3.6 CHANNEL ANALYSIS AND DESIGN

3.6.1 Open Channel Flow Concepts

This section provides a summary of hydraulic terms and concepts that are basic to the understanding of open channel flow. For further discussion, consult Open Channel Hydraulics *(Chow, 1959)* and FHWA, HDS-4 *(FHWA, 2008)*.

3.6.2 Hydraulic Head

Hydraulic Head is the energy possessed by a unit weight of water at any specific point. The Hydraulic Head is comprised of three parts:

- Elevation Head: the height of water above a reference datum
- Pressure Head: the pressure of the water referenced to atmospheric pressure
- Velocity Head: the kinetic energy of the water

In open channel flow, Pressure Head is assumed to be zero for the surface of the flowing water. Water will flow from areas of higher Hydraulic Head to areas of lower Hydraulic Head, down the hydraulic gradient.

3.7 SPECIFIC ENERGY

Specific energy, E, is defined as the energy (head) relative to the channel bottom at the point of analysis. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head.

$$\boxed{E = y + a\left(\frac{v^2}{2g}\right)}$$

(Eq. 3.1)
Specific Energy Equation

Where:

E = Specific Energy, ft

y = depth, ft

a = velocity distribution coefficient (1 for uniform channels)

v = mean velocity, ft/s

g = gravitational acceleration, 32.2 ft/s^2

The velocity distribution coefficient is taken to have a value of one for turbulent flow in prismatic channels but may be significantly different than one in natural channels.

A. Energy Gradeline

The total head is the specific energy head plus the elevation of the channel bottom with respect to the reference datum. The line joining the total head from one cross section to the next defines the energy gradeline.

B. Steady and Unsteady Flow

Steady flow is an unchanging discharge over the time period of interest. Unsteady flow is a discharge that varies over time.

C. Uniform Flow and Non-Uniform Flow

Uniform flow is a condition where the velocity and depth of flow do not change. Uniform flow can only occur in a prismatic channel with a constant cross section, roughness, and slope. Non-uniform flow is the condition in which the velocity and/or depth vary along the channel. Non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

D. Gradually Varied and Rapidly Varied Flow

A non-uniform flow in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected is referred to as a gradually varied flow; otherwise, it is considered to be rapidly varied.

E. Flow Classification

The classification of open-channel flow can be summarized as follows:

Steady Flow

- Uniform flow
- Non-uniform Flow
    - Gradually varied flow
    - Rapidly varied flow

Unsteady Flow

- Unsteady uniform flow (rare)
- Unsteady non-uniform flow
    - Gradually varied unsteady flow
    - Rapidly varied unsteady flow

The steady, uniform flow case and the steady, non-uniform flow case are the most fundamental types of flow treated in highway engineering hydraulics.

F. Froude Number

The Froude number, F_r, represents the ratio of inertial forces to gravitational forces and is defined by:

$$\boxed{F_r = \frac{v}{[gd]^{1/2}}}$$

(Eq. 3.2)
Froude Number

Where:

v = mean velocity, ft/s

g = acceleration of gravity, 32.2 ft/s^2

d = hydraulic depth (hydraulic mean depth for non-rectangular sections), ft

This expression for Froude number applies to any open channel or channel subsection with uniform or gradually varied flow. For rectangular channels, the hydraulic depth is equal to the flow depth.

The Froude number indicates whether flow is supercritical, critical (F_r = 1), or subcritical. Values of F_r greater than 1 represent supercritical flow and values less than 1 represent subcritical flow.

G. Critical Flow

Critical flow occurs when the specific energy is a minimum for a given discharge in a channel cross sections. The depth at which the specific energy is a minimum is called critical depth. At critical depth, the Froude number has a value of 1. Critical depth is also the depth of maximum discharge when the specific energy is held constant. During critical flow, the velocity head is equal to half the hydraulic depth. The general expression for flow at critical depth is:

$$\boxed{\frac{aQ^2}{g} = \frac{A^3}{T}}$$

(Eq. 3.3)
Critical Flow

Where:

a = velocity distribution coefficient

Q = total discharge, ft^3/s

g = gravitational acceleration, 32.2 ft/s^2

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

T = channel top width at the water surface, ft

When flow is at critical depth, Eq. 3.3 must be satisfied, no matter what the shape of the channel.

H. Subcritical Flow

The flow depth is greater than critical depth in subcritical flow, and the Froude number is less than one. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.

I. Supercritical Flow

The flow depth is less than critical depth in supercritical flow, and the Froude number is greater than one. Small water surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.

J. Hydraulic Jump

A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, a hydraulic jump is sometimes employed to dissipate energy and control erosion at highway drainage structures.

3.7.1 Open Channel Flow Equations

The design and analysis of both natural and constructed channels proceed according to the basic principles of open-channel flow (see Chow, 1959 and Henderson, 1966). The basic principles of fluid mechanics (e.g., continuity, momentum, energy) can be applied to open-channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the primary objectives of open-channel flow analysis.

The following equations are those most commonly used to analyze open channel flow.

A. Continuity Equation

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of one-dimensional, steady flow of an incompressible fluid, it assumes the simple form:

$$\boxed{Q = A_1 V_1 = A_2 V_2}$$

(Eq. 3.4)
Continuity Equation

Where:

Q = discharge, ft^3/s

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

V = mean cross-sectional velocity, ft/s (which is perpendicular to the cross section)

The subscripts 1 and 2 refer to successive cross sections along the flow path.

B. Manning's Equation

For a given depth of flow in an open channel with a steady, uniform flow, the mean velocity, V, can be computed with Manning's equation.

$$\boxed{V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2}}$$

(Eq. 3.5)
Manning's Equation

Where:

V	=	velocity, ft/s
n	=	Manning's roughness coefficient
R	=	hydraulic radius = A/P, ft
P	=	wetted perimeter, ft
S	=	slope of the energy gradeline, ft/ft (Note: For steady uniform flow, *S* = channel slope, ft/ft)

The selection of Manning's "n" is generally based on observation; however, considerable experience is essential in selecting appropriate "n" values. The selection of Manning's "n" is discussed in Section 3.8.3. The range of "n" values for various types of channels and floodplains, as well as recommended values for design, is shown in Table 3-3. For roadside channel applications, the hydraulics engineer should use the "n" value for the chosen liner. The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

$$\boxed{Q = \left(\frac{1.486}{n}\right) AR^{2/3} S^{1/2}}$$

(Eq. 3.6)
Uniform Flow

For a given channel geometry, slope, roughness and discharge, a unique value of depth occurs in steady, uniform flow. This unique depth is referred to as normal depth and is computed from Eq. 3.6 after the area and hydraulic radius are expressed in terms of depth. The resulting equation may require a trial-and-error solution. If the normal depth is greater than critical depth, the slope is classified as a mild slope. If the normal depth is less than critical depth, the slope is classified as a steep slope. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

C. Conveyance

In channel analysis, it is often convenient to group the channel cross section properties of Eq. 3.2 in a single term called the channel conveyance *K*:

$$\boxed{K = \left(\frac{1.486}{n}\right) AR^{2/3}}$$

(Eq. 3.7)
Channel Conveyance

and then Eq. 3.6 can be written as:

$$\boxed{Q = KS^{1/2}}$$

(Eq. 3.8)
Simplified Uniform Flow

D. Energy Equation

The energy equation expresses conservation of energy in open channel flow expressed as energy per unit weight of fluid, which has dimensions of length and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities that give the total energy head at any cross section when added. Written between an upstream open channel cross section designated 1 and a downstream open channel cross section designated 2 (see Figure 3-1), the energy equation is:

$$\boxed{h_1 + a_1\left(\frac{(v_1)^2}{2g}\right) = h_2 + a_2\left(\frac{(v_2)^2}{2g}\right) + h_L}$$

(Eq. 3.9)
Energy Equation

Where:

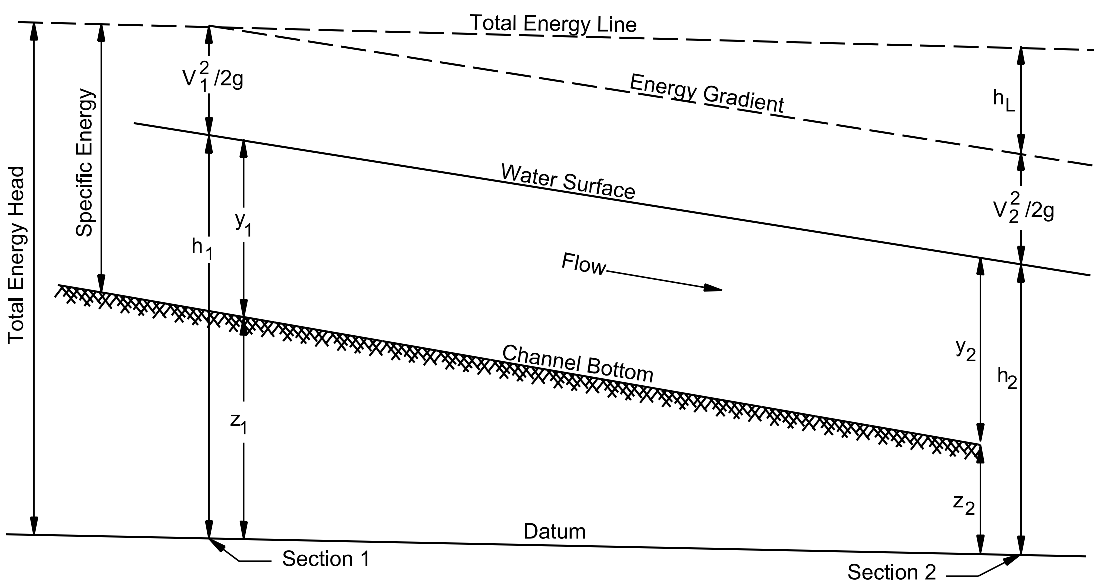
h_1, h_2 = the upstream and downstream stages, respectively, ft

a = velocity distribution coefficient

v = mean velocity, ft/s

h_L = head loss due to local cross-sectional changes (minor loss) and boundary resistance, ft

The terms in the energy equation are illustrated graphically in Figure 3-1. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.



Source: HDS-4 (6)

Figure 3-1: Terms in the Energy Equation

3.8 CHANNEL HYDRAULIC ANALYSIS

3.8.1 General

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness, and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings and highway drainage structures.

The two methods most commonly used to analyze open channel flow regimes are single-section analysis (Section 3.8.5) and step-backwater method.

The single-section analysis method is a simple application of Manning's equation to determine flow depth and velocity in a known channel, or tailwater rating curves for culverts, or to analyze other situations where uniform or nearly uniform flow conditions exist. The designer should take care to correctly model the characteristics of the channel.

The Step-Backwater method, also known as the Standard Step Method, is used to compute the complete water surface profile in a stream reach or to analyze other gradually varied flow situations in open channels. Since the step-backwater method is most appropriate for evaluating the unrestricted water surface elevations for bridge hydraulic design and major channel design, it is outside the scope of this manual, and not recommended for analysis of roadside channels. If a situation arises that necessitates the use of this method, designers are encouraged to use the U.S. Army Corps of Engineers (USACE) HEC-RAS software.

Occasionally, the hydraulics engineer may need to use a more detailed method of analysis than the single-section method or the computation of a water surface profile using the step-backwater method. Special analysis techniques include two-dimensional analysis, water and sediment routing, and unsteady flow analysis. When the engineer believes these to be necessary, the project scope and appropriate methodology should be discussed with MDOT SHA prior to beginning work on the analysis.

3.8.2 Cross Sections

The cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation that locate individual ground points. The cross section is taken normal to the flow direction along a single, straight line where possible but, in wide floodplains or bends, it may be necessary to use a section along intersecting straight lines; i.e., a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the sub reaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections (e.g., free overfalls, bends, contractions), or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals to better model the change in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry or roughness, or both, as for overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection.

3.8.3 Manning's "n" Value Selection

Roadside / Median / Outfall Channels

To promote consistency in computations for channels constructed for MDOT SHA projects, selection of Manning's "n" values will follow the following procedure:

For grass channels maintained by MDOT SHA, the Manning's "n" values will use the following protocol for all applications. This accounts for the increased resistance of established grass to a flow depth of 4 inches, and the proportional decrease in "n" value as flow depth increases.

- For flow depths 4" and below: Manning's "n" = 0.15
- For flow depths (d, inches) between 4" and 12": "n" = 0.207 – (0.0145 * d)
- For flow depths greater than 12": "n" = 0.033

Because the Manning's "n" value is dependent on depth of flow, with certain computations (such as determining velocity of a known discharge within a channel) it will be necessary to verify that the flow depth computed in the channel is the value from which the Manning's "n" is computed. Multiple iterations may be required for the flow depth assumption and the computed flow depth to converge.

For channels with other linings maintained by MDOT SHA, the Design Value on Table 3.3 should be used. Design values given are applicable to a range of flow depth values from approximately 0.5 ft to 2 ft. For flows depths significantly outside this range, designers may be justified in using values at the higher (low flow depths) or lower (high flow depths) ends of the range.

Table 3-3: Manning's "n" Values for Various Channel Linings

TYPE OF LINING	RANGE OF VALUES	DESIGN VALUE
Established Vegetation (Class C Vegetation)	Use grass channel guidance above	
Bare Soil	0.020 to 0.023	0.020
Type A SSM (Curled Wood Matting)	0.028 to 0.066	0.033
Type B SSM (Synthetic Matting)	0.021 to 0.036	0.025
Type C SSM (Turf Reinforcement Matting)	Use bare soil value until establishment of vegetation, vegetated value after establishment	
Type D SSM (Jute Net)	0.019 to 0.028	0.022
Type E SSM (Degradable straw/coconut blend fibers)	0.028 to 0.066	0.033
Paved Surface	0.013 to 0.015	0.013
Class I Riprap	0.035	0.035
Class II Riprap	0.040	0.040
Gabions	0.035	0.035
Gravel (D_{50}=2 in)	0.034 to 0.066	0.041
Rock Cut	0.025 to 0.045	0.035

Natural Channels

Manning's "n" is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and floodplains for which the discharge has been measured and Manning's "n" has been calculated are very useful, such as those found in USGS Water

Supply Papers 1849 (*1978)* and 2339 *(1984)*. For situations lying outside the hydraulics engineer's experience, a more regimented approach is presented in WSP 2339. Once the Manning's "n" values have been selected, it is highly recommended that they be verified or calibrated with historical high-water marks or gaged streamflow data, or both.

3.8.4 Calibration

When appropriate, the results of an open channel hydraulic analysis should be compared to other information for calibration, such as historical high-water marks or gaged streamflow data, to ensure that the analysis accurately represents local channel conditions. The following parameters, in order of preference, should be used for calibrations: Manning's "n", slope, discharge, and cross section.

3.8.5 Single-Section Analysis (Slope-Area Method)

The single-section analysis method (also known as the slope-conveyance or slope-area method) is simply a solution of Manning's equation for the normal depth of flow given the discharge and cross section properties, including geometry, slope, and roughness. The analysis assumes the existence of steady, uniform flow. However, uniform flow rarely exists in either artificial or natural stream channels. Nevertheless, the single-section method is often used as a first approximation to design constructed channels or to develop a stage-discharge (rating) curve in a stream channel.

A stage-discharge curve is a graphical relationship of streamflow depth or elevation to discharge at a specific point on a stream. This relationship should cover a range of discharges up to at least the base (100-year) flood. A stage-discharge curve may be developed for tailwater determination at a culvert or storm drain outlet:

- Select the typical cross section at or near the location where the stage-discharge curve is needed.
- Subdivide the cross section and assign Manning's "n" values to subsections as described in Sections 3.8.2 and 3.8.3.
- Estimate water-surface slope. Because uniform flow is assumed, the average slope of the channel can usually be used.
- Apply a range of incremental water surface elevations to the cross section.
- Calculate the discharge using Manning's equation for each incremental elevation. Total discharge at each elevation is the sum of the discharges from each subsection at that elevation. In determining hydraulic radius, the wetted perimeter should be measured only along the solid boundary of the cross section and not along the vertical water interface between subsections.
- After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge should be made. This plot is the stage-discharge curve, and it can be used to determine the water surface elevation corresponding to the design discharge or other discharge of interest.

Although the above procedure can be accomplished manually, software such as the FHWA Hydraulic Toolbox is normally used to compute flows in for trapezoidal and prismatic channels.

In stream channels, the transverse variation of velocity in any cross section is a function of subsection geometry and roughness and may vary considerably from one stage and discharge to another. It is important to know this variation for designing erosion control measures and locating relief openings in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single-section method can be used by dividing the cross section into subsections of relatively uniform roughness and geometry. It is assumed that the energy grade line slope is the same across the cross section so that the total conveyance K_t of the cross section is the sum of the subsection conveyances. The total discharge is then $K_tS^{1/2}$ and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, $V = Q/A$.

Alluvial channels present a more difficult problem in establishing stage-discharge relations by the single-section method because the bed itself is deformable and may generate bed forms (e.g., ripples, dunes) in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's "n" is not straightforward *(FHWA*, 2001*)*. However, if the bed form can be assessed as stable, a single section can be used to estimate culvert tailwater.

There may be locations where a stage-discharge relationship has already been measured in a channel. These usually exist at gaging stations on streams monitored by USGS. Measured stage-discharge curves will generally yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described above.

An example of the application of the single-section method for determining the discharge in an irregularly shaped channel is presented as example 4.3 in the HDS-4 *(FHWA, 2008)*.

3.9 REFERENCES

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3.10 DESIGN AIDS

Chapter 3 Design Aids include the following Tables, Figures and Charts

- Table 3-4 Classification of Vegetal Covers as to Degrees of Retardancy
- Table 3-5 Grass Roughness Coefficient, C_n, for SCS Retardance Classes

Table 3-4: Classification of Vegetal Covers as to Degrees of Retardancy

RETARDANCE	COVER	CONDITION
A	Weeping lovegrass Yellow bluestem Ischaemum	Excellent stand, tall (average 30 in.) Excellent stand, tall (average 36 in.)
B	Kudzu Bermuda grass Native grass mixture: little bluestem, bluestem, blue gamma other short- and long-stem Midwest grasses Weeping lovegrass *Lespedeza sericea* Alfalfa Weeping lovegrass Kudzu Blue gamma	Very dense growth, uncut Good stand, tall (average 12 in.) Good stand, unmowed Good stand, tall (average 24 in.) Good stand, not woody, tall (average 19 in.) Good stand, uncut (average 11 in.) Good stand, unmowed (average 13 in.) Dense growth, uncut Good stand, uncut (average 13 in.)
C	Crabgrass Bermuda grass Common lespedeza Grass-legume mixture: summer (orchard grass redtop, Italian ryegrass and common lespedeza) Centipede grass Kentucky bluegrass	Fair stand, uncut (10 in.–48 in.) Good stand, mowed (average 6 in.) Good stand, uncut (average 11 in.) Good stand, uncut (6 in.–8 in.) Very dense cover (average 6 in.) Good stand, headed (6 in.-12 in.)
D	Bermuda grass Common lespedeza Buffalo grass Grass-legume mixture: fall, spring (orchard grass redtop, Italian ryegrass and common lespedeza) *Lespedeza serices*	Good stand, cut to $2\frac{1}{2}$ in. Excellent stand, uncut (average $4\frac{1}{2}$ in.) Good stand, uncut (3 in.-6 in.) Good stand, uncut (4 in.–5 in.) After cutting to 2 in. (very good before cutting)
E	Bermuda grass Bermuda grass	Good stand, cut to $1\frac{1}{2}$ in. Burned stubble

Note: Covers classified have been tested in experimental channel. Covers were green and generally uniform. Source of table HEC-15 (4).

Table 3-5: Grass Roughness Coefficient, C_n, for SCS Retardance Classes

RETARDANCE CLASS	A	B	C	D	E
Stem Height, in.	36	24	8.0	4.0	1.6
C_s	33	7.1	3.9	2.7	3.8
C_n	0.605	0.418	0.220	0.147	0.093

Source: HEC-15 *(8)* Overview

4 CULVERTS

4.1 OVERVIEW

4.1.1 Introduction

This chapter provides design procedures for the hydraulic design of highway culverts and is based on FHWA Hydraulic Design Series No. 5 (HDS-5), Hydraulic Design of Highway Culverts *(FHWA, 2012)*.

4.1.2 Culvert Definition

A culvert is defined as one or more transverse and fully enclosed drainage structures that convey runoff through roadways or embankments via a pipe, channel, or similar conduit, and includes any drainage structure or part of a drainage network that crosses through an embankment. The Office of Highway Development (OHD) is responsible for developing design guidance and asset management strategies for smaller culverts within Maryland's State highway system that are outside the Office of Structure's (OOS) inventory. The OOS is responsible for the design and asset management of bridges and larger culverts that are classified as "small structures".

Structures with an opening greater than 20 feet as measured along the centerline of the roadway, are considered bridges for purpose of the OOS bridge inventory. This includes extreme ends of multiple boxes and multiple pipes where the clear distance between the opening is less than half of the smaller contiguous opening.

Small structures are defined as structures having an opening measured along the centerline of roadway greater than or equal to 5 feet but less than or equal to 20 feet. Small structures also include structures having an opening equal to or greater than 3 feet and less than 5 feet where the depth of fill over the structure measures less than the diameter of the opening.

This Manual focuses on culverts outside the OOS inventory. Guidance on bridges and small structures can be found in the OOS Manual for Hydrologic and Hydraulic Design *(MDOT SHA OOS, 2020)*.

4.1.3 Definitions

Following are defined terms that are relevant to culvert design.

Backwater. An increase in upstream water surface elevation resulting from obstruction of flow.

Critical Depth. In channels with regular cross section, critical depth is the depth at which the specific energy of a given flow rate is at a minimum. For a given discharge and cross-section geometry, there is only one critical depth.

Crown. The crown is the inside top of the culvert.

Flow Type. USGS has established seven culvert flow types, which assist in determining the flow conditions at a culvert site *(USGS, 1968)*. Diagrams of these flow types are provided in Section 4.7.

Free Outlet. Free outlet happens when tailwater depth is equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Headwater. The depth from the inlet invert to the energy grade line at the upstream end of a culvert or pipe. Also used to refer to the pool of water at the upstream end of a hydraulic structure.

Improved Inlet. An improved inlet has an entrance geometry that decreases the flow contraction at the inlet and thus increases the capacity of a culvert. These inlets are referred to as either side- or slope-tapered. The side-tapered inlet has a face wider than the culvert. The slope-tapered inlet has both a larger face and increased flow-line slope at the entrance. Beveled edges at the culvert face may also improve the hydraulic capacity of a culvert for both conventional and improved inlets.

Invert. The invert is the flowline of the culvert (inside bottom).

Normal Depth. Normal depth occurs in a channel or culvert when the slope of the water surface and channel bottom is the same and the water depth remains constant. The discharge and velocity are constant throughout the reach. Normal flow will exist in a culvert operating on a constant slope provided that the culvert is sufficiently long.

Probable Maximum Flood. A hypothetical flood event that represents the flood that can be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in a region.

Slope. The measurement of inclination of a pipe, representing the difference in elevation of the inlet and outlet inverts along the centerline of the pipe. A steep slope occurs where the normal depth is less than the critical depth. A mild slope occurs where the normal depth is greater than the critical depth.

Submerged. A submerged outlet occurs where the tailwater elevation is higher than the crown of the culvert. A submerged inlet occurs where the headwater is greater than 1.2D, where D is the culvert diameter or barrel height.

Tailwater. The depth of water at the downstream end of the culvert. Also used to refer to the water located immediately downstream from a hydraulic structure.

4.2 GENERAL CONSIDERATIONS

Hydrologic and Hydraulic guidance in this Manual applies to culverts with drainage areas less than 400 acres (0.625 square mile). For drainage areas 400 acres and greater, refer to the OOS Manual for Hydrologic and Hydraulic Design Section 1.3. *(MDOT SHA OOS, 2020)*

Prepare the hydraulic design of culverts in accordance with good engineering judgment and comply with 23 CFR 650A and the National Flood Insurance Program (NFIP). Design generally considers topographic features, channel characteristics, aquatic life, high-water information, existing structures, soil and water chemical characteristics, abrasion potential, and other related site-specific information. The detail of documentation for each culvert site is typically proportional to the risk and importance of the structure. Design data and calculations shall be assembled in an orderly fashion and documented for future reference as provided for in Section 1.9. Where design criteria cannot or should not be met, obtain a design exception from the Highway Hydraulics Division Chief.

4.3 DESIGN CONSIDERATIONS

4.3.1 Clear Zones

A Clear Zone is an unobstructed, traversable roadside area that allows for the recovery of errant vehicles. Projecting culvert openings, headwalls, and endwalls can sometimes be considered a roadside hazard. Clear zone criteria should be factored into the design and location of culverts and their associated appurtenances to minimize traffic hazards. The MDOT SHA Guidelines for Traffic Barrier Placement and End Treatment Design provides additional information on the Clear Zone concept.

4.3.2 Waters of the United States

Waters of the United States, commonly abbreviated as WUS, are often broken up into tidal and non-tidal waters. The presence of WUS will result in additional considerations for the designer. For instance, per Section 26.17.04.06 of COMAR, the length of culverts shall be limited to 150 feet in length unless it can be demonstrated through an environmental study that any adverse impacts will be adequately mitigated. Additionally, culverts shall have at least one cell placed at least 1 foot below the invert of the stream to promote the development of natural channel bottom. When culverts provide a natural bottom, the effects upon stream downcutting must be considered. Culvert installations should be designed to maintain stream stability and to provide aquatic organism passage (AOP) and/or wildlife passage where applicable. No asphalt coatings may be used on pipes within stream systems since the coating may abrade and enter the stream environment.

4.3.3 FEMA Regulated Floodways

Design shall be consistent with standards established by FEMA for locations where a regulatory floodway has been designated or where studies are underway to establish a regulatory floodway. Consider the discharges and methods specified in the FEMA flood insurance study. FEMA has established administrative procedures for updating effective Flood Insurance Rate Maps (FIRMs) and Flood Insurance Studies (FIS). For projects that encroach into a regulatory floodway, coordinate with the appropriate local government flood insurance program official.

4.3.4 Economics

The designer should always consider the total life-cycle cost of a culvert, including maintenance, rehabilitation, and replacement costs, in an evaluation of alternatives. Pipe material service life is a major factor in estimating the lifespan of a culvert and additional guidance is provided in Section 4.10. Facilitating inspection and maintenance access to a culvert during design is one way to minimize asset preservation costs in the future. Designers can consider ensuring pull-off areas are available for service vehicles and avoiding overly steep slopes near culvert openings. The designer should also weigh the cost savings of multiple uses (utilities, stock and wildlife passage, land access, and fish and aquatic organism passage) against the advantages of separate facilities, where applicable.

4.4 DESIGN FREQUENCY

4.4.1 Permanent Facilities

The design storm frequency used to design or review a culvert is primarily based on the roadway functional classification, as shown in Table 4-1.

Table 4-1: Design Storm Frequencies

FUNCTIONAL CLASSIFICATION[1]	ANNUAL EXCEEDANCE PROBABILITY (%)	RETURN PERIOD (YEAR)
Interstates and Principal Arterials	1%	100
Minor Arterials	2%	50
Collectors	4%	25
Local Roads	10%	10

[1]Ramps between facilities of different classifications should use the return period of the higher classification

Other considerations include:

- The location of FEMA mapped floodplains
- Maryland Department of the Environment (MDE) Dam Safety guidelines and policies on roadway embankments outlined in Section 4.5.5 and on the MDE Dam Safety Program website
- An economic assessment or analysis to justify the flood frequencies greater or lesser than the minimum flood frequencies listed below

In addition to the design storm, designers must always analyze the overtopping flood or base flood (100-year frequency flood), whichever is greater, to consider the risk. The Probable Maximum Flood may be used wherever overtopping is not practical. Design software can often run multiple storm events simultaneously. Document this analysis, including peak stages and discharges, in the Drainage Report.

4.4.2 Temporary Facilities

Design storm frequency for temporary culverts used by the traveling public should be based upon the permitted duration of the adjacent work. Temporary culverts shall cause no more than a one-foot increase in the Design Storm Frequency (DSF) flood elevation immediately upstream within MDOT SHA right of way or easement. Increases outside of MDOT SHA property must be limited to less than 0.1 feet. Minimum design frequencies for temporary culverts and bridges are shown in Table 4-2.

Table 4-2: Design Storm Frequencies of Temporary Facilities

DURATION OF WORK	ANNUAL EXCEEDANCE PROBABILITY (%)	RETURN PERIOD
< 12 Months	50%	2 years
12-36 Months	10%	10 years
> 36 Months		Use ***Table 4-1***

4.5 DESIGN CRITERIA

4.5.1 Hydrologic Analysis

Hydrologic analyses shall be performed in accordance with Chapter 2 Hydrology as appropriate for the site. Per Section 26.17.04.04 of COMAR, hydrologic calculations for culverts constructed on Nontidal Waters and Floodplains shall be based on the ultimate development of the watershed, assuming existing zoning.

4.5.2 Design Limitations

A. Allowable Headwater

Allowable headwater is the depth of water that shall not be exceeded at the upstream end of the culvert which will be limited by one or more of the following:

a. Is non-damaging to and does not increase depth upon upstream property,

b. Below the edge of the pavement for the design frequency storm,

c. No greater than the low point in the road grade,

d. Equal to the elevation where flow diverts around the culvert,

e. HW/D to be between 1.0 and 1.5 for the design frequency storm, and

f. HW/D ≤ 2.0 for the 100-year storm.

When one or more of the allowable headwater elevation criteria cannot or should not be met, a design exception request must be approved by the Office of Highway Development, Highway Hydraulics Division Chief.

B. Regulated Floodways

Projects located in floodplains regulated by the Federal Emergency Management Agency (FEMA) must comply with floodplain regulations as per the Code of Federal Regulations Title 44 (44 CFR). This includes adherence to 44 CFR 60(d)(3) which indicates that culvert improvements within a regulated floodway shall not increase the base flood (100-year) elevation.

When the Federal Insurance Administrator has identified 100-year flood elevations within a flood prone area, but has not identified a regulatory floodway, the CFR indicates that water surface elevations of the 100-year flood may not increase more than one foot. However, MDOT SHA maintains more restrictive standards requiring no increase to the depth on upstream property owners. Any increase in the base flood elevation must be contained within the MDOT-SHA right-of-way or perpetual easement.

C. Tailwater Relationship – Channel

Evaluate the hydraulic conditions of the downstream channel to determine a tailwater depth for a range of discharges which includes the review discharge (see Chapter 3 Channels). Calculate backwater curves at sensitive locations or use a single cross section analysis. Culverts operating in outlet control may require additional analysis of downstream conditions to properly size the culvert. Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall. Use the headwater elevation of any nearby, downstream culvert if it is greater than the channel depth.

D. Tailwater Relationship – Confluence or Large Water Body

Use the high-water elevation of the confluence or large water body corresponding to:

a. The same frequency as the design flood if events are known to occur concurrently (statistically dependent), or
b. Evaluate the joint probability of flood magnitudes and use a likely combination resulting in the greater tailwater depth (statistically independent), or
c. When tidal conditions are present, use the mean high tide.

E. Length and Slope

Choose culvert length and slope to approximate existing topography, and to the degree practicable align the culvert invert with the channel bottom and the skew angle of the stream, and the culvert entrance to match the geometry of the roadway embankment.

F. Maximum Velocity

The maximum velocity at the culvert exit shall be consistent with the velocity in the natural channel or shall be mitigated with:

a. Channel stabilization (see Chapter 3 Channels), and
b. Energy dissipation (see Section 4.8 Outfall Protection).

G. Minimum Velocity

The minimum velocity in the culvert barrel shall result in a tractive force (shear stress) greater than critical shear stress of the transported streambed material at low flow rates.

a. Use 2.5 feet per second when streambed material size is not known.
b. If clogging with sediment is probable, consider sizing the culvert to facilitate cleaning.

H. Storage: Temporary or Permanent

If storage is being assumed upstream of the culvert, consider:

a. The total area of flooding for the design storm and/or the base flood event,
b. Limiting the average time that bankfull stage is being exceeded for the design flood to 48 hours in rural areas or 6 hours in urban areas,

c. Ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement,

d. Limiting the headwater elevation to a depth below that which will necessitate evaluation of the embankment as a dam.

I. Debris Control

Debris-control devices may be designed using the procedures shown in FHWA's Hydraulic Engineering Circular No. 9, "Debris Control Structures" *(FHWA, 2005)* and should be considered:

a. Where experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris,

b. Where fish migration is a concern,

c. For culverts located in mountainous or steep regions,

d. For culverts that are under high fills, and

e. Where clean out access is limited. However, access must be available to clean out the debris control device.

J. Structural Design

Design to meet HS-25 loading according to AASHTO LRFD Specifications.

4.5.3 Coastal Zone

Coastal storms (e.g., hurricanes, northeasters) should be considered, when appropriate, as possible design storms in targeting the most challenging hydrologic and hydraulic conditions on a highway project. Historical records should be investigated, and potential site conditions analyzed in determining the most critical coastal storm event.

When coastal roadways are integrated into emergency management plans, the coastal storm hydrology frequency should align with the criteria and function targeted in the local emergency management plan. In these plans, some evacuation routes may be mandated for use during or immediately after coastal storms and must be designed to not overtop and to survive wave attack during the coastal design storm. In these cases, consider surge height, wave and wind setup, wave heights, wave runup, and culvert buoyancy in keeping the roadway usable during the coastal storm. Design practice is to assign a return period of 100-years to such critical roadways.

4.5.4 Structural Evaluation

MDOT SHA requires all culverts meet HS-25 loading according to AASHTO LRFD Specifications. Section 4.15.2 provides minimum and maximum cover guidance. Materials and sizes not listed in Section 4.15.2 shall be evaluated using AASHTO design guidelines and industry recommendations and modified as necessary to be consistent with MDOT SHA guidance and any applicable specifications and installation procedures.

4.5.5 Roadway Embankment Design Criteria

The Code of Maryland Regulations (COMAR) 26.17.04.02.A (4) defines a "dam" as:

"*...any obstruction, wall or embankment, together with its abutments and appurtenant works, if any, in, along, or across any stream, heretofore or hereafter constructed for the purpose of storing or diverting water or for creating a pool upstream of the dam...*"

Under certain conditions, roadway embankments can be considered to be dams subject to MDE Dam Safety review and approval or as small ponds subject to MDE Small Pond review and approval.

MDE Dam Safety and Small Pond review and approval processes may be revised and are subject to change without notice. MDE issued a Dam Safety Policy Memorandum #2, dated June 11, 2019 (revised in February of 2022) to provide guidance for when a roadway/railroad embankment is functioning as a dam. Refer to the MDE Dam Safety Program website for more information and all subsequent updates to aforementioned policies/documents.

Roadway embankments should avoid impounding water excessively for the purposes for stormwater management or otherwise. Designers should investigate and propose design solutions which will result in roadway embankments being classified as culverts per MDE Dam Safety Policy Memorandum #2.

4.5.6 Jack and Bore

Refer to the MDOT SHA Utility Manual *(MDOT SHA OOC, 2021)* for drainage structure installations using jack and bore techniques.

4.6 DESIGN FEATURES

4.6.1 Culvert Sizes and Shape

The culvert size and shape selected shall be based on engineering and economic criteria related to site conditions.

a. The following minimum pipe diameters shall be used to avoid maintenance issues and clogging:

   - 24 inches for lengths greater than 60 feet
   - 18 inches for other systems
   - 15 inches for a side-drain or driveway.

b. Other requirements such as aquatic organism and wildlife passage can dictate a larger or different barrel geometry than required for hydraulic considerations.

c. Use arch or oval shapes only if required by hydraulic limitations, site characteristics, structural criteria, or environmental criteria.

4.6.2 Multiple Barrels

Multiple barrel culverts may be an economical means of conveying large flows in areas with minimal cover over the pipes. When located in natural stream channels, designers should place a single barrel within the natural dominant channel with additional barrels placed in the overbank areas at higher elevations with minimal widening of the channel to avoid conveyance loss through sediment deposition.

Multiple barrel culverts are to be avoided in the following situations:

a. The approach flow is high velocity, particularly if supercritical (these sites require either a single barrel or special inlet treatment to avoid adverse hydraulic jump effects).

b. Aquatic organism passage is required, unless special treatment is provided to ensure adequate low flows (commonly one barrel is lowered).

c. A stream channel has high sediment transport rates. Multiple barrel culverts exhibit a greater propensity for sedimentation that reduces the effective culvert cross sectional area.

The minimum spacing to ensure adequate compaction between cells is ½ pipe diameter but no less than 2 feet.

4.6.3 Material Selection

The material selection shall consider replacement cost and difficulty of construction as well as traffic delay.

a. The material selected shall be based on a comparison of the total cost of alternate materials over the design life of the structure which is dependent upon the following:

- Durability (service life),
- Installed structural performance,
- Hydraulic roughness,
- Flow velocity,
- Bedding conditions,
- Abrasion and corrosion resistance, and
- Water tightness requirements.
- Installation considerations such as steep slopes and access.

b. The selection shall not be made using initial construction cost as the only criteria.

c. See Section 4.9.3 for approved pipe materials.

4.6.4 Skew

The culvert skew shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without discussion with Highway Hydraulics Division.

4.6.5 Curved Alignments

Abrupt changes in a culvert's horizontal or vertical alignment should be avoided as they reduce hydraulic efficiency and generate durability and maintenance issues. Changes in direction are usually accomplished at manhole structures. Uses of radius pipe or prefabricated bends will only be allowed with approval of the Highway Hydraulics Division.

A culvert should be located in the existing stream bed or as close to it as possible. Any abrupt change of direction at either end of a culvert will retard flow and may trap debris and cause scouring or silting. A channel change, if unavoidable, should be minimized and made at the culvert outlet, rather than at the inlet.

4.6.6 End Treatment

All culvert ends (inlet or outlet) shall be protected. Consideration shall be given to safety since some end treatments can be hazardous to errant vehicles. If the culvert cannot be extended outside the clear zone, the use of end grates may be required as per AASHTO Roadside Design Guide. Culvert end treatments such as headwalls and endwalls within the clear zone shall be oriented parallel to the roadway; otherwise place perpendicular to the pipe.

A. Headwalls

Properly designed and constructed headwalls and endwalls improve culvert capacity and efficiency while providing embankment stability and erosion protection. They also protect culverts against buoyancy, shorten the required structure length and reduce maintenance damage.

a. Concrete endwalls or headwalls shall be provided for proposed culverts larger than 36-inch diameter or for any size culvert with base flow or where backwater condition is anticipated.

b. For 48-inch, 54-inch and 60-inch diameter culverts, MDOT SHA type ‘B’ headwalls (standard detail MD-352.01 or MD-352.02) or type ‘G’ endwalls (standard detail MD-360.01) are preferred – rather than type ‘C’ endwalls.

c. For 66-inch, 72-inch, 78-inch and 84-inch diameter culverts, MDOT SHA type ‘B’ headwalls (standard detail MD-352.02) shall be specified.

B. Inlet Types (Square-edged, Beveled, or Tapered)

Square-edged inlets are the standard type of culvert inlet and are typically used in the majority of locations for MDOT SHA culverts. When it is necessary for hydraulic reasons, improved inlets may be used on culvert locations.

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. A tapered inlet may have a throat depression incorporated into the inlet structure or located upstream of the inlet. The depression is used to exert more head on the throat section for a given headwater elevation. Therefore, tapered inlets improve culvert performance by providing a more efficient control section (the throat). Tapered inlets are not recommended for use on culverts flowing in outlet control because the simple beveled edge is of equal benefit.

When beveled or tapered improved inlets are used in the design, the designer must specifically convey the type of culvert inlet on the resulting plans in order to ensure that the culvert is constructed to the intended design.

C. End Sections

Concrete or metal end sections can be provided for proposed culverts up to 36-inch in diameter (or equivalent elliptical/arch size). They help prevent embankment erosion and incur less damage from maintenance. End sections:

a. Should provide the requisite service life.

b. May improve projecting pipe entrances by increasing hydraulic efficiency, reducing the accident hazard, and improving their appearance.

c. Limited testing has suggested that end sections are hydraulically equal to a headwall and can be equal to a beveled or side- tapered entrance if a flared, enclosed transition takes place before the barrel. MDOT will accept designs that model them as equivalent.

d. Should not be used where the culvert end is submerged for a prolonged time.

e. Should not be used where the culvert skew exceeds 30 degrees as measured from a line perpendicular to the roadway centerline.

f. Should not be used where the culvert carries a base flow.

g. Should not be used with steep pipe or channel bed slopes greater than 10%.

h. If located within the ‘clear zone’, safety end sections shall be provided (standard details MD-372.00 or MD-373.00).

i. Where Corrugated Polyethylene Pipe (CPP) or High-Density Polyethylene Pipe (HDPE) is proposed, polyethylene end sections should not be used – due to degradation from ultraviolet (UV) radiation exposure, resistance issues to fire and errant vehicle damage, etc. Concrete or metal end section with appropriate coupler, or concrete endwall/headwall should be provided.

D. Wingwalls

a. Are used to retain the roadway embankment to avoid a projecting culvert barrel.

b. Are used where the side slopes of the channel are unstable.

c. Are used where the culvert is skewed to the normal channel flow.

d. Can affect hydraulic efficiency if the flare angle is < 30° or > 60°.

E. Inlet Aprons

a. Are used to reduce scour from high headwater depths or from approach velocity in the channel.

b. Shall extend at least one pipe diameter upstream.

c. Shall not protrude above the normal streambed elevation.

d. Provide soil key-in to avoid undermining.

4.6.7 Safety Considerations

Traffic shall be protected from culvert ends as follows:

a. Small culverts 30” diameter or less, may use an end section where appropriate or a sloped headwall.

b. Culverts greater than 30” diameter shall receive one of the following treatments.

- Extended to the appropriate "clear zone" distance per AASHTO Roadside Design Guide.
- Safety treated with a grate if the consequences of clogging and causing a potential flooding hazard is less than the hazard of vehicles impacting an unprotected end. If a grate is used, an open area shall be provided between the bars of 1.5 to 3.0 times the

area of the culvert entrance. See Safety end sections MD 372.00, MD 372.01, MD 373.00 or MD 373.01.
- Shielded with a traffic barrier if the culvert is very large, cannot be extended, has a channel which cannot be safely traversed by a vehicle, or has a significant flooding hazard with a grate.

c. Periodically inspect each site to determine if safety problems exist for traffic or for the structural safety of the culvert and embankment.

4.6.8 Performance Curves

Performance curves shall be developed for all culverts for evaluating the hydraulic capacity of a culvert for various headwaters, outlet velocities, and scour depths. These curves will display the consequence of high flow rates at the site and provide a basis for evaluating flood hazards.

4.6.9 Related Designs

A. Buoyancy Protection

Headwalls, endwalls, slope paving or other means of anchoring to provide buoyancy protection shall be considered for all culverts. Buoyancy is more serious with lighter materials, steeper culvert slopes, high headwater depths (debris blockage may increase), flatness of the upstream fill slope, minimal fill heights, large culvert skews, mitered ends, or high groundwater levels.

B. Subgrade Analysis

Soft or loose soils can result in complications such as differential settlement in culverts. When soft or loose soils are encountered in the field or described in the boring logs, coordinate with the Office of Materials Technology Engineering Geology Division to determine if a settlement analysis is needed.

C. Outlet Protection

See Section 4.8.

D. Land Use Culverts

A land-use culvert is a culvert designed to carry the design flood and to provide passage under a highway for utilities, stock and wildlife animals, farmers, machinery, etc. Consideration shall be given to combining drainage culverts with other land use requirements, including the following scenarios:

- Areas where the land use is temporarily inundated during the selected design flood, but available during lesser floods.
- Situations where two or more barrels are required with one barrel intended to be dry during floods less than the selected design flood.

Such culverts shall meet the following additional criteria:

- The outlet of the higher land use barrel must be protected from headcutting.
- The land use culvert shall be sized to ensure it can serve its intended land use function up to and including a 2-year flood.
- The height and width constraints shall satisfy both the hydraulic and land use requirements.

E. Relief Opening

Where multiple use culverts or culverts serving as relief openings have their outlet set above the normal stream flow line, headcuts that would undermine the culvert outlet or cause damage to downstream properties shall be prevented. Refer to Section 4.8 for outfall protection.

F. Maintenance of Stream Flow

Temporary measures to maintain stream flow shall be included in the construction plans. These measures shall be consistent with the latest guidelines issued by the Maryland Department of the Environment.

G. Aquatic Organism Passage

Designers shall ensure that the minimum requirements for fish and other aquatic organism passage are met in nontidal waters per Section 26.17.04.06 of the Code of Maryland Regulations (COMAR).

H. Environmental Considerations

Care must be exercised in selecting the location of the culvert site to control erosion, sedimentation and debris. Select a site that will permit the culvert to be constructed and will limit the impact on the stream or wetlands. For additional guidance, contact the Office of Environmental Design.

4.7 CULVERT ANALYSIS AND DESIGN

4.7.1 Hydraulic Design

An exact theoretical analysis of culvert flow is extremely complex because the following is required:

- Analyzing non-uniform flow with regions of both gradually varying and rapidly varying flow;
- Determining how the flow type changes as the flow rate and tailwater elevations change;
- Applying backwater and drawdown calculations, energy, and momentum balance;
- Applying the results of hydraulic model studies; and
- Determining if hydraulic jumps occur and if they are inside or downstream of the culvert barrel.

Most of the above complications are addressed in the FHWA software HY-8. The following discussion provides the basic equations that are used by HY-8 and other culvert analysis software.

4.7.2 Standard Practice

HDS-5 *(FHWA, 2012)* is the standard practice for the hydraulic design of culverts. The hydraulics engineer has the option of performing an analysis using the equations outlined in this chapter, using the nomographs in section 4.14 or using software that is consistent with the equations provided in HDS-5. Refer to Table 1-1 in Section 1.10 for approved software.

A. Design Discharge

Culverts will be designed for a constant discharge that will normally be the peak discharge. This will yield a conservatively sized structure where temporary storage is available but not used.

B. Control Section

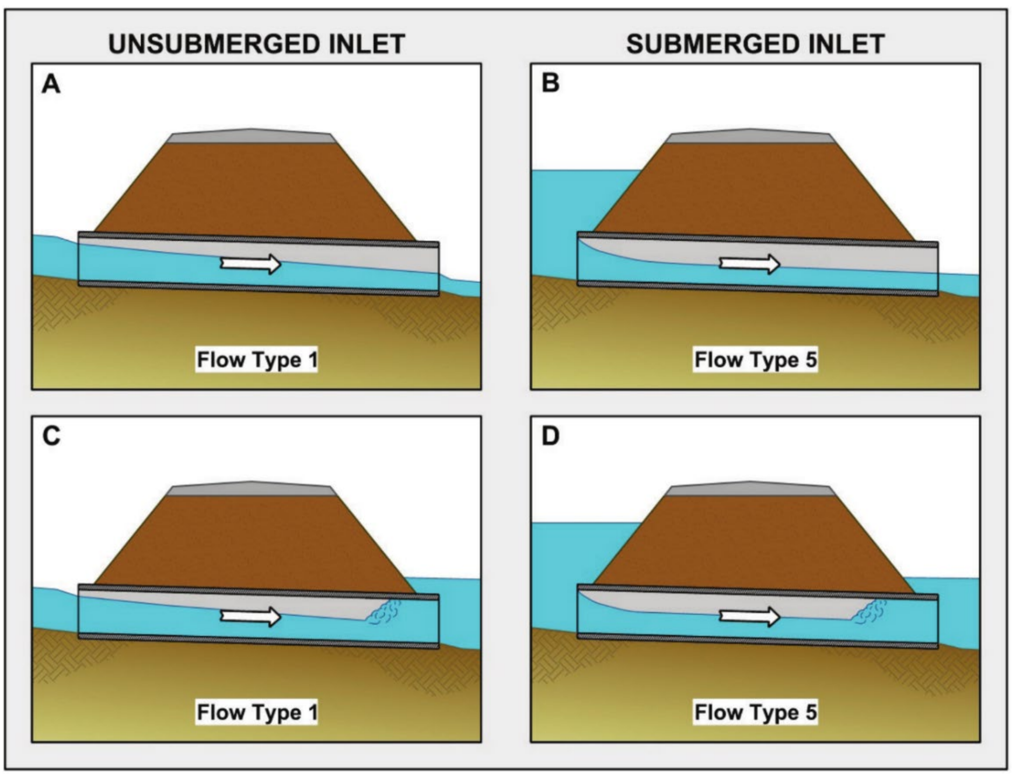
The control section is the location where there is a unique relationship between the flow rate and the upstream water surface elevation. Inlet control is governed by the inlet geometry. Outlet control is governed by a combination of the culvert inlet geometry, the barrel characteristics, and the tailwater or critical depth.

C. Minimum Performance

Minimum performance is assumed by analyzing both inlet and outlet control and using the highest headwater. The culvert may operate more efficiently at times (more flow for a given headwater level), but it will not operate at a lower level of performance than calculated.

D. Inlet Control

Figure 4-1 illustrates the types of inlet control flow. The USGS flow type *(USGS, 1968)* depends on the submergence of the inlet and outlet ends of the culvert. In all of these examples, the control section is at the inlet end of the culvert. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.



Source: HDS-5, 2012

Figure 4-1: Types of Inlet Control

Factors Influencing Inlet Control

Since the control is at the upstream end, only the headwater and the inlet factors affect the culvert performance:

- Headwater depth is measured from the invert of the inlet control section to the surface of the upstream pool.
- Inlet area is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area, but for tapered inlets the face area is enlarged, and the control section is at the throat.
- Inlet configuration describes the entrance type. Some typical inlet configurations are thin edge projecting, mitered, square edges in a headwall, and beveled edge.
- Inlet shape is usually the same as the shape of the culvert barrel; however, it may be enlarged as in the case of a tapered inlet. Typical shapes are rectangular, circular, and elliptical. Whenever the inlet face is a different size or shape than the culvert barrel, the possibility of an additional control section within the barrel exists.
- Barrel slope influences inlet control performance, but the effect is small. Inlet control nomographs assume a slope of 2 percent for the slope correction term (0.5S for most inlet types). This results in lowering the headwater required by 0.01D. In the FHWA computer program HY-8, the actual slope is used as a variable in the calculation.

Hydraulics

Inlet control performance is defined by the three regions of flow shown in Figure 4-2: unsubmerged, transition, and submerged. For low headwater conditions, as shown in Figure 4-1, panels A and C, the entrance of the culvert operates as a weir. A weir is an unsubmerged flow control section where the upstream water surface elevation can be predicted for a given flow rate. The relationship between flow and water surface elevation must be determined by model tests of the weir geometry or by measuring prototype discharges. These tests or measurements are then used to develop equations for unsubmerged inlet control flow. HDS-5, Appendix A *(FHWA, 2012)* contains the equations which were developed from the National Bureau of Standards (NBS) and other model test data.

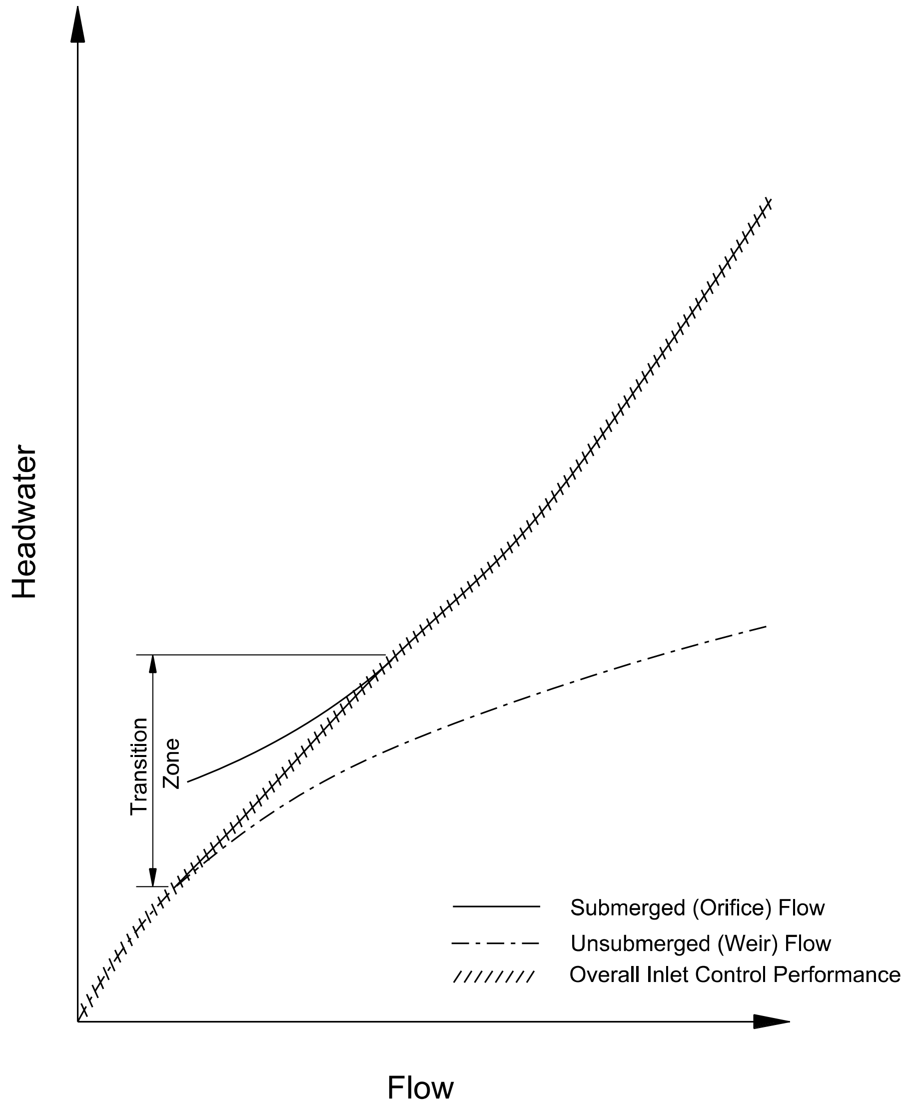


Figure 4-2: Inlet Control Curves

For headwaters submerging the culvert entrance, as shown in Figure 4-1, panels B and D, the entrance of the culvert operates as an orifice. An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side, which functions as a control section. The relationship between flow and headwater can be defined based on results from model tests (see HDS-5 *(FHWA, 2012)*).

The flow transition zone between the low headwater (weir control) and the high headwater (orifice control) flow conditions is poorly defined. This zone is approximated by plotting the unsubmerged and submerged flow equations and connecting them with a line tangent to both curves, as shown in Figure 4-2.

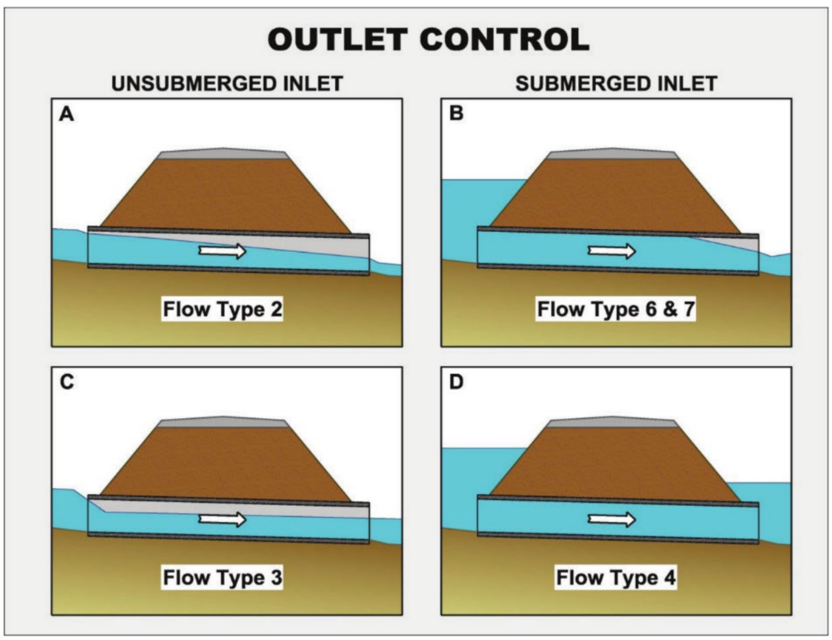
The inlet control flow versus headwater curves which are established using the above procedure are the basis for constructing the inlet control design nomographs and for developing equations used in software. The original equations for computer software were generally 5th order polynomial curve fitted equations that were developed to be as accurate as the nomograph solution (plus or minus 10 percent) within the headwater range of 0.5D to 3.0D. These equations are still being used in HY-8 but have been supplemented with a weir equation from 0.0D to 0.5D and an orifice equation above 3.0D.

Inlet Depression

Inlet depression is created by constructing the entrance inlet below the streambed. The amount of inlet depression is defined as the depth from the natural streambed at the face to the inlet invert. The inlet control equations or nomographs provide the depth of headwater above the inlet invert required to convey a given discharge through the inlet. This relationship remains constant regardless of the elevation of the inlet invert. If the entrance end of the culvert is constructed below the streambed, more head can be exerted on the inlet for the same headwater elevation.

E. Outlet Control

Figure 4-3 illustrates the types of outlet control flow. The USGS flow type *(USGS, 1968)* depends on the submergence of the inlet and outlet ends of the culvert. In all cases, the control section is at the outlet end of the culvert or further downstream. For the partly full flow situations, the flow in the barrel is subcritical.



Source: HDS-5, 2012

Figure 4-3: Types of Outlet Control

Factors Influencing Outlet Control

Since the control is at the downstream end, the headwater is influenced by all of the culvert factors. The inlet factors influencing the performance of a culvert in inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length, and slope) and the tailwater elevation affect culvert performance in outlet control:

- Barrel roughness is a function of the material used to fabricate the barrel. Typical materials include concrete, corrugated metal, and plastic. The roughness is represented by a hydraulic resistance coefficient such as the Manning's "n" value. Select Manning's "n" value from Table 4-13.

- Barrel area is a function of the culvert dimensions. A larger barrel area will convey more flow.

- Barrel shape is a function of culvert type and material. Based on the location of the center of gravity for a given area, a box is the most efficient shape, then the arch shape, followed by the circular shape.

- Barrel length is the total culvert length from the entrance to the exit of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.

- Barrel slope is the actual slope of the culvert barrel. The barrel slope is often the same as the natural stream slope. However, when the culvert inlet is raised or lowered, the barrel slope is different from the stream slope. The slope is not a factor in calculating the barrel losses for USGS Flow Types 4, 6, and 7; but is a factor in calculating USGS Flow Types 2 and 3 when a water surface profile is calculated.

- Tailwater elevation is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define the tailwater elevation.

Hydraulics (Full Barrel Flow)

Full flow in the culvert barrel, as depicted in Figure 4-3, Panel D, is the best flow type for describing the hand computation of outlet control hydraulics. Outlet control flow conditions can be calculated based on an energy balance from the tailwater pool to the headwater pool. The total energy (H_L) required to pass the flow through the culvert barrel is made up of the entrance loss (H_e), the friction losses through the barrel (H_f), and the exit loss (H_v). Other losses, including bend losses (H_b), losses at junctions (H_j), and losses at grates (H_g) should be included as appropriate (**Equation [Eq.] 4.1**). These other losses are discussed in Chapter 5 of HDS-5 *(FHWA, 2012)*.

$$\boxed{H_L = H_e + H_f + H_o + H_b + H_j + \mathrm{H}_g}$$

(Eq. 4.1)
Head Loss

Where:

H_L = total energy losses, ft

H_e = entrance headloss, ft

H_f = friction headloss, ft

H_o = exit headloss, ft

H_b = bend headloss, ft

H_j = headloss at junction, ft

H_g = headloss at grate, ft

The barrel velocity (Eq. 4.2) is calculated as follows:

$$V = Q/A$$

(Eq. 4.2)
Barrel Velocity

Where:

V	=	average barrel velocity, ft/s
Q	=	flow rate, ft^3/s
A	=	cross sectional area of flow with the barrel full, ft^2

The velocity head is:

$$H_v = \frac{V^2}{2g}$$

(Eq. 4.3)
Velocity Head

Where:

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

The entrance loss is a function of the velocity head in the barrel, and can be expressed as a coefficient times the velocity head:

$$H_e = K_e\left(\frac{V^2}{2g}\right)$$

(Eq. 4.4)
Velocity Head

Where:

Ke = entrance loss coefficient (see Table 4-14)

The friction loss in the barrel is also a function of the velocity head. Based on the Manning equation, the friction loss is:

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

(Eq. 4.5)
Friction Loss

Where:

n	=	Manning's roughness coefficient for a culvert with uniform material on the full perimeter (for composite roughness (n_c) see Section 4.14, Table 4-13.
L	=	length of the culvert barrel, ft
A	=	cross-sectional area of the barrel, ft^2
R	=	hydraulic radius of the full culvert barrel = A/P, ft
P	=	wetted perimeter of the barrel, ft
V	=	velocity in the barrel, ft/s

The exit loss is a function of the change in velocity at the outlet of the culvert barrel. For a sudden expansion such as an endwall, the exit loss is:

$$H_o = 1.0\left[\left(\frac{V^2}{2g}\right) - \left(\frac{V_d^2}{2g}\right)\right]$$

(Eq. 4.6)
Exit Loss

Where:

V_d = channel velocity downstream of the culvert, ft/s

Eq. 4.6 may overestimate exit losses, and a multiplier of less than 1.0 can be used (see *HEC-14 FHWA 2006*) for a transition loss. The downstream velocity is usually neglected, in which case the exit loss is equal to the full flow velocity head in the barrel, as shown in Eq. 4.7.

$$H_o = H_v = \frac{V^2}{2g}$$

(Eq. 4.7)
Exit Loss – HEC-14

Eq. 4.7 is the standard option in HY-8. If the hydraulics engineer chooses the Utah State University (USU) Method (which is the alternate in HY-8), the following equation will be used:

$$H_o = \frac{(V - V_d)^2}{2g}$$

(Eq. 4.8)
Exit Loss - USU

Inserting the above relationships for entrance loss (Eq. 4.4), friction loss (Eq. 4.5), and exit loss (Eq. 4.6, 4.7 or 4.8) into Eq. 4.1, the following equation for barrel losses (*H*) is obtained:

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

(Eq. 4.9)
Barrel Losses

Energy Grade Line

Figure 4-4 depicts the energy grade line and the hydraulic grade line for full flow in a culvert barrel. The energy grade line represents the total energy at any point along the culvert barrel. The headwater depth HW_o is the depth from the inlet invert to the energy grade line. The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel straight lines separated by the velocity head except in the vicinity of the inlet where the flow passes through a contraction. The headwater and tailwater conditions as well as the entrance, friction, and exit losses are also shown in Figure 4-4. Equating the total energy at Sections 1 and 2, upstream and downstream of the culvert barrel in Figure 4-4, the following relationship results:

$$HW_o + LS + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L$$

(Eq. 4.10)
Headwater Depth

Where:

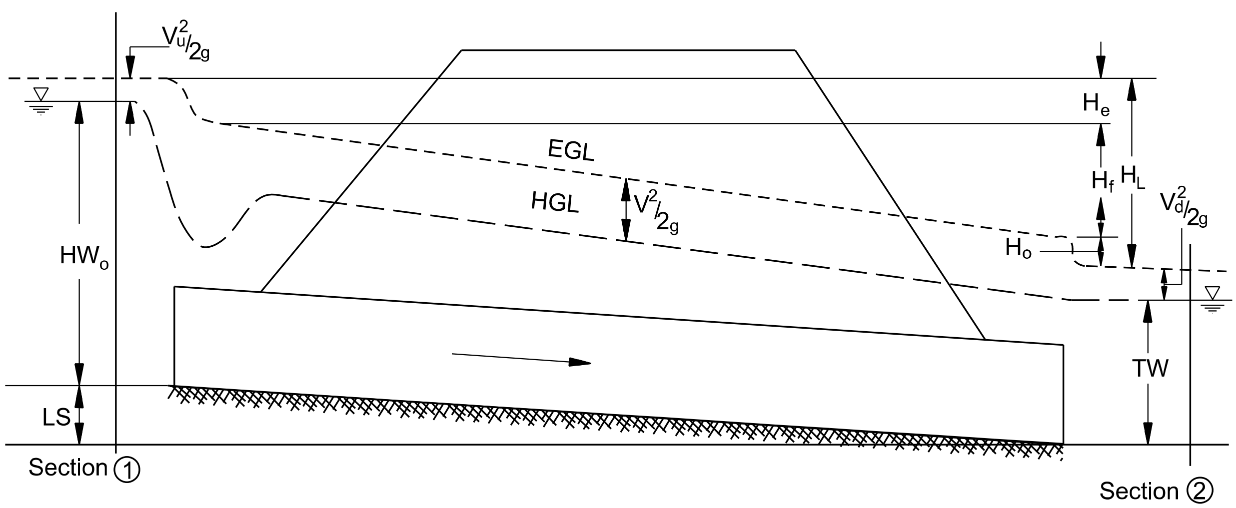
HW_o	=	headwater depth above the outlet invert, ft
V_u	=	approach velocity, ft/s
TW	=	tailwater depth above the outlet invert, ft
V_d	=	downstream velocity, ft/s
H_L	=	sum of all losses (Eq. 4-1)
LS	=	drop through the culvert, ft

NOTE: *The total available upstream headwater (HW_o) includes the depth of the upstream water above the inlet invert and the approach velocity head. In most instances, the approach velocity is low, and the approach velocity head is neglected. However, it can be considered to be a part of the available headwater and used to convey the flow through the culvert.*

Likewise, the velocity downstream of the culvert (V_d) is usually neglected. When both approach and downstream velocities are neglected, Eq. 4.10 becomes:

$$HW_o = TW + H_L - LS$$

(Eq. 4.11)
Headwater Depth - Alt.



Source: HDS-5, 2015

Figure 4-4: Full Flow Energy and Hydraulic Grade Lines

HDS-5 Nomographs (Full Flow)

The nomographs were developed assuming that the culvert barrel is flowing full and:

- TW ≥ D, Flow Type 4 (see Figure 4-3, Panel D); or

- $d_c \geq D$, Flow Type 6 (see Figure 4-3, Panel B).

V_u is small and its velocity head can be considered to be a part of the available headwater (HW_o) used to convey the flow through the culvert. V_d is small and its velocity head can be neglected. Eq. 4.11 is used with the outlet control nomographs to determine outlet control headwater (HW_o).

HDS-5 Nomographs (Partial Full Flow) - Approximate Method

Based on numerous backwater calculations performed by the FHWA staff, it was found that the hydraulic grade line pierces the plane of the culvert outlet at a point approximately $^1/_2$ of the way between critical depth and the top of the barrel, or $(d_c + D)/2$ above the outlet invert. The approximation should only be used if the barrel flows full for part of its length or the headwater is at least 0.75D. If neither of these conditions is met, a water surface profile should be used to establish the hydraulic grade line. *TW* should be used if higher than $(d_c + D)/2$. The following equation should be used:

$$\boxed{HW_o = h_o + H - S_o L}$$

(Eq. 4.12)
Headwater Depth – HDS-5

Where:

h_o = the larger of TW or $(d_c + D)/2$, ft

F. Outlet Velocity

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually result in outlet velocities that are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion. If outlet erosion protection is necessary, the flow depths and Froude number may also be needed.

Inlet Control

The velocity is calculated from Eq. 4.2 after determining the outlet depth. Either of the following methods may be used to determine the outlet depth:

- Calculate the water surface profile through the culvert. Begin the computation at d_c at the entrance and proceed downstream to the exit. Determine at the exit the depth and flow area.

- Assume normal depth and velocity. This approximation may be used because the water surface profile converges towards normal depth if the culvert is of adequate length. This outlet velocity may be slightly higher than the actual velocity at the outlet. Normal depth may be obtained by hand computation or by software (e.g., FHWA Hydraulic Toolbox).

Outlet Control

The cross-sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the conduit:

- Critical depth is used where the tailwater is less than critical depth.

- Tailwater depth is used where tailwater is greater than critical depth but below the top of the barrel.

- The total barrel area is used where the tailwater exceeds the top of the barrel.

G. Roadway Overtopping

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad-crested weir. Flow coefficients for flow overtopping roadway embankments are found in Section 4.15.1, Figure 4-24 (Chart 60B):

$$\boxed{Q_o = \mathrm{C}_d\, L\, \mathrm{HW}_r^{\,1.5}}$$

(Eq. 4.13)
Overtopping Flow Rate

Where:

Q_o = overtopping flow rate, ft^3/s

C_d = overtopping discharge coefficient (weir coefficient) = $k_t\, C_r$

k_t = submergence factor from Figure 4-24

C_r = discharge coefficient from Figure 4-24

L = length of the roadway crest, ft

HW_r = the upstream depth, measured above the roadway crest, ft

Roadway Crest Length
The length is difficult to determine where the crest is defined by a roadway sag vertical curve.

a. Recommend subdividing into a series of segments. The flow over each segment is calculated for a given headwater. The flows for each segment are then added together to determine the total flow.

b. The length can be represented by a single horizontal line (one segment). The length of the weir is the horizontal length of this segment. The depth is the average depth (area/length) of the upstream pool above the roadway.

Total Flow
Total flow is calculated for a given upstream water surface elevation by adding the Culvert Flow (Q) and Overtopping flow (Q_o):

$$\boxed{Q_T = \mathrm{Q} + \mathrm{Q_O}}$$

(Eq. 4.14)
Total Flow Rate

- Roadway overflow plus culvert flow must equal total design flow.
- A trial-and-error process is necessary to determine the flow passing through the culvert and the amount flowing across the roadway.
- Performance curves for the culvert and the road overflow may be summed to yield an overall performance.

Performance Curves

Performance curves are plots of flow rate versus headwater depth or elevation, velocity, or outlet scour. The culvert performance curve consists of the controlling portions of the individual performance curves for each of the following control sections (see Figure 4-5):

- The inlet performance curve is developed using the inlet control nomographs.
- The outlet performance curve is developed using Eqs. 4.1 through 4.11, the outlet control nomographs, or backwater calculations.
- The roadway performance curve is developed using Eq. 4.14.

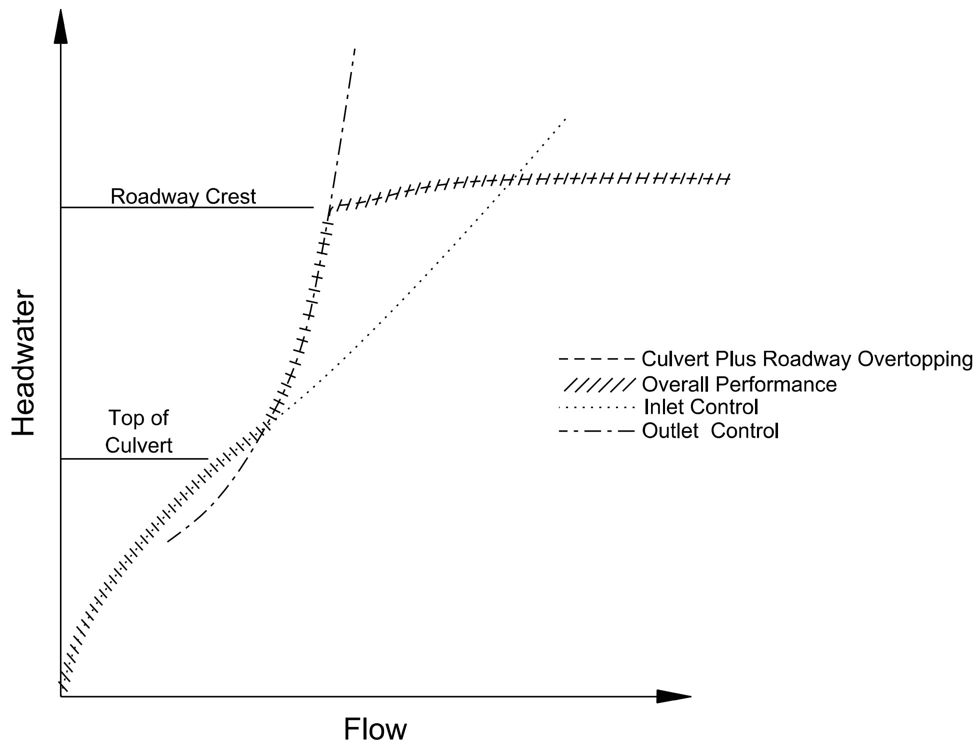


Figure 4-5: Overall Performance Curve

The overall performance curve is the sum of the flow through the culvert and the flow across the roadway. The curve can be determined by performing the following steps:

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

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

Step 3 When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and Eq. 4.7 to calculate flow rates across the roadway.

Step 4 Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve as shown in Figure 4-5.

H. Culvert Design Form

The Culvert Design Form, Figure 4-25 has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description, and the designer's identification. Summaries of hydrologic data of the form are also included. At the top right is a small sketch of the culvert with blanks for inserting important dimensions and elevations.

4.8 OUTFALL PROTECTION

4.8.1 Overview

The failure or damage of many culverts and detention basin outlet structures can be traced to unchecked erosion. Erosive forces, which are at work in the natural drainage network, are often increased by the construction of a highway or by urban development. The interception and concentration of overland flow and constriction of natural waterways inevitably results in an increased erosion potential. To protect the culvert and adjacent areas, all outfalls shall be protected or designed to ensure that outflow velocity during a design storm is not erosive to the downstream channel or slope.

Protection against scour at culvert outlets varies from limited riprap placement to complex and expensive energy dissipation devices. At some locations, use of a rougher culvert material or a flatter slope alleviates the need for a special outlet protection device. Preformed scour holes, approximating the configuration of naturally formed holes, dissipate energy while providing a protective lining to the streambed. Riprapped channel expansions and concrete aprons protect the channel and redistribute or spread the flow. Barrel outlet expansions operate in a similar manner. Headwalls and cutoff walls protect the integrity of the fill. When outlet velocities are high enough to create excessive downstream problems, consideration should be given to more complex energy dissipation devices. These include hydraulic jump-basins, impact basins, drop structures, and stilling wells. Design information for the general types of energy dissipators is provided in FHWA Hydraulic Engineering Circular Number 14 (HEC-14), "Hydraulic Design of Energy Dissipators for Culverts and Channels" *(FHWA, 2006)*.

4.8.2 Erosion Hazards

Erosion at culvert outlets is a common condition under both low flow and high flow conditions. The natural channel flow is usually confined to a lesser width and greater depth as it passes through a culvert barrel. An increased velocity results with potentially erosive capabilities as it exits the barrel. Turbulence and erosive eddies form as the flow expands to conform to the natural channel. However, the velocity and depth of flow at the culvert outlet and the velocity distribution upon reentering the natural channel are not the only factors which need consideration. The characteristics of the channel bed and bank material, velocity, and depth of flow in the channel at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Determination of the local scour potential and channel erodibility is a standard procedure in the design of all highway culverts.

Scour in the vicinity of a culvert outlet can be classified into two separate types: local scour and long-term channel degradation.

A. Local Scour

The first type is called local scour and is typified by a scour hole produced at the culvert outlet. This is the result of high culvert outlet velocities and though the effects extend only a limited distance downstream, the resultant scour hole can compromise the stability of the outfall if left unprotected. Coarse material scoured from the circular or elongated hole is deposited immediately downstream, often forming a low bar. Finer material is transported further downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the peak flow. Methods for predicting scour hole dimensions are found in HEC-14 *(FHWA, 2006)*.

B. Long-Term Channel Degradation

Culverts are generally constructed at crossings of small streams, many of which are eroding to reduce their slopes from daily flows or cyclical-flow events such as spring runoff. This long-term degradation may proceed in a fairly uniform manner over a long length of stream, and it is often exacerbated by severe runoff events. The upstream progression of degradation or erosion, referred to as headcutting, can be detected by location surveys or by periodic maintenance inspections following construction. Information regarding the degree of instability of the outlet channel is an essential part of the culvert site investigation. If substantial doubt exists as to the long-term stability of the channel, measures for protection should be included in the initial construction. FHWA Hydraulic Engineering Circular Number 20 (HEC-20) *(FHWA, 2012)*, “Stream Stability at Highway Structures,” provides procedures for evaluating horizontal and vertical channel stability.

4.8.3 Design Storm Frequency

Design outfall protection to accommodate the same flood frequency as the culvert, as shown in Section 4.4. Where existing pipe outfalls do not pass the design storm, design for full flow conditions.

4.8.4 Design Philosophy

To mitigate this erosion, discharge energy can be dissipated prior to release downstream. The engineer should treat the culvert, energy dissipator and channel protection designs as an integrated system. Energy dissipators can change culvert performance and channel protection requirements. Some debris-control structures represent losses not normally considered in the culvert-design procedure. Velocity can be increased or decreased by changes in the culvert design. Downstream channel conditions (velocity, depth, and channel stability) are important considerations in energy dissipator design.

For some sites, appropriate energy dissipation may be achieved by design of a flow transition, anticipating an acceptable scour hole, and/or allowing for a hydraulic jump given sufficient tailwater. However, at many other sites more involved dissipator designs may be required. These designs generally fall into the following categories:

- Internal Dissipators
- Stilling Basins
- Streambed Level Dissipators
- Riprap Basins and Aprons (preferred designs)
- Drop Structures
- Stilling Wells

4.8.5 Design Procedure

Many outfalls can be adequately protected via the construction of riprap aprons, which is preferred due to low cost and ease of installation. However, physical constraints may limit the ability to use them, and alternative energy dissipation systems may need to be considered. The recommended design procedure involves collecting all relevant data, investigating the use of riprap, and finally, the completion of the HEC-14 *(FHWA, 2006)* design procedure, if necessary.

The purpose of HEC-14 is to provide design procedures for energy dissipator designs for highway applications. It discusses the overall analysis framework that is recommended and provides a step-by-step guide to selecting the appropriate measures. This procedure is comprehensive but can be time-consuming to perform for all possible energy dissipation methods, as it recommends completing the procedure for a single combination of culvert, energy dissipator, and channel protection at a time. It is therefore recommended to select a subset of methods which are likely to be appropriate for the site and compare those in a more focused manner.

Step 1: Identify and Collect Design Data

Step 1 of the design procedure involves collecting all relevant geometric and material data for the culvert, channel, and transition. The Stability Assessment should be performed. An Allowable Scour Estimate should also be performed when a natural (unprotected) outlet is being considered.

Stability Assessment: The channel, culvert, and related structures should be evaluated for stability considering potential erosion plus buoyancy, shear, and other forces on the structure (HEC-14, Chapter 2). If these are assessed as unstable, estimate the depth of degradation or height of aggradation, which will occur over the design life of the structure.

Allowable Scour Estimate: In the field, the hydraulics engineer should determine if the bed material at the planned exit of the culvert is erodible. If yes, the potential extent of scour (i.e., depth, h_s; width, W_s; and length, L_s) should be estimated using the equations in HEC-14 or HY-8. These estimates should be based on the physical limits to scour at the site. For example, the length (L_s) can be limited by a rock ledge or vegetation. The following soil parameters in the vicinity of planned culvert outlets should be provided. For non-cohesive soil, a grain size distribution including D_{16} and D_{84} is needed. For cohesive soil, the values needed are saturated shear strength (S_v) and plasticity index (PI).

Additionally, if the cross section is a trapezoid, it is defined by the bottom width (B) and side slope (Z), which is often expressed as ZH:1V. HDS-4 *(FHWA, 2008)* provides examples of how to compute normal depth in channels. Software such as the FHWA Hydraulic Toolbox, discussed below, can be used to determine tailwater for uniform cross sections.

Step 2: Evaluate Velocities

Compute the culvert exit velocity and compare with the downstream channel velocity to determine if the exit velocity and flow depth approximate the natural flow condition in the downstream channel.

Step 3: Determine Suitability of Rock Outlet Protection

Design guidance for the use of Rock Outlet Protection (riprap aprons) is found in the Maryland Department of Environment's Standards and Specifications for Soil Erosion and Sediment Control *(MDE, 2011)*, section D-4-1. The designer must determine whether the outfall will be governed by the Minimum or Maximum Tailwater Condition and use the appropriate figure to determine the dimensions and nominal material size of the apron. Rock outlet protection designed per MDE Section D-4-1 must have the entire riprap apron placed on 0% slope. When a flat riprap apron is not feasible due to site constraints, aprons on a longitudinal slope can be evaluated using FHWA Hydraulic Toolbox. Designers should use cutoff walls at the downstream end of all riprap aprons.

If the appropriate apron size is too large to fit within the ROW or easement, proceed to Step 4, which corresponds with Step 3 of the HEC-14 design procedure.

Step 4: Evaluate Outlet Scour Hole
Use the HEC-14 *(FHWA, 2006)* procedure or HY-8 software to compute the size of the scour hole and determine whether it is acceptable or whether alternative energy dissipators will be required.

Step 5: Design Alternative Energy Dissipators
Specific Energy Dissipator designs can now be selected based on the design data identified in Step 1 and the attributes and limitations of various energy dissipators listed in HEC-14 Table 1.1 *(FHWA, 2006)*. Debris, tailwater channel conditions, site conditions, and cost must also be considered in selecting alternative designs.

Step 6: Select Energy Dissipator
Compare the design alternatives and select the dissipator that has the best combination of cost and velocity reduction. Each situation is unique, and engineering judgment will always be necessary. The engineer should document the alternatives considered.

4.8.6 Common Energy Dissipator Measures

Energy dissipator designs which are commonly used fall into five broad categories listed in the Table 4-3 below. The designer should consider the following recommendations when performing the alternatives analysis for their site.

Table 4-3: Dissipator Guidelines

DISSIPATOR TYPE	RECOMMENDED WHEN
Natural Scour Holes	• Undermining of the culvert outlet will not occur, or it is practicable to be controlled by a cutoff wall; • The expected scour hole will not cause property damage; and • There is no nuisance effect.
Internal Dissipators	• A scour hole at the culvert outlet is unacceptable; • Right-of-way is limited; • Debris is not expected; and • Moderate velocity reduction is needed.
External Dissipators	• An outlet scour hole is not acceptable; • Moderate amount of debris is present; and • The culvert outlet velocity is moderate, the Froude number ≤ 3.
Stilling Basins	• An outlet scour hole is not acceptable; • Debris is present; and • The culvert outlet velocity is high, the Froude number > 3.
Drop Structures	• The downstream channel is degrading; or • Channel headcutting is present.

The designer must evaluate and select alternatives that best satisfy site conditions, engineering criteria, design policies, and other considerations. The selected dissipator should meet required structural and hydraulic criteria, and the selection should be based on:

- Construction and maintenance costs
- Risk of failure or property damage
- Traffic safety
- Environmental or aesthetic considerations
- Political or nuisance considerations
- Land-use requirements

Some types of energy dissipators may be incompatible with Aquatic Organism Passage (AOP). When AOP is required, the designer must select a method of energy dissipation that does not present a barrier to AOP. Refer to FHWA HEC-26 for additional information.

4.8.7 Design Software

FHWA has produced two freely-available software packages which can automate or provide verification to several aspects of the culvert design and energy dissipator selection process.

HY-8: Software that automates the design methods described in HDS No. 5, "Hydraulic Design of Highway Culverts," HEC No.14, “Hydraulic Design of Energy Dissipators for Culverts and Channels,” and HEC No. 26, “Culvert Design for Aquatic Organism Passage.”

Hydraulic Toolbox: A stand-alone suite of calculators that performs routine hydrologic and hydraulic computations. The program allows a user to perform and save hydraulic calculations in one project file, analyze multiple scenarios, and create plots and reports of these analyses.

4.9 PIPE MATERIALS

4.9.1 Structural Design

A. Categories of Structural Materials: Rigid or Flexible

Pipes can be divided into two broad categories: flexible and rigid. Flexible pipes have little structural bending strength. The materials they are made of, corrugated metal or thermoplastic, can be flexed or distorted significantly without cracking. Flexible pipes depend on support from the backfill to resist deformation. Rigid pipes, primarily concrete pipes, are stiff and do not appreciably deflect.

B. Structural Behavior of Flexible Pipes

A flexible pipe is a composite structure made up of the pipe barrel and the surrounding soil. The barrel and the soil are both vital elements to the structural performance of the pipe. Flexible pipe has relatively little bending stiffness or bedding strength on its own. As loads are applied to the pipe, the pipe attempts to deflect. In the case of round pipe, the vertical diameter decreases and the horizontal diameter increases, as shown in Figure 4-6. When adequate soil support and backfill material are well compacted around the pipe, the increase in the horizontal diameter of the pipe is resisted by the lateral soil pressure. The result is a relatively uniform radial pressure around the pipe, which creates a compressive force in the pipe walls called thrust. To ensure that a stable soil envelope around the pipe is attained during construction, follow the guidelines in MDOT SHA Specification 303.03 for pipe installation.

As vertical loads are applied, a flexible culvert attempts to deflect. The vertical diameter decreases while the horizontal diameter increases. Soil pressures resist the increase in horizontal diameter. The thrust can be calculated, based on the diameter of the pipe and the load placed on the top of the pipe, and is then used as a parameter in the structural design of the pipe.

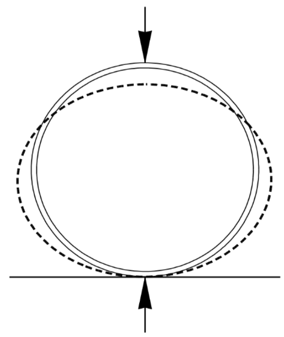


Figure 4-6: Deflection of Flexible Pipes

The flexibility of a pipe also allows for some bend in the horizontal when designing the pipe layout. No bend is allowed in the design. New pipe that exhibits deflection greater than 5% will be rejected.

C. Structural Behavior of Rigid Pipes

The load carrying capability of rigid pipes is essentially provided by the structural strength of the pipe itself, with some additional support given by the surrounding bedding and backfill. When vertical loads are applied to a rigid pipe, zones of compression and tension are created as illustrated in Figure 4-7. Reinforcing steel is added to the tension zones to increase the tensile strength of concrete pipe.

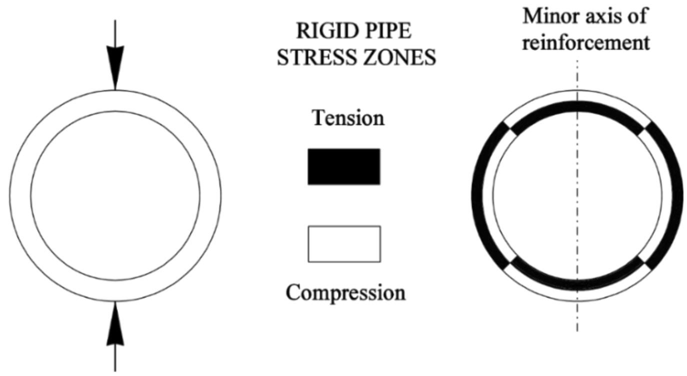


Figure 4-7: Zones of Tension and Compression in Rigid Pipes

Rigid pipe is stiffer than the surrounding soil and it carries a substantial portion of the applied load. Shear stress in the haunch area can be critical for heavily loaded rigid pipe on hard foundations, especially if the haunch support is inadequate. Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials defines proper pipe installation procedures.

D. Foundations, Bedding, and Backfill

A foundation capable of providing uniform and stable support is important for both flexible and rigid pipes. The foundation must be able to uniformly support the pipe at the proposed grade and elevation without concentrating the load along the pipe. Establishing a suitable foundation requires removal and replacement of any hard spots or soft spots that would result in load concentration along the pipe.

Bedding is needed to level out any irregularities in the foundation and to ensure adequate compaction of the backfill material. (See the Standard Specifications for Construction and Materials (Section 303) Backfilling for guidelines.) Any trenching conditions not described in the Contract Plans or the MDOT SHA Standard Specifications for Construction and Materials require a design exception from HHD.

When using flexible pipes, the bedding should be shaped to provide support under the haunches of the pipe. When using rigid pipe, the bedding should be shaped to provide uniform support under the haunches and also shaped to provide clearance for the bell ends on bell and spigot type pipe.

The bedding and backfill must also be installed properly to prevent piping from occurring. Piping is a term used to describe the movement of water around and along the outside of a pipe, washing away backfill material that supports the pipe. Piping is primarily a concern in culvert applications, where water at the culvert inlet can saturate the embankment and move into the pipe zone. Piping may be prevented through the use of headwalls, dikes, or plugs. Headwalls are described in Section 4.6.6.

4.9.2 Selection

The selection of pipe material is dependent upon various factors such as: durability, installed structural performance, hydraulic roughness, flow velocity, bedding conditions, abrasion and corrosion resistance, water tightness requirements, and installation considerations such as steep slopes and access.

4.9.3 Approved Materials

The following pipe materials are approved as noted. It is the designer's responsibility to ensure that the selected material meets all the required needs.

A. Reinforced Concrete Pipe (RCP), Reinforced Concrete Pressure Rated Pipe (RCPP) and Horizontal Elliptical Reinforced Concrete Pipe (HERCP)

a. **RCP** - Specification: AASHTO M-170, Class IV minimum

   Joints shall be bell and spigot with watertight gaskets conforming to ASTM C-443.

   - Spans: 15” to 54”
   - Spans 60” and larger must be approved by the Office of Structures

b. **RCPP** - Specification: ASTM C-361 Low-Head Pressure rated pipe is required for pond spillways subject to Code 378

c. **HERCP** - Specification: AASHTO M-207 Horizontal Elliptical only

   Joints shall be bell and spigot and sealed conforming to ASTM C-990.

   - Sizes: up to 53” X 34”
   - Span dimensions 60” and larger must be approved by Office of Structures

d. If water soluble chlorides exceed 400 ppm, protective measures are necessary.

e. If soils, as indicated in the NRCS Web Soil Survey, have a high corrosion potential, additional protective measures may be necessary. The rate of concrete corrosion depends on texture, occurrence of organic horizons, pH, and the amounts of magnesium and sodium sulfate or sodium chloride in the saturation extract (NRCS Soil Survey Manual).

f. Refer to Section 4.9.4 of this manual for cover requirements.

g. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

B. Corrugated Polyethylene Pipe (CPP)

a. Specification: AASHTO M-294, Type 'S' (smooth interior) and Type ‘D’ (smooth interior/exterior).

b. Spans: 15” to 48".

c. Joints shall be bell and spigot with watertight joints as per D3212.

d. Refer to Section 4.9.4 of this manual for cover requirements.

e. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

f. Corrugated Polyethylene Pipe is under evaluation by the MDOT SHA and is not currently allowed under Interstates and Principal Arterials; however, it is permitted under minor arterials, collectors, local roads, driveway entrances, and outside the loading influence of all roadway functional classifications. Corrugated Polyethylene Pipe may be considered under Principal Arterials under unique conditions such as emergency use and material supply issues with the approval of the Directors of the Office of Materials Technology and the Office of Construction.

C. Corrugated Polypropylene Drainage Pipe (CPDP)

a. Specification AASHTO M-330.

b. Spans: 15” to 48".

c. Joints shall be bell and spigot with watertight joints as per D3212.

d. Refer to Section 4.9.4 of this manual for cover requirements.

e. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

f. Corrugated Polypropylene Drainage Pipe is under evaluation by the MDOT SHA and is not currently allowed under Interstates and Principal Arterials; however, it is permitted under minor arterials, collectors, local roads, driveway entrances, and outside the loading influence of all roadway functional classifications. Corrugated Polypropylene Drainage Pipe may be considered under Principal Arterials under unique conditions such as emergency use and material supply issues with the approval of the Directors of the Office of Materials Technology and the Office of Construction.

D. Polyvinyl Chloride Profile Wall (PPWP)

a. Specification: AASHTO M-304.

b. Spans: 15" to 36".

c. Joints shall be bell and spigot with watertight joints as per D3212.

d. Refer to Section 4.9.4 of this manual for cover requirements.

e. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

f. Polyvinyl Chloride Profile Wall Pipe is under evaluation by the MDOT SHA and is not currently allowed under Interstates and Principal Arterials; however, it is permitted under minor arterials, collectors, local roads, driveway entrances, and outside the loading influence of all roadway functional classifications. Polyvinyl Chloride Profile Wall Pipe may be considered under Principal Arterials under unique conditions such as emergency use and material supply issues with the approval of the Directors of the Office of Materials Technology and the Office of Construction.

E. Corrugated Steel Pipe (CMP), Pipe-Arch (CMPA) and Spiral Rib Pipe (SRP)

a. Specification: AASHTO M-36 and AASHTO M-245.

b. Minimum Thickness: 14 gage under roadways, 16 gage under entrances.

c. Coatings: Aluminized (Type 2) as per AASHTO M-274 is to be used under roadways. Additional/alternative protective coatings may be used on a case-by-case basis.

d. Joints shall have rubber O-ring gaskets and be silt tight.

e. Spans: 15" to 54".

f. Spans 60” and larger must be approved by the Office of Structures.

g. Soil and water pH testing are required for installations beneath the roadway. Acceptable pH range is from 5.5 to 8.5. Minimum soil resistively shall be in excess of 1500 ohm-cm.

h. Abrasion must be considered in the Service Life computations.

i. Refer to Section 4.9.4 of this manual for cover requirements.

j. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

k. Not for use as stormwater management pond riser structures or spillways.

F. Corrugated Aluminum Pipe (CAP), Pipe-Arch (CAPA), and Spiral Rib Pipe (ASRP)

a. Specification: AASHTO M-196.

b. Joints shall have rubber O-ring gaskets and be silt tight.

c. Spans: 15" to 54".

d. Spans 60” and larger must be approved by the Office of Structures.

e. Soil and water pH testing are required for installations beneath the roadway. Acceptable pH range is from 5.5 to 8.5. Minimum soil resistively shall be in excess of 1500 ohm-cm.

f. Refer to Section 4.9.4 of this manual for cover requirements.

g. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

h. Not for use as stormwater management pond riser structures or spillways.

G. Steel Structural Plate Pipe and Pipe Arch (SPP, SPPA)

a. Specification: AASHTO M-197.

b. Spans: 60" to 96" must be approved by the Office of Structures.

c. Bottom plate shall be 1 gage thicker than the rest of the structure.

d. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

H. Aluminum Plate Pipe and Pipe Arch (APP, APPA)

a. Specification: AASHTO M-219.

b. Spans: 60” to 96” must be approved by the Office of Structures.

c. Bottom plate shall be 1 gage thicker than the rest of the structure.

d. Refer to Section 303.03 of the MDOT SHA Standard Specifications for Construction and Materials for bedding and backfill requirements.

I. Alternative Materials

Other pipe materials will be considered on a case-by-case basis subject to approval by the Highway Hydraulics Division Chief.

4.9.4 Pipe Cover Requirements

A. Minimum Pipe Cover

Minimum cover requirements for common pipe materials are provided in Tables 4-4, 4-5, and 4-6. During construction, an absolute minimum of two feet and preferable minimum of three feet of cover must be maintained in locations where heavy equipment is anticipated to operate over any pipe installation. When cover criteria cannot be met underneath the roadway, a structurally adequate design must be developed in accordance with AASHTO LRFD Design Guidance *(AASHTO, 2020)* and a design exception request must be approved by the Highway Hydraulics Division Chief. When cover criteria cannot be met for driveways, every effort should be made to achieve minimum cover and any deviations from the minimum requirements will require prior approval from the responsible MDOT SHA Engineer.

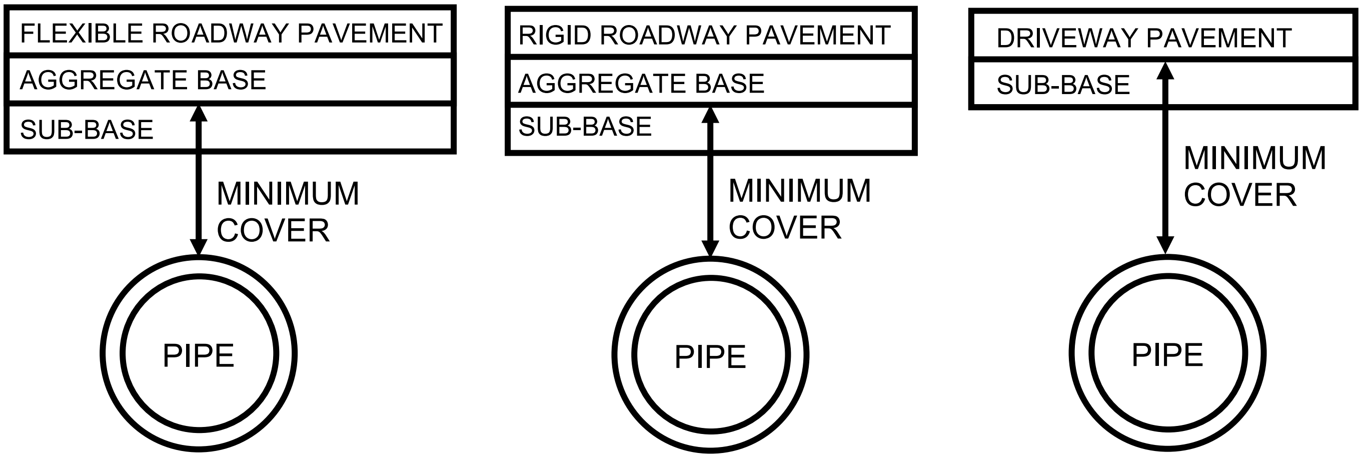


Figure 4-8: Pipe Cover Requirements

Table 4-4: Concrete Pipe (Round and Elliptical)

MINIMUM COVER (IN)		
FLEXIBLE ROADWAY PAVEMENT	RIGID ROADWAY PAVEMENT	UNPAVED AREAS AND DRIVEWAYS
12	9	12

Table 4-5: Plastic Pipe

PIPE TYPE	INNER DIAMETER	MINIMUM COVER (IN)	
		UNDER ROADWAYS AND COMMERCIAL DRIVEWAYS	UNPAVED AREAS AND RESIDENTIAL DRIVEWAYS
Polyethylene, Polypropylene, Polyvinyl Chloride Profile Wall	≤ 24"	18	12
	24"<d≤48”	24	24

Table 4-6: Metal Pipe

PIPE TYPE	INNER DIAMETER	MINIMUM COVER (IN)	
		UNDER ROADWAYS AND COMMERCIAL DRIVEWAYS	UNPAVED AREAS AND RESIDENTIAL DRIVEWAYS
Corrugated Metal Pipe	≤ 60"	12	12
Spiral Rib Steel Pipe	≤ 48”	12	12
	48"<d≤54”	15	15
Spiral Rib Aluminum Pipe	≤ 24”	15	12
	24"<d≤54”	24	24

B. Maximum Pipe Cover

Maximum fill height is dependent upon pipe material and loading. It is the responsibility of the designer to ensure maximum fill height is not exceeded in accordance with AASHTO LRFD Design Guidance *(AASHTO, 2020)* and the MDOT SHA Standards and Specifications for Construction and Materials.

4.10 PIPE CULVERT SERVICE LIFE

4.10.1 Overview

A 100-year minimum pipe service life is required for Interstates and Principal Arterials. A 50-year minimum pipe service life is required for all other highway functional classifications; however, a 75-year minimum pipe service life is required for situations where the depth of pipe cover exceeds 10 feet. The projected service life of a culvert is the duration of service time before significant deterioration is predicted to occur. At this point, maintenance, major rehabilitation, lining or replacement should be considered.

The prediction of service life of drainage facilities is difficult because of the large number of variables, continuing changes in materials, wide range of environments, and use of various protective coatings. The projected service life of a drainage asset is defined as the duration of time the asset is anticipated to remain in a state of good repair.

Environmental conditions that factor into the projected service life include soil and water chemistry, pH, and resistivity (corrosion), presence of baseflow and anticipated fines (abrasion), and UV exposure.

For corrugated metal pipes (CMP), projected service life, with respect to corrosion, abrasion and/or durability, is the anticipated number of years from installation until the deterioration reaches the point of perforation at any location on the culvert.

For reinforced concrete pipe culverts (RCP), projected service life, with respect to corrosion, abrasion and/or durability, is the anticipated number of years from installation until the deterioration reaches the point of exposed reinforcement or a 0.1 in or larger longitudinal crack develops at any point on the culvert.

Concrete and metal culverts are subject to deterioration from corrosion, abrasion, or both. Corrosion may result from active elements in the soil, water and/or atmosphere. Mechanical wear depends upon the frequency, duration and velocity of flow, and the amount and character of bedload. To assure that the projected service life is achieved, metal pipe may require added thickness and/or protective coatings. Concrete pipe may require extra thickness of concrete cover over the steel reinforcement, high density concrete, and/or protective coatings.

Thermoplastic pipe culverts (CPP, CPDP, and PPWP) are subject to deterioration from abrasion and ultraviolet (UV) light. The service life ends when deflection exceed 10 percent of vertical diameter, or a crack appears in the pipe that is extensive enough to impair the integrity of the barrel ring in compression or permit infiltration of groundwater or backfill. Plastic pipes sometimes crack from initial field loadings but can also crack through a creep / rupture mechanism called slow crack growth.

It is the responsibility of the designer to verify that the chosen pipe material has an adequate projected service life for field conditions.

4.10.2 Service Life Computation

Culvert installations where soil and water chemistry fall within any limitations set forth in Section 4.9.3 have an estimated material service life as shown in the following table:

Table 4-7: Estimated Material Service Life

PIPE MATERIAL	ESTIMATED MATERIAL SERVICE LIFE
Reinforced Concrete (RCP/RCPP)	75-100 Years
Polyethylene (CPP)	75-100 Years
Polypropylene (CPDP)	75-100 Years
PVC Profile Wall (PPWP)	75-100 Years
Aluminum	50-75 Years
Aluminized Steel	25-50 Years
Galvanized Steel	15-25 Years

It is the designer's responsibility to consider the environmental conditions and demonstrate that the material is adequate and/or what additional protective measures should be implemented to provide the expected service life. An evaluation of soil and water chemistry to estimate service life must be conducted for locations where metal pipes are proposed beneath the roadway and when any of the following conditions exist – culvert span of 36" or greater in a coastal, wetland, or stream environment (perennial or intermittent), and installations in an area of documented premature pipe material failure. An evaluation of soil and water chemistry is not required for driveway culvert applications. Refer to the MDOT SHA Pavement and Geotechnical Design Guide and coordinate with the Office of Materials Technology for further information on soil and water chemistry testing. Soil maps such as the NRCS Web Soil Survey may be used to determine suitable materials in lieu of site-specific testing for projects installing a small amount of pipe.

The computer program Culvert Service Life Estimator (CSLE) is a suggested tool developed by the Florida Department of Transportation for estimating the service life of culvert materials. The tool requires inputs for pH, resistivity, chlorides, and sulfates.

4.11 INSPECTION

4.11.1 Introduction

The responsibility for pipe asset management and programmatic inspection is divided between the Highway Hydraulics Division (HHD) within the Office of Highway Development and the Structures Inspection and Remedial Engineering Division (SIRED) within the Office of Structures. The Structures Inspection and Remedial Engineering Division is responsible for inventory and inspection of pipes that meet any of the following criteria:

1. Any pipe with an opening equal to or greater than 5 feet and less than or equal to 20 feet with any depth of fill.

2. Any pipe with an opening equal to or greater than 3 feet and less than 5 feet with a depth of fill less than the largest pipe opening.

3. Multiple pipes carrying the same stream if the pipes have openings greater than 3 feet AND

a. The clear distance between pipes is greater than half the smallest pipe opening and there's a depth of fill less than the largest pipe opening, or

b. The clear distance between pipes is equal to or less than half the smallest pipe opening and there's a depth of fill less than the largest pipe opening, but an overall out-to-out width less than 20 feet.

The Highway Hydraulics Division is responsible for inventory of all pipes that do not meet the above criteria. The inspection guidance henceforth pertains to these pipes.

4.11.2 Asset Management Overview

To protect roadway infrastructure from damage, existing pipes must be periodically inspected. It takes a substantial effort from many stakeholders to inspect and maintain the thousands of pipes within the MDOT SHA inventory. Pipe inspection may occur during routine maintenance activities, during project development, or programmatically. District maintenance personnel will often observe issues of concern in the field and report these issues to the Highway Hydraulics Division for further inspection. During project development, drainage pipes in the vicinity of the project limits must be inspected at planning or concept stages to determine levels of corrosion, spalling, abrasion, piping, misalignment, deflection (deformation), obstructions, and to evaluate outfall conditions. The results of this inspection are evaluated by the project team to determine whether corrective action is necessary and whether any pipe rehabilitation or replacement should be incorporated into the project. The Office of Highway Development is responsible for the programmatic inspection of drainage assets including pipes.

4.11.3 Inspection Methods

Inspection may be done manually or with remote-controlled video equipment. Results must be documented for further evaluation by the engineer.

4.11.4 Pipe Rating

Engineers should document the observed condition of existing pipes after inspection. It is suggested that the Pipe Condition Rating Form (Figure 4-27) in Section 4.14 is used to document pipe condition and facilitate the decision-making process for the handling of existing pipes per Section 4.11.5. A summary of the pipe condition rating system is shown in Table 4-8.

Table 4-8: Pipe Condition Ratings

CONDITION	APPROXIMATE REMAINING SERVICE LIFE
No Rating	Pipe not rated
Poor	Less than 2 years
Marginal	Between 2 and 10 years
Fair	Between 10 and 20 years
Good	Greater than 20 years
Excellent	Pipe has nearly full material service life remaining

4.11.5 Repair or Replacement

Pipes will typically be repaired, rehabilitated, or replaced (as recommended by the engineer) if in poor, marginal, or fair condition. Pipes in good or excellent condition are expected to have sufficient remaining service life to safely function until addressed by a future project or programmatic activity. Pipes that are unable to be properly inspected shall be reported to the engineer for further investigation.

Once a pipe has been identified as needing attention, the decision whether that such structure should be totally replaced versus undertaking a repair and rehabilitation program needs to be made by the engineer. Factors affecting this decision include the extent of damage and distress the pipe has experienced; site conditions; the effect on the travelling public for replacement work as opposed to any repair or rehabilitation efforts; the relative costs of either approach; availability of funding; functionality and safety issues applicable to each type of undertaking; and estimation of the remaining service life of the existing pipe, whether the rehabilitation or repairs are made or not. Local safety codes and regulations, combined with any state or Federal guidelines, stipulations, or requirements, are all factors influencing the scope of work involved.

Often, the work associated with repair or rehabilitation of a pipe is significantly less expensive, less time consuming, and less disruptive to the travelling public than what is involved with total replacement of that pipe. Eliminating costly and time-consuming detours, combined with the inherent functionality and safety advantages to keeping the roadway open, may be key advantages to a repair/rehabilitation-versus-replacement decision. Conserving material resources and preserving budget dollars for other infrastructure needs in the local community are other benefits associated with keeping the existing pipe in service and simply repairing (i.e., fixing joints) or rehabilitating (i.e., relining) it. The cost and time associated with engineering services for a new pipe (i.e., replacement) versus those associated with a repair or rehabilitation undertaking are yet other factors to consider.

If a pipe is deemed to be structurally, geometrically, and hydraulically functional, needing only some relatively minor repairs to ensure continued satisfactory performance and a reasonable service life, rehabilitation is a logical approach.

4.12 PIPE REHABILITATION

In those cases where repair or rehabilitation of the existing pipe is considered feasible and will meet the intended lifespan of the pipe, remediation of the deficiencies of the existing pipe may be employed rather than replacement of the pipe. FHWA Publication FHWA-CFL/TD-05-003, Culvert Pipe Liner Guide and Specifications, is an excellent resource for more in-depth explanations of many pipe-related problems and repair and rehabilitation techniques named here.

Newly lined pipes shall be hydraulically reanalyzed to determine if capacity and headwater elevation have been affected. This includes both flow characteristics and inlet characteristics.

4.12.1 Reinforced Concrete Pipe (RCP)

The following remediation techniques assume the pipe has reached equilibrium and is structurally capable of supporting the loads it is subjected to. If not, stabilization must first be achieved. A non-structurally sound pipe typically must be replaced.

Table 4-9: Remediation Options for RCP

OBSERVED DEFECT	REMEDIATION OPTIONS
Longitudinal Cracking < 0.01”	• No repair necessary
Longitudinal Cracking ≥ 0.01” and < 0.1”	• No repair if pipe is located within non-corrosive environment. • Repairs with Portland cement mortar or epoxy materials should be considered for pipes in coastal or riverine sites.
Longitudinal Cracking ≥ 0.1” and < 0.2”	• Pipe requires structural evaluation. Cracks may be repaired with Portland cement mortar or structural epoxy materials.
Slabbing & Spalling	• Patch with Portland cement mortar or epoxy materials.

Infiltration / Exfiltration due to Joint Separation	• Chemical grouting
	• Stainless steel or PVC sleeves
	• Rubber gasket with stainless steel banding
	• Liner Pipe Rehabilitation:
	  ○ Plastic
	  ○ Spiral Wound
	  ○ Other liner type/material approved by the MDOT SHA
	• Steel Reinforced Thermoplastic Ribbed Pipe

Chemical Erosion	• Apply chemical resistant spray-on coating after patching or cement mortar lining. • Conduit rehabilitation using Resin Based Liner, cured-in-place-pipe (CIPP). • Liner Pipe Conduit rehabilitation using Spiral Wound Liner. • Steel Reinforced Thermoplastic Ribbed Pipe.
Deteriorated Invert	• Invert paving with Portland cement concrete • Conduit rehabilitation using Spray Applied Structural Liner Liner Pipe • Conduit Renewal Using Spiral Wound Liner • Steel Reinforced Thermoplastic Ribbed Pipe

Semi-structural Renewal Methods:

Table 4-10: Remediation Options for Semi-Structurally Deficient RCP

OBSERVED DEFECT	REMEDIATION OPTIONS
Semi-Structurally Deficient Requiring Pipe Rehabilitation / Replacement	• Pipe rehabilitation using Spray Applied Structural Liner • Liner Pipe • Segmental lining, stainless steel or PVC sleeves • Pipe rehabilitation using Spiral Wound Liner • Steel Reinforced Thermoplastic Ribbed Pipe

4.12.2 Flexible Pipe

Generally, a structurally sound pipe can be rehabilitated whereas a non-structurally sound pipe typically must be replaced. Common flexible pipe rehabilitation options are listed in Tables 4-11 and 4-12.

A. Steel & Aluminum Pipe

Table 4-11: Remediation Options for Metal Pipe

OBSERVED DEFECT	REMEDIATION OPTIONS
Semi-Structurally Deficient Requiring Pipe Rehabilitation / Replacement	• Pipe rehabilitation using Spray Applied Structural Liner • Liner Pipe • Pipe rehabilitation using Spiral Wound Liner • Steel Reinforced Thermoplastic Ribbed Pipe • Structural steel pipe liner • Tunnel liner plate • Pipe jacking • Pipe bursting
Corrosion – Light to No Invert Perforations	• Invert paving with Portland cement concrete • Pipe rehabilitation using Resin Based Liner
Corrosion – Heavy Perforations to the Pipe and/or Invert	• Invert paving with Portland cement concrete • Pipe renewal using Resin Based Liner • Pipe renewal using Spray Applied Structural Liner, cured-in-place-pipe (CIPP) • Liner Pipe • Pipe renewal using Spiral Wound Liner • Steel Reinforced Thermoplastic Ribbed Pipe
Infiltration / Exfiltration	• Portland cement grout • Chemical grouting • Cured-in-place-pipe (CIPP) • Liner Pipe • Pipe rehabilitation using Spiral Wound Liner • Steel Reinforced Thermoplastic Ribbed Pipe

B. Plastic Pipe

Table 4-12: Remediation Options for Plastic Pipe

OBSERVED DEFECT	REMEDIATION OPTIONS
Cracks	• Welding • Welding with sheet reinforcing • Chemical grouting • Stainless steel or PVC sleeves • Cured-in-place-pipe (CIPP) - UV cured only
Infiltration / Exfiltration due to Joint Separation	• Chemical grouting • Stainless steel or PVC sleeves • Rubber gasket with stainless steel banding • Cured-in-place-pipe (CIPP) - UV cured only

4.13 REFERENCES

AASHTO LRFD Bridge Design Specifications, Chapter 12, Buried Structures and Tunnel Liner, 2020

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4.14 DESIGN AIDS

This section presents several tables, figures, and forms required for the hydraulic design of culverts. These include:

- Table 4-13: Manning's "n" values that have been determined in the laboratory are provided in Table 4-13 with the recommended design "n" value. Culvert materials are either treated as smooth or a corrugated. In this way, alternative materials can be substituted for a given structure.
- Table 4-14: Entrance Loss Coefficients (k_e).
- The following culvert nomographs for circular and rectangular shapes are included; see HDS-5 *(3)* for other culvert nomographs:
    - Figure 4-9 Headwater Depth for Concrete Pipe Culverts with Inlet Control,
    - Figure 4-10 Headwater Depth for C. M. Pipe Culverts with Inlet Control,
    - Figure 4-11 Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control,
    - Figure 4-12 Critical Depth (Circular Pipe),
    - Figure 4-13 Head for Concrete Pipe Culverts Flowing Full (n = 0.012),
    - Figure 4-14 Head for Standard C. M. Pipe Culverts Flowing Full (n = 0.024),
    - Figure 4-15 Head for Structural Plate Corrugated Metal Pipe Culverts Flowing Full (n = 0.0328 to 0.0302),
    - Figure 4-16 Headwater Depth for Box Culverts with Inlet Control,
    - Figure 4-17 Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls 18º to 33.7º and 45º with Beveled Edge at Top of Inlet,
    - Figure 4-18 Headwater Depth for Inlet Control, Rectangular Box Culverts, 90º Headwall, Chamfered or Beveled Inlet Edges,
    - Figure 4-19 Headwater Depth for Inlet Control, Single Barrel Box Culverts, Skewed Headwalls, Chamfered or Beveled Inlet Edges,
    - Figure 4-20 Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls, Normal and Skewed Inlet Edges, ¾" Chamfer at Top of Opening,
    - Figure 4-21 Headwater Depth for Inlet Control, Rectangular Box Culverts, Offset Flared Wingwalls and Beveled Edge at Top of Inlet,
    - Figure 4-22 Critical Depth (Rectangular Section),
    - Figure 4-23 Head for Concrete Box Culverts Flowing Full (n = 0.012), and
    - Figure 4-24 Discharge Coefficients for Roadway Overtopping.

- The following design forms are presented for hand calculations for the hydraulic design of culverts:
    - Figure 4-25 Culvert Design Form, which is the standard form used for culverts. The procedure in Section 11.3 is based on this form.
    - Figure 4-26 Side/Slope Tapered Design Form.
    - Figure 4-27 Pipe Condition Rating Form.

Table 4-13: Manning's "n" Values for Pipes

TYPE OF CONDUIT	WALL DESCRIPTION	MANNING'S *n* LABORATORY[1]	DESIGN VALUE
Concrete Pipe	Smooth	0.010-0.011	0.013
Concrete Boxes	Smooth	0.012-0.015	0.013
Spiral Rib Metal Pipe	Smooth walls	0.012-0.013	0.013
Corrugated Metal Pipe, Pipe-Arch	2⅔ in. × ½ in. Annular	0.022-0.027	0.024
	2⅔ in. × ½ in. Helical	0.011-0.023	0.024
	6 in. × 1 in. Helical	0.022-0.025	0.024
	5 in. × 1 in.	0.025-0.026	0.024
	3 in. × 1 in.	0.027-0.028	0.024
	6 in. × 2 in. Structural Plate	0.033-0.035	0.035
	9 in. × 2½ in. Structural Plate	0.033-0.037	0.035
Corrugated Polyethylene (CPP-S)	Type S Smooth	0.009-0.015	0.013
Corrugated Polyethylene (CPP-C)	Type C Corrugated	0.018-0.025	0.024
Polyvinyl Chloride Profile Wall Pipe (PPWP)	Type S Smooth	0.009-0.011	0.013
Corrugated Polypropylene (CPDP)	Smooth		0.013

[1]*Source: HDS-5 (1)*

Design Values to be used for all design purposes. Laboratory values are provided for context only.

Table 4-14: Entrance Loss Coefficients (Outlet Control, Full or Partly Full)

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

Source: HDS-5 (1)

[1] *"End section 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.*

CHART 1B

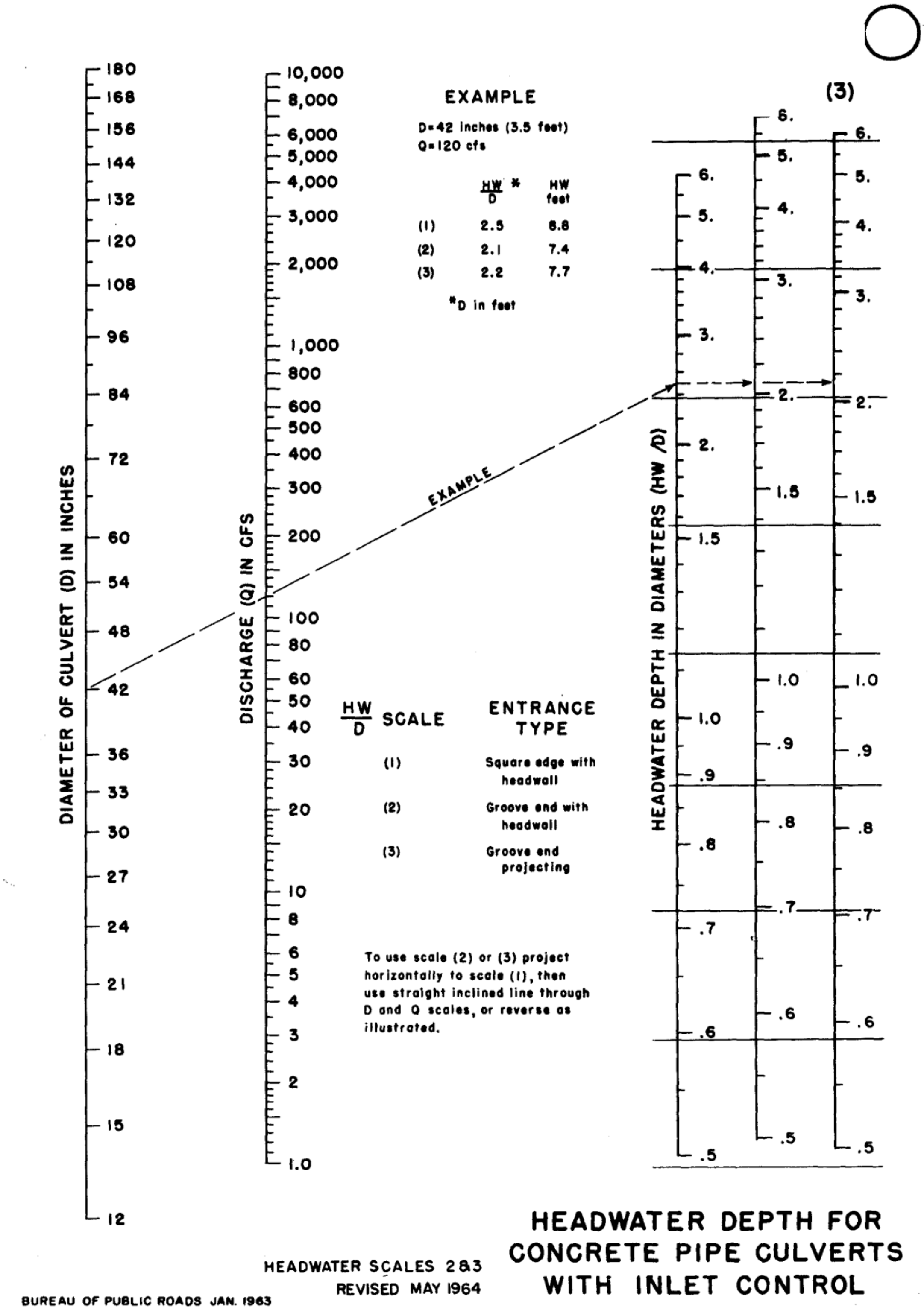


Figure 4-9: Headwater Depth for Concrete Pipe Culverts under Inlet Control (Chart 1B)

CHART 2B

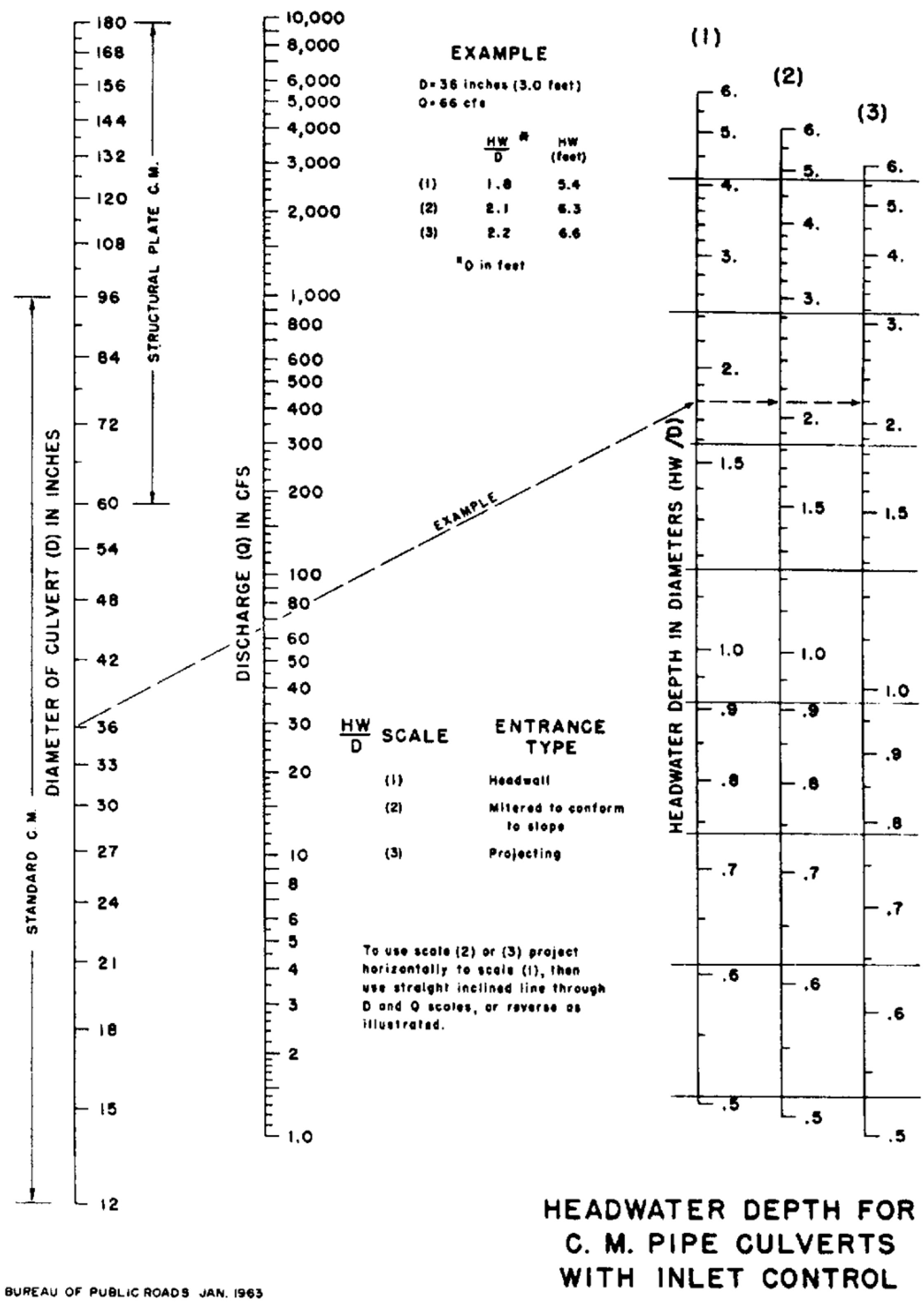


Figure 4-10: Headwater Depth for CMP Culverts under Inlet Control (Chart 2B)

CHART 3B

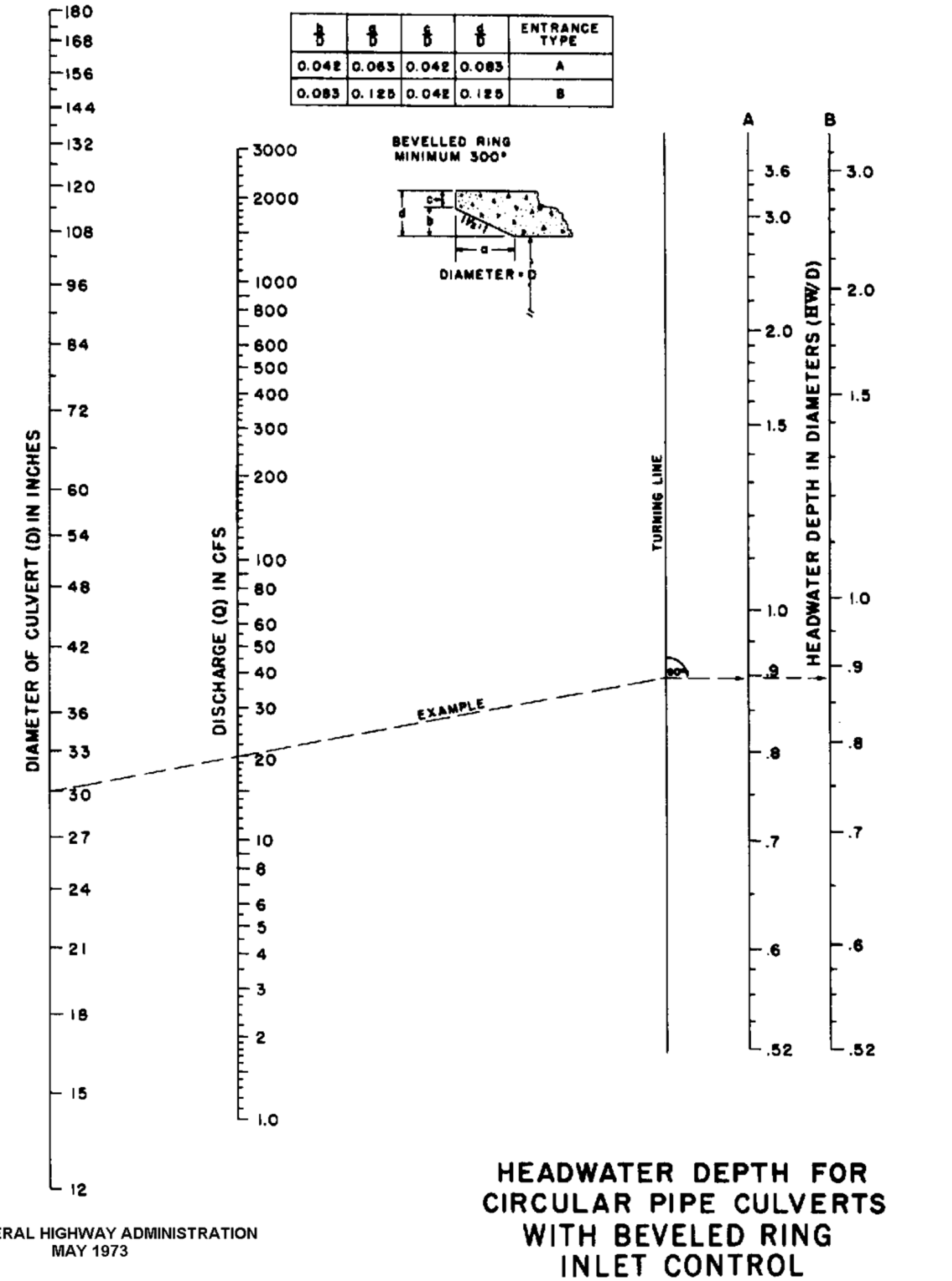


Figure 4-11: Headwater Depth for Circular Pipe Culverts with Beveled Ring Inlet Control (Chart 3B)

CHART 4B

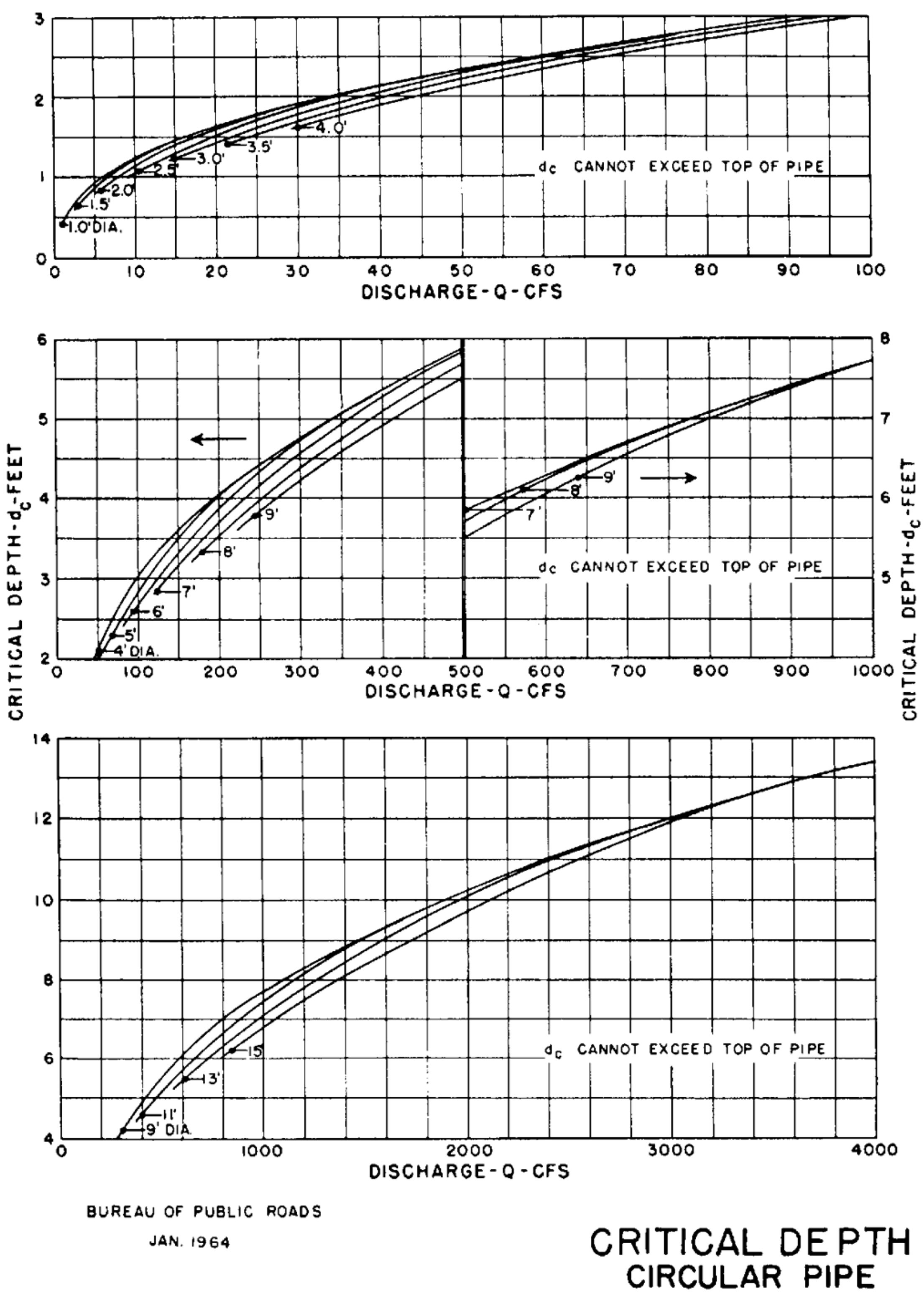


Figure 4-12: Critical Depth (Circular Pipe) (Chart 4B)

CHART 5B

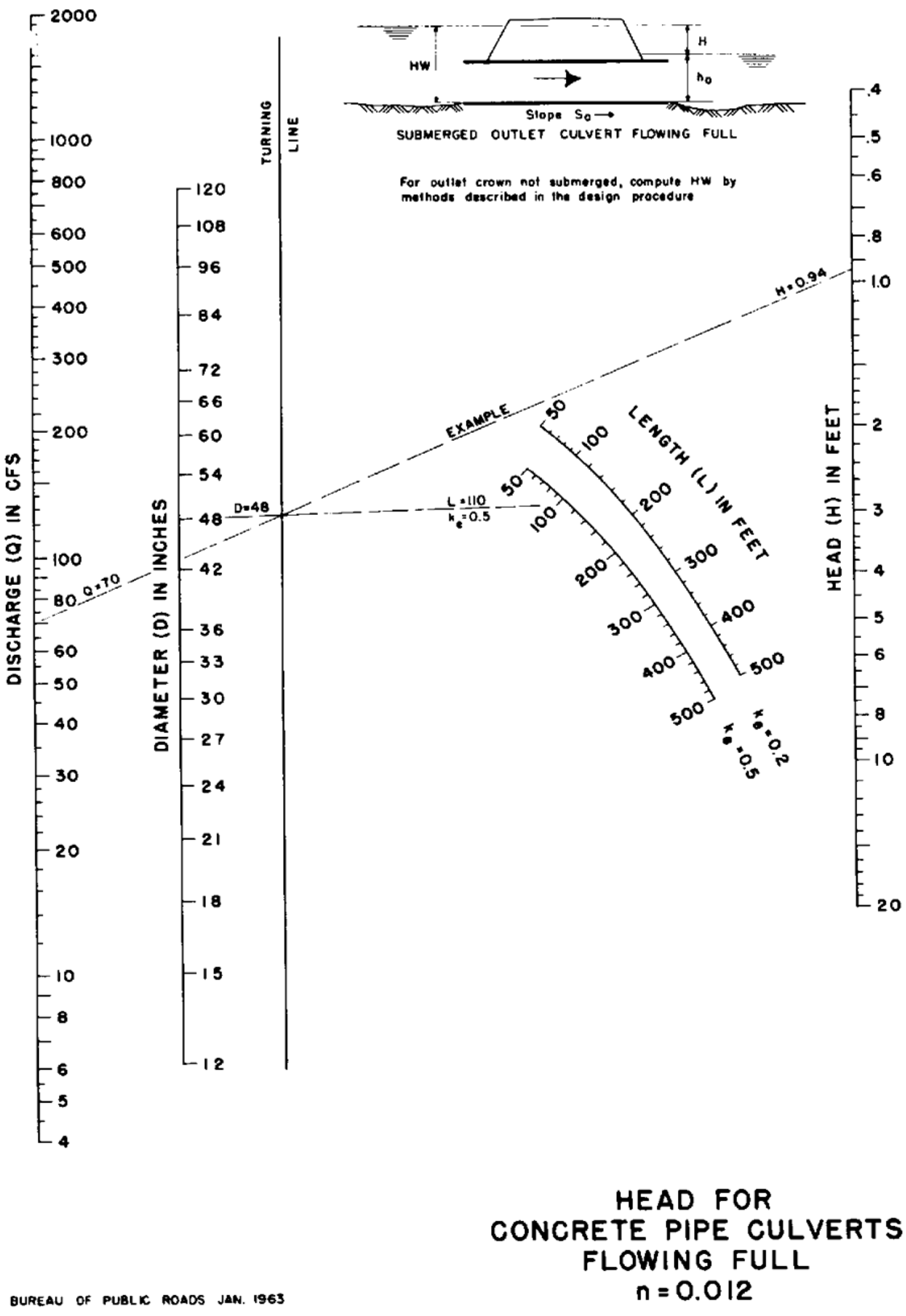


Figure 4-13: Head for Concrete Pipe Culverts Flowing Full (n= 0.012) (Chart 5B)

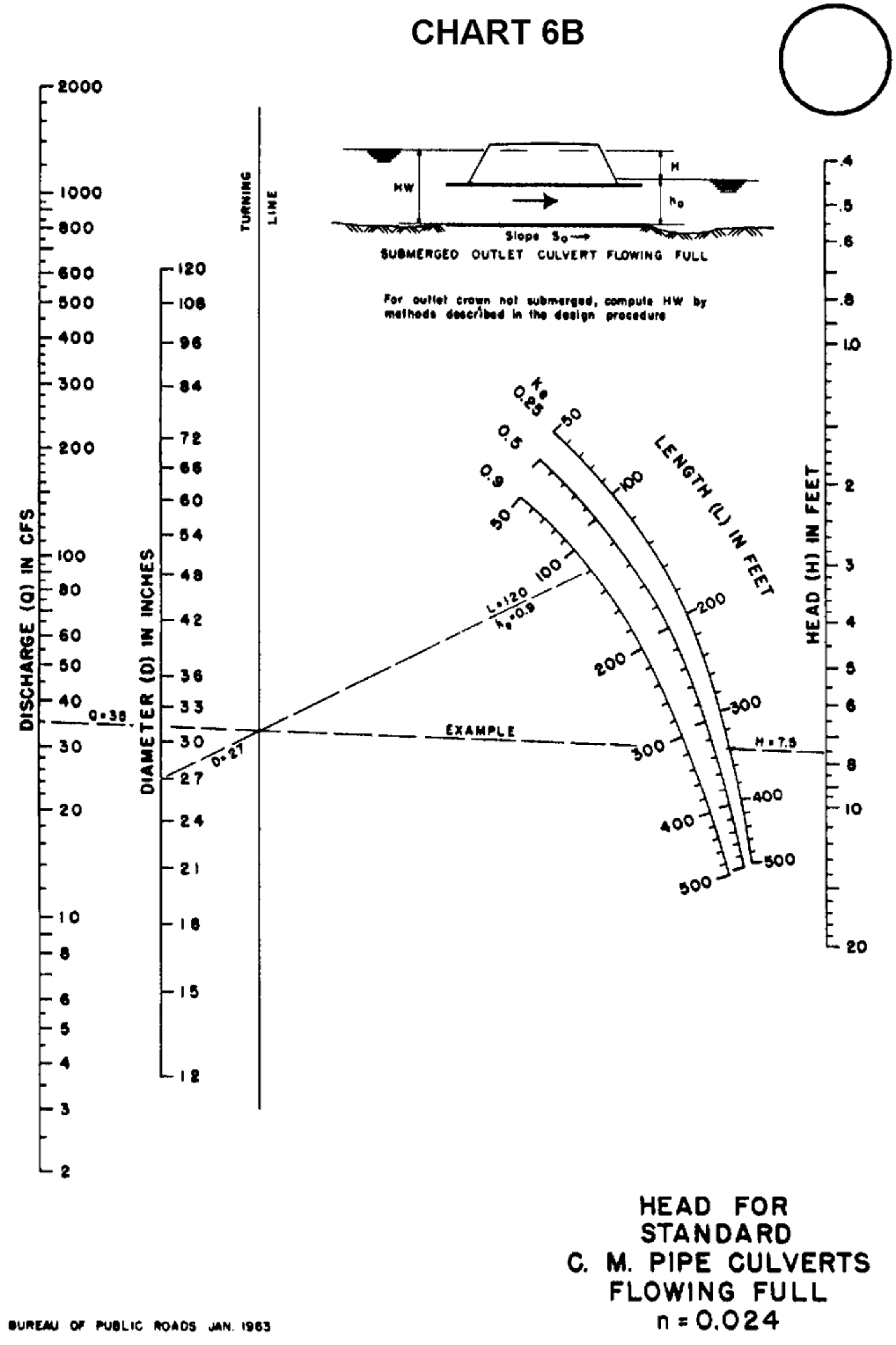


Figure 4-14: Head for Standard C. M. Pipe Culvert Flowing Full (n = 0.024) (Chart 6B)

CHART 7B

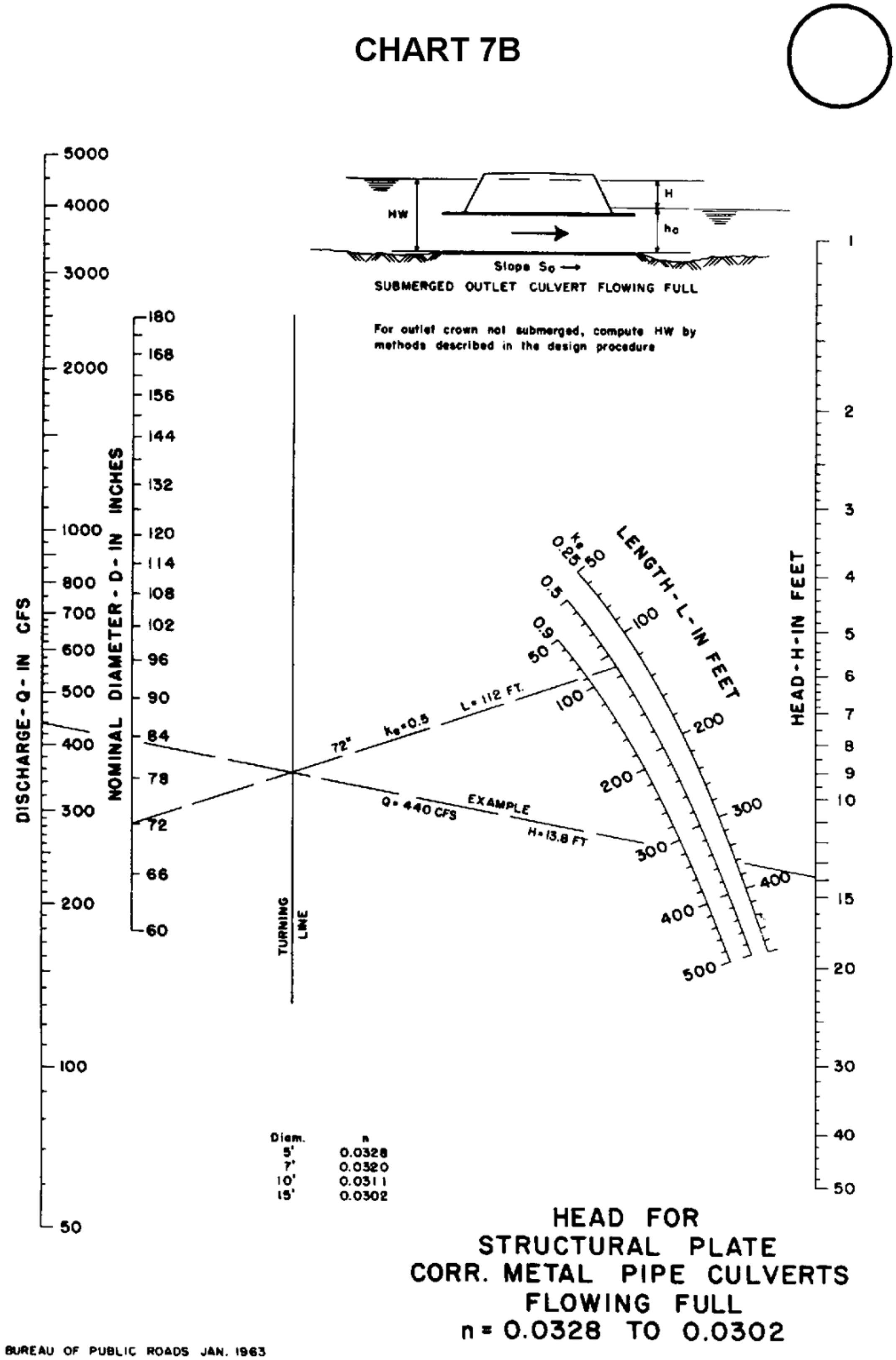


Figure 4-15: Head for Structural Plate Corrugated Metal Pipe Culverts Flowing Full (n = 0.0328 to 0.0302) (Chart 7B)

CHART 8B

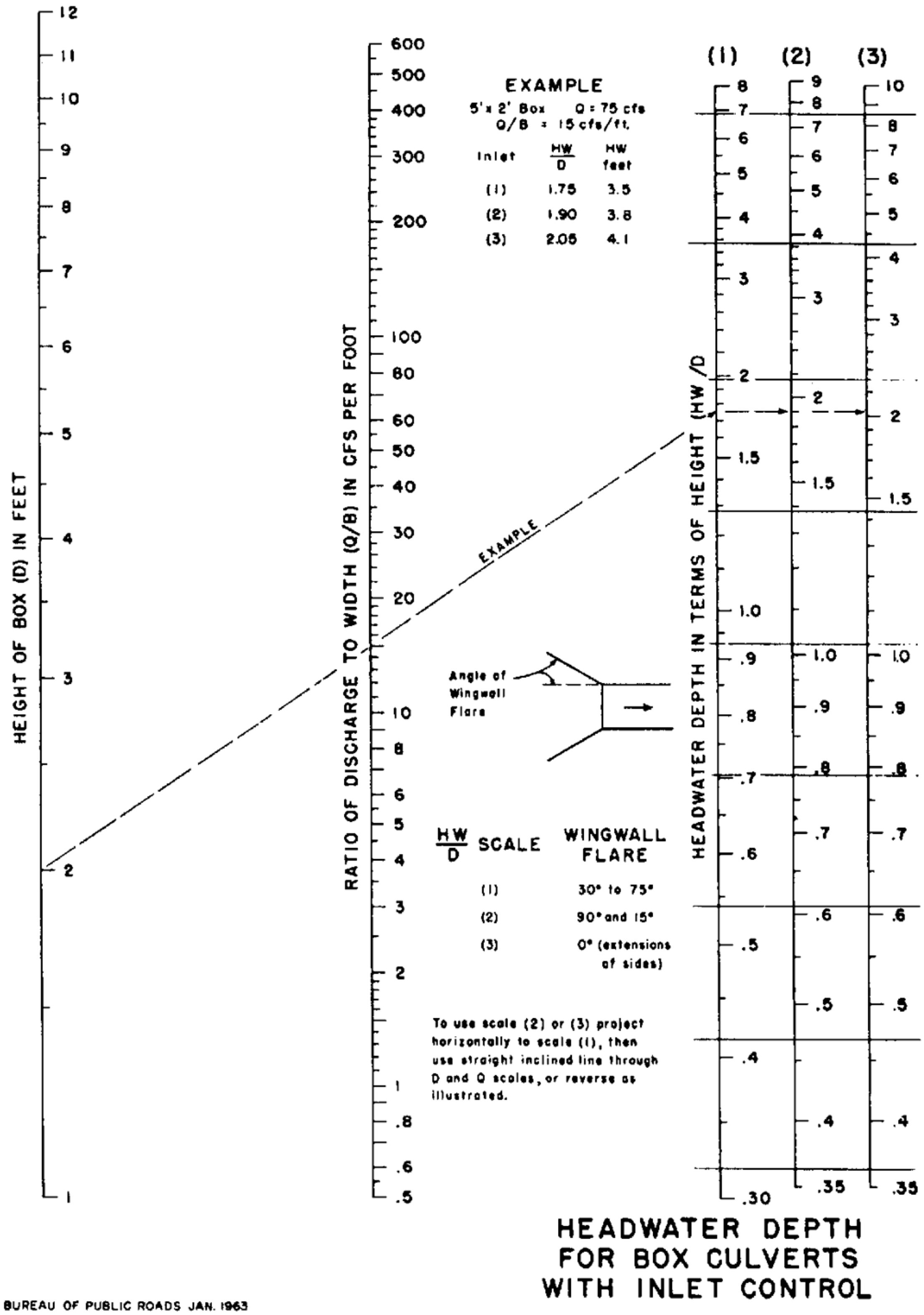


Figure 4-16: Headwater Depth for Box Culverts with Inlet Control (Chart 8B)

CHART 9B

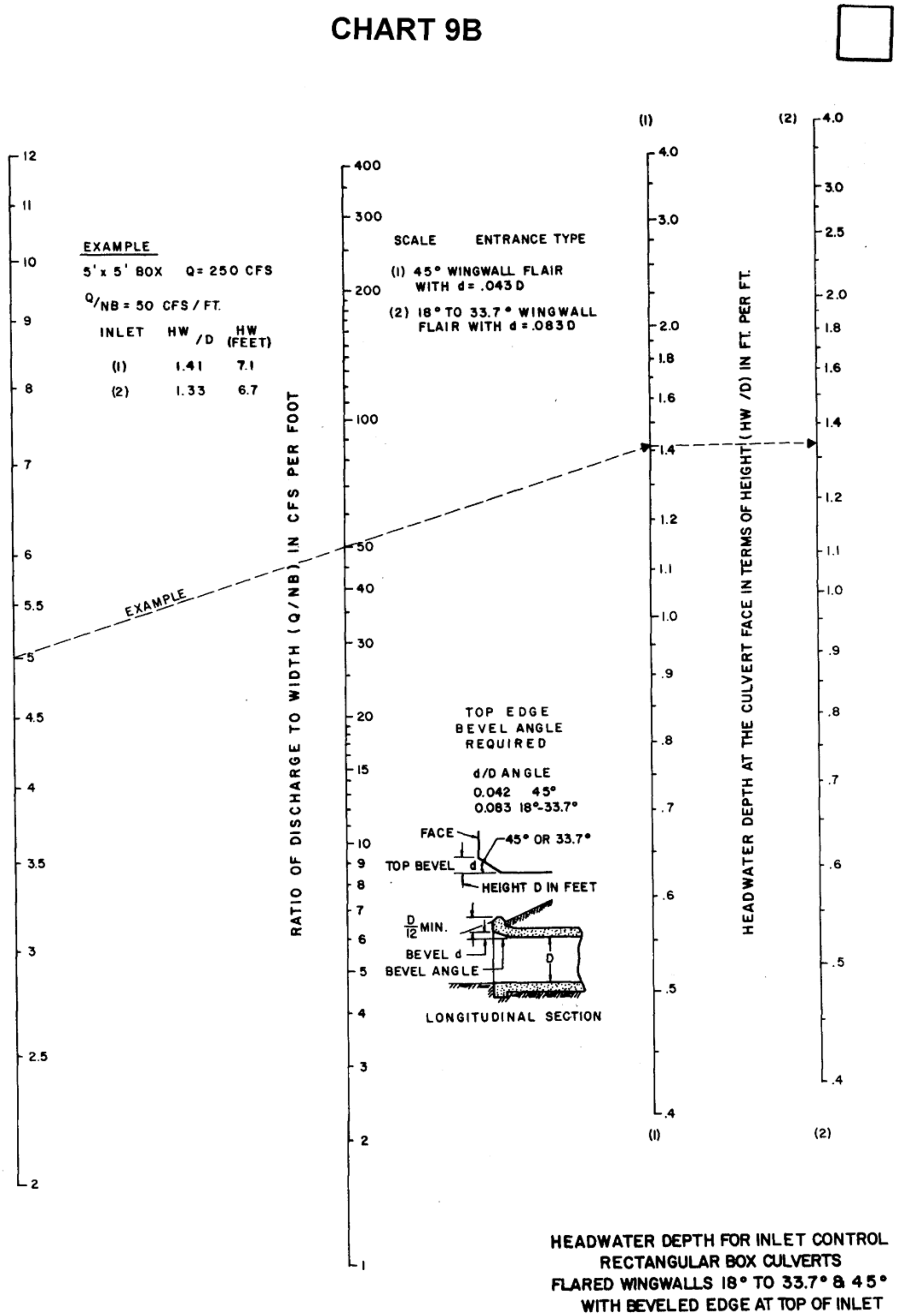


Figure 4-17: Headwater Depth for Inlet Control, Box Culverts, Flared Wingwalls 18° to 33.7°, and 45° with Beveled Edge at Top of Inlet (Chart 9B)

CHART 10B

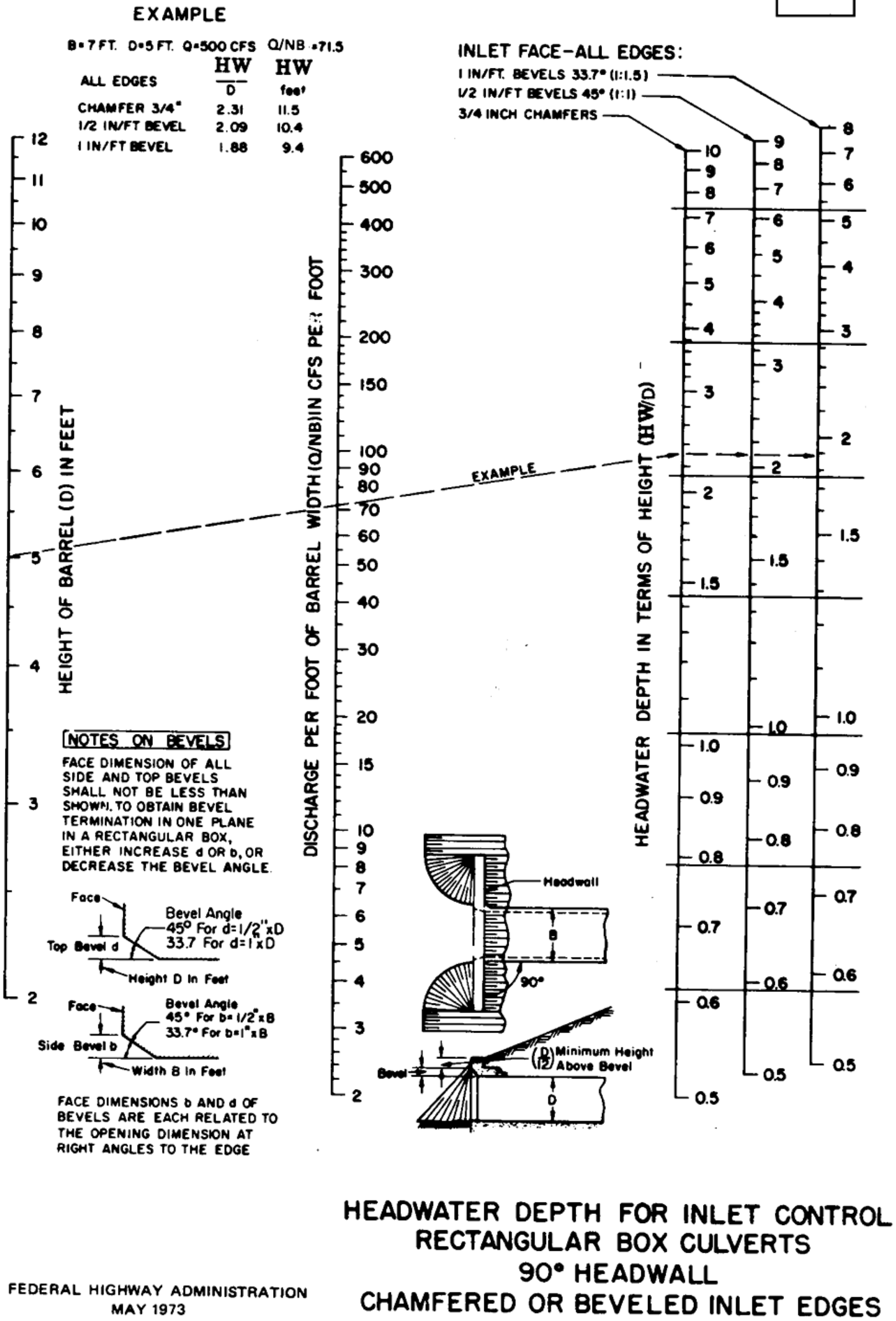


Figure 4-18: Headwater Depth for Inlet Control, Rectangular Box Culverts, 90° Headwall, Chamfered, or Beveled Inlet Edges (Chart 10B)

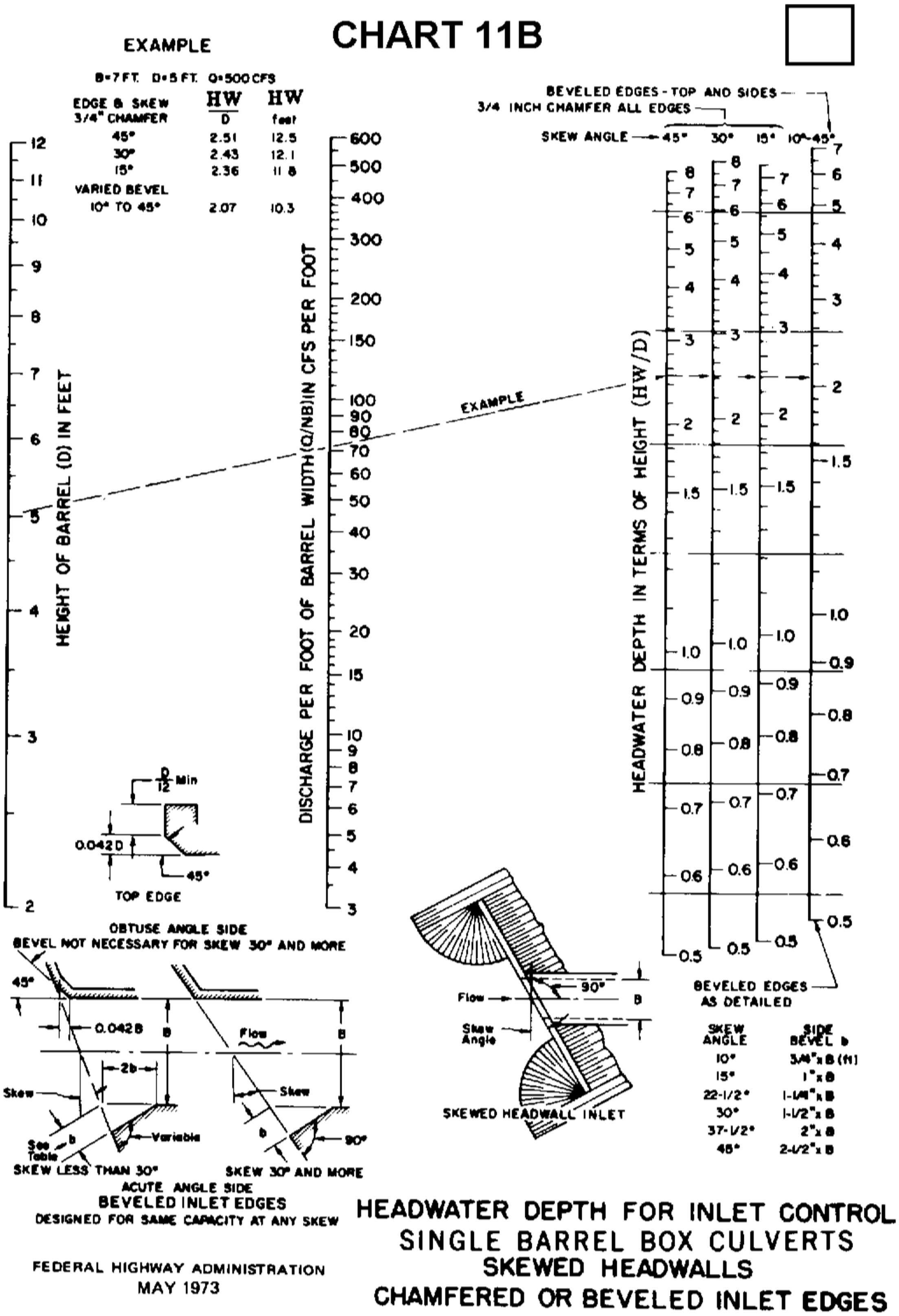


Figure 4-19: Headwater Depth for Inlet Control, Single Barrel Box Culverts, Skewed Headwalls, Chamfered, or Beveled Inlet Edges (Chart 11B)

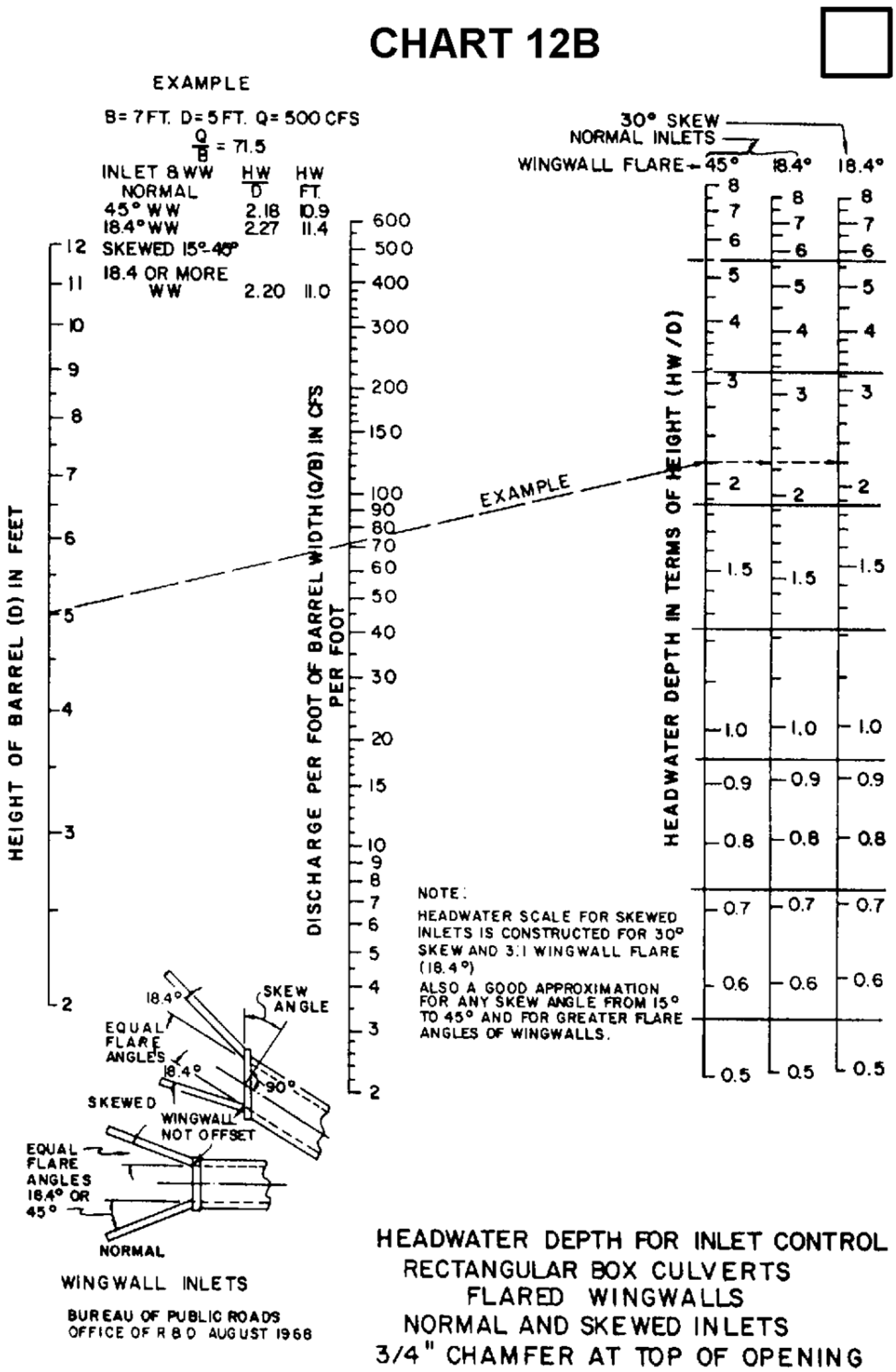


Figure 4-20: Headwater Depth for Inlet Control, Rectangular Box Culverts, Flared Wingwalls, Normal and Skewed Inlet Edges, ¾″ Chamfer at Top of Opening (Chart 12B)

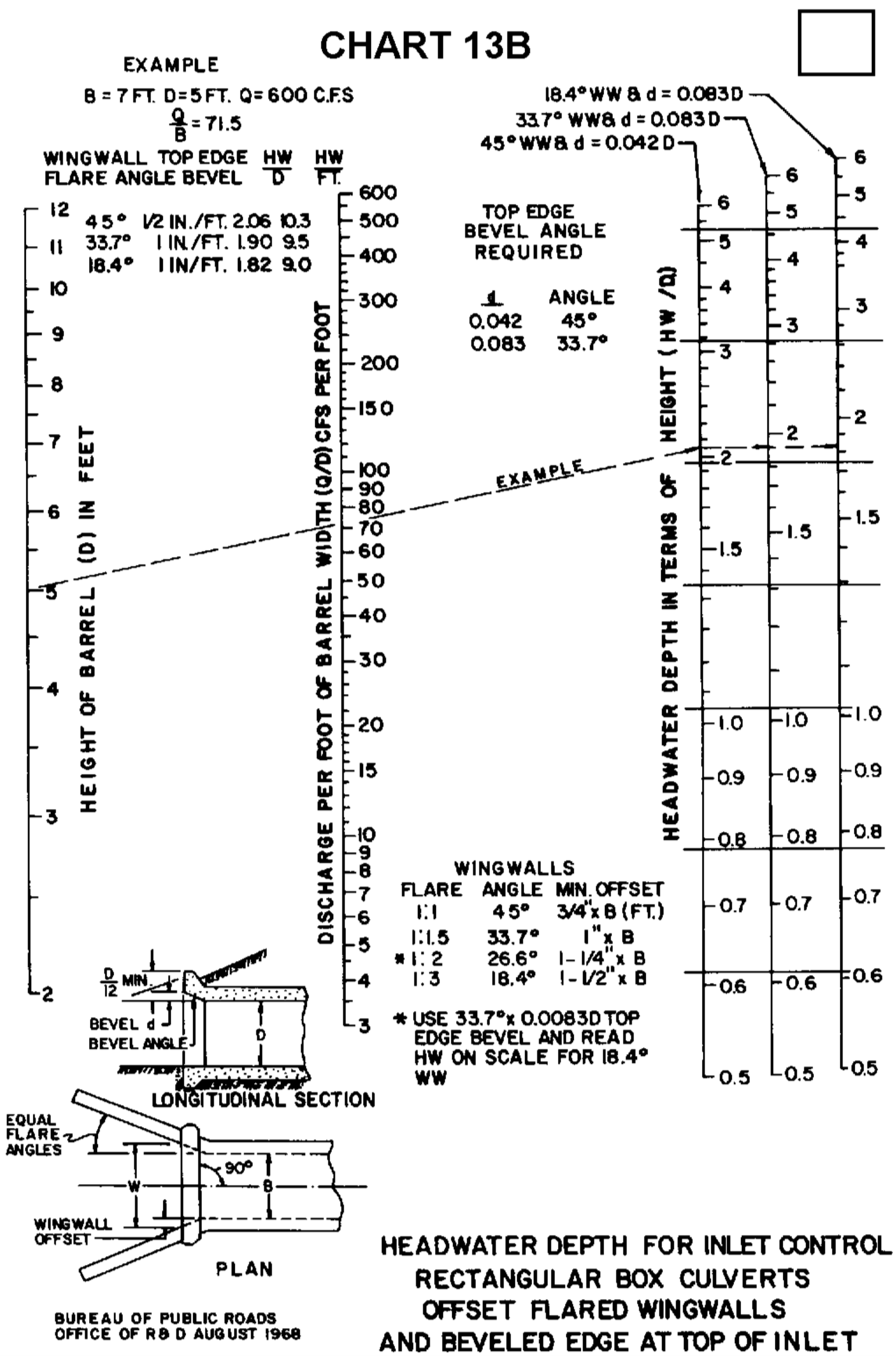


Figure 4-21: Headwater Depth for Inlet Control, Rectangular Box Culverts, Offset Flared Wingwalls, and Beveled Edge at Top of Inlet (Chart 13B)

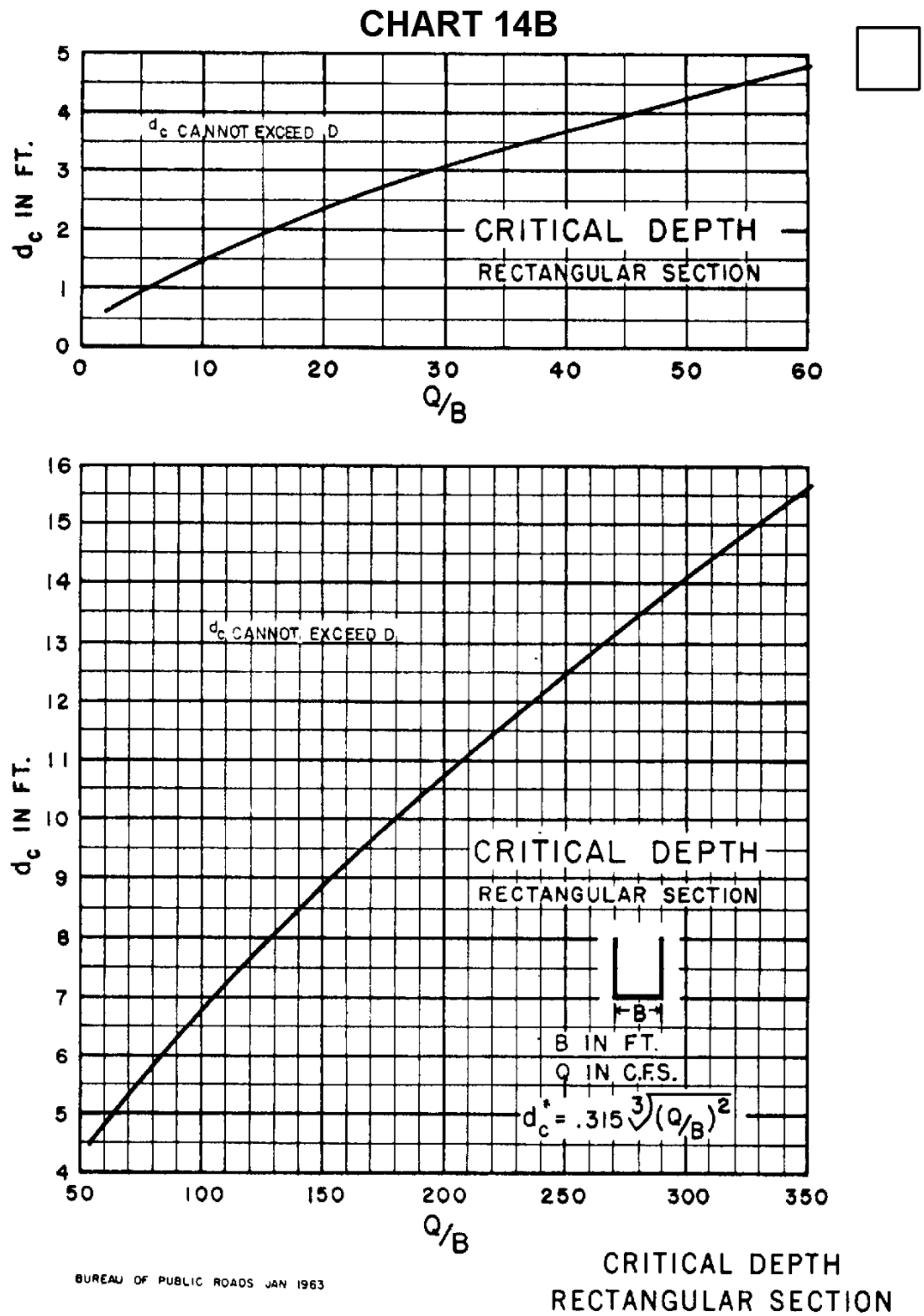


Figure 4-22: Critical Depth (Rectangular Section) (Chart 14B)

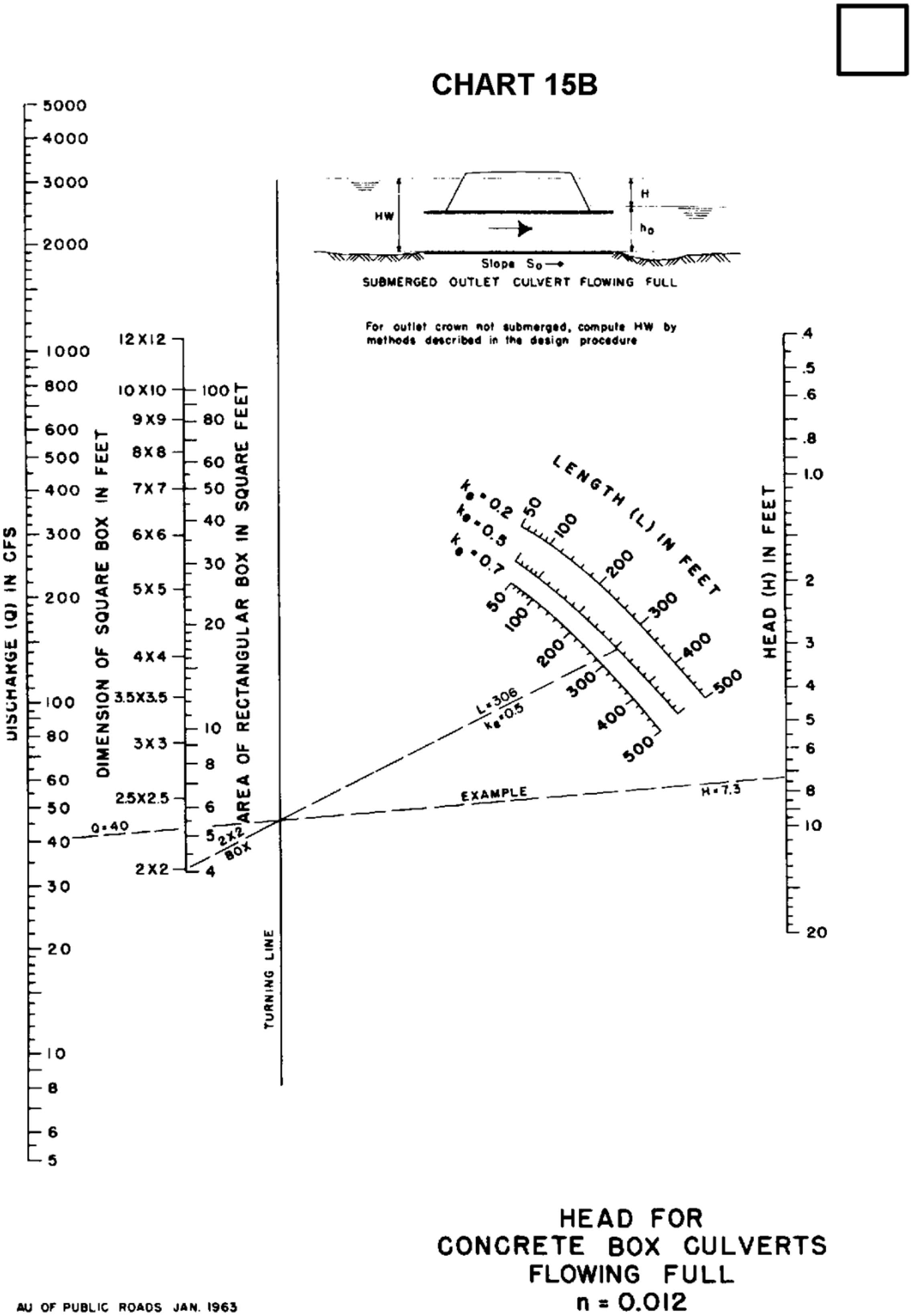
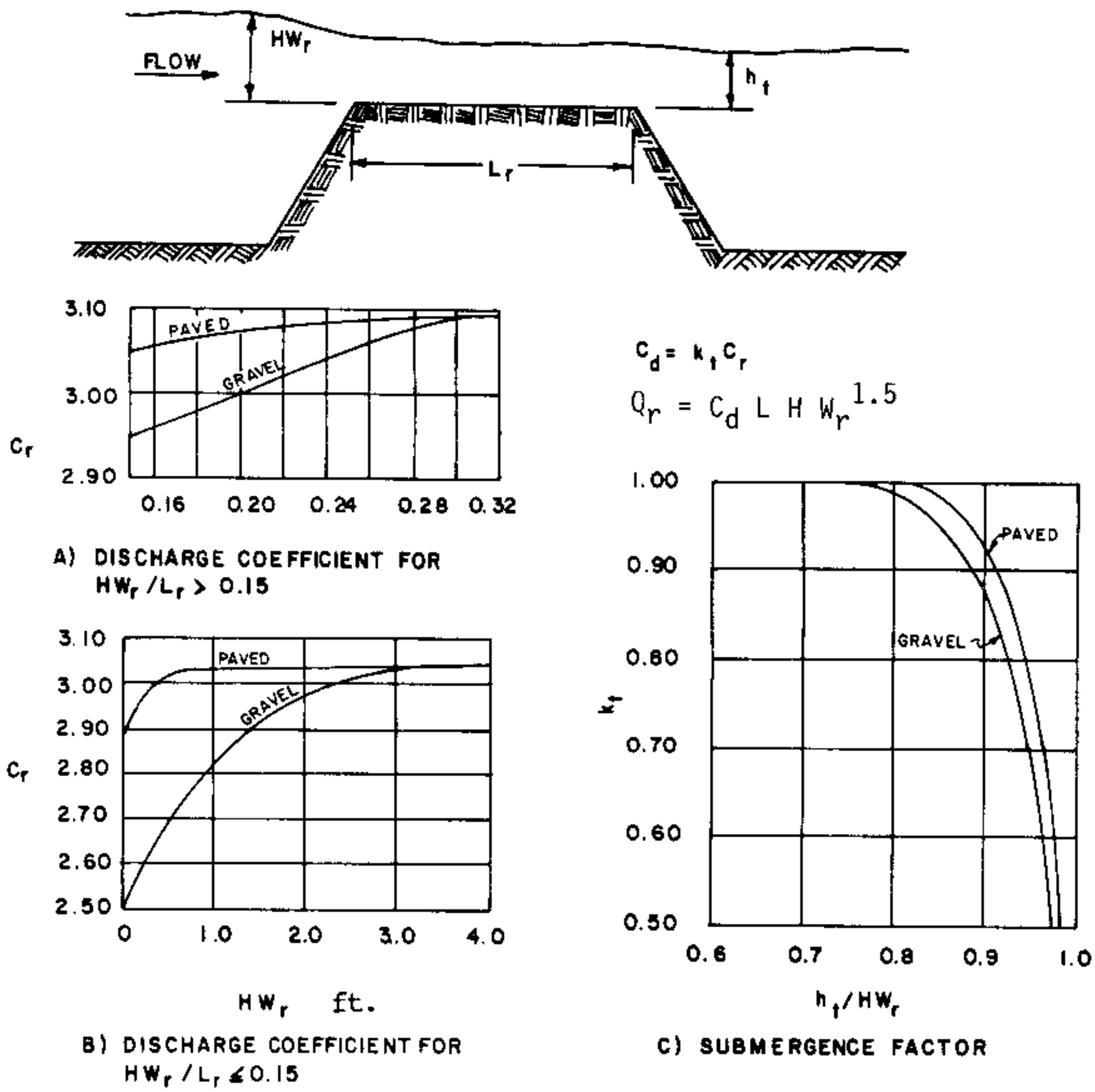


Figure 4-23: Head for Concrete Box Culverts Flowing Full (n = 0.012) (Chart 15B)

CHART 60B



English Discharge Coefficients
for Roadway Overtopping

Figure 4-24: Discharge Coefficients for Roadway Overtopping (Chart 60B)

PROJECT: ______	STATION: ______	**CULVERT DESIGN FORM**
	SHEET ______ OF ______	DESIGNER / DATE: ______ / ______
		REVIEWER / DATE: ______ / ______

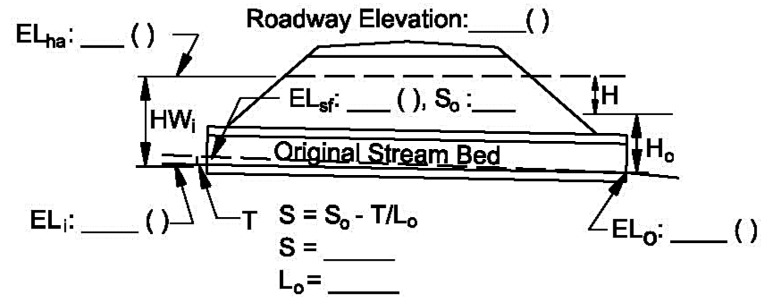
HYDROLOGICAL DATA

See Add'l Shts.

- ☐ METHOD: ______
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ______
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

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



CULVERT DESCRIPTION: MATERIAL – SHAPE – SIZE – ENTRANCE	Total Flow Q (cfs)	Flow Per Barrel Q / N (1)	HEADWATER CALCULATIONS: INLET CONTROL				OUTLET CONTROL							Control Headwater Elevation	Outlet Velocity	Comments
			HW/D (2)	HW_i	T (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)			

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

(3) T = HW_i – (EL_{hd} – EL_{sf})
T IS ZERO FOR CULVERTS ON GRADE

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

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

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

(7) $H = [1 + k_e + (K_u\, n^2\, L) / R^{1.33}]\, v^2 / 2g$ WHERE Ku = 19.63 (29 IN ENGLISH UNITS)

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

SUBSCRIPT DEFINITIONS:

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

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

Figure 4-25: Culvert Design Form

PROJECT: ______________	STATION: ______________	**TAPERED INLET DESIGN FORM**
	SHEET ______ OF ______	DESIGNER / DATE: ________ / ________
		REVIEWER / DATE: /

DESIGN DATA:

Q _____ = _______ ( ); EL_{hi} ____________ ( )

EL. THROAT INVERT _____________ ( )

EL. STREAM BED AT FACE __________ ( )

T ________ TAPER ________: 1 (4 : 1 TO 6 : 1)

STREAM SLOPE, S_0 = _________ ( ) / ( )

SLOPE OF BARREL, S = ________ ( ) / ( )

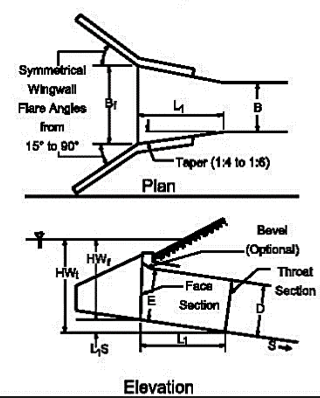
S_D ________ : 1 (2 : 1 TO 3 : 1)

BARREL SHAPE AND MATERIAL: ____________________

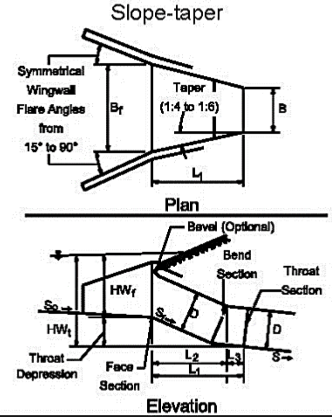
N = ____________, B = ______________, D = __________

INLET EDGE DESCRIPTION ______________________

Side-taper



Slope-taper



COMMENTS

									SLOPE-TAPERED ONLY						SIDE-TAPERED w/ depression		
Q ( )	EL_{hi} ( )	EL. Throat Invert	EL. Face Invert (1)	HW_f (2)	$\frac{HW_f}{E}$ (3)	$\frac{Q}{B_f}$ (4)	MIN. B_f (5)	Selected B_f	MIN. L_3 (6)	L_2 (7)	Check L_2 (8)	Adj. L_3 (9)	Adj. Taper (10)	L_1 (11)	EL. Crest Inv.	HW_c (12)	MIN. W (13)

(1) SIDE-TAPERED : EL. FACE INVERT = EL. THROAT INVERT + 1 FT (0.3 M APPROX.)
SLOPE-TAPERED : EL. FACE INVERT = EL. STREAM BED AT FACE

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

(3) 1.1 D ≥ E ≥ D; E = D FOR BOX CULVERTS

(4) FROM DESIGN CHARTS

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

(6) MIN. $L_3 = 0.5$ NB

(7) L_2 = (EL. FACE INVERT- EL. THROAT INVERT) S_D

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

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

(10) If (7) >(8), ADJ. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(11) SIDE-TAPERED : $L = \left[\frac{B_f - NB}{2}\right] \cdot TAPER$

SLOPE-TAPERED : $L_1 = L_2 + L_3$

(12) $HW_C = EL_{hi}$ - EL. CREST INVERT

(13) MIN. $W = K_u\, Q / HW_c^{1.5}$ Where K_u = 0.35 (0.64 SI)

SELECTED DESIGN

B_f ______________

L_1 ______________

L_2 ______________

L_3 ______________

BEVELS ANGLE ___________

b = ______( );d = __________( )

TAPER __________ : 1

S_D = ____________ : 1

Figure 4-26: Side/Slope Tapered Design Form

Date: _____________ **Rater(s):** _______________________________________ **SHA/Consultant Firm:** ________________

Location: ____ **County:**__________________ **District:** ________________

Upstream Structure Number: _________________________ **Pipe Length/Size/Material:** ___________________________

Downstream Structure Number: __________________________ **Year Pipe Constructed:** _____________________________

Instructions: Check all observations that apply. The most critical defect observed along the individual pipe length decides the overall condition rating. For example, two checked boxes associated with fair condition and one checked box associated with poor condition would yield an overall poor condition rating.

0 No Rating:

☐ Pipe not accessible (include reasoning in comments section)
☐ Pipe submerged
☐ Pipe buried
☐ Overgrown vegetation
☐ Pipe could not be located
☐ Pipe full of sediment/debris

☐ Other ____

1 Excellent Condition (pipe has nearly full material service life remaining):

☐ No structural defects/in "like new" condition

☐ Other ____

2 Good Condition (minimum collapse risk in short term - pipe unlikely to fail for at least 20 years):

☐ Minor rust with no perforations; slight pitting
☐ Deflection <5% in flexible pipe
☐ Minor pipe misalignment due to settlement or contractor grade control
☐ Minor joint defects; minor joint separation (no infiltration of soil or water), joints deteriorated at isolated locations
☐ Minor hairline cracks or minor cracking at bolt holes
☐ Small areas of wearing, spalling or scaling (finished concrete surface chunks cracking off) along pipe invert
☐ Small areas of corrosion/rust; however, no perforations observed

☐ Other ____

3 Fair Condition (collapse unlikely in near future but further deterioration likely - pipe may fail in 10-20 years):

☐ Longitudinal cracking (≥0.1" for concrete pipes), multiple cracks or significant seam cracking near bolts on metal pipes
☐ Deflection 5-7.5% in flexible pipe
☐ Minor horizontal or vertical displacement of pipe segments (less than one pipe wall thickness)
☐ Minor to moderate perforations in pipe, scattered heavy rust and/or deep pitting
☐ Moderate joint defects; moderate joint separation (less than one pipe wall thickness) allowing soil or water infiltration
☐ Minor undermining of pipe/structure; possible piping
☐ Endwall/headwall/end section separated from pipe segments with no embankment failure
☐ Moderate cracks or spalling and/or large areas of scaling (peeling or flaking less than 0.25 inches deep); isolated locations of exposed reinforcing steel
☐ Moderate areas of wearing less than 0.5 inch deep
☐ Moderate areas of corrosion/rust with isolated perforations

☐ Other ____

Figure 4-27: Pipe Condition Rating Form

<u>4 Marginal Condition (collapse likely in foreseeable future - pipe may fail within 2-10 years)</u>:

☐ Evidence of roadway settlement or previous patching; depressions in roadway
☐ Deflection 7.5-10% in flexible pipe
☐ Pipe broken or has multiple fractures; seams cracked 3 inches or more at bolts; evidence of efflorescence (crystalline whitish deposit on surface of concrete)
☐ Misalignment and/or ponding from sagging segments shifted more than one pipe wall thickness (significant infiltration/exfiltration at joints; fill material visible)
☐ Major undermining of pipe/structure; evidence of piping (runoff along outside of pipe eroding soil around and beneath)
☐ Pipe exposed behind endwall/headwall structure
☐ Partial or complete collapse of endwall/headwall structure with embankment failure not encroaching on the roadway
☐ Embankment failure near pipe NOT encroaching on the roadway
☐ Extensive areas of spalling (chipping/splintering concrete) or slabbing (large slabs of concrete peeling) with exposed corroding reinforcing steel
☐ Extensive areas of corrosion/rust with scattered perforations and deep pitting
☐ Large areas of wearing greater than 0.5 inch deep (removal and deformation of surface material in pipe)

☐ Other ____

<u>5 Poor Condition (collapsed or collapse imminent - pipe has failed or will fail within 2 years)</u>:

☐ Roadway severely sagging/caving due to settlement; shoulder or road closed to traffic
☐ Partial or complete collapse of pipe
☐ Deflection >10% in flexible pipe, or >7.5% with signs of stress cracking
☐ Pipe invert section loss
☐ Extensive areas of pipe material missing and/or extensive perforations in pipe
☐ Partial or complete collapse of endwall/headwall structure with embankment failure encroaching on the roadway
☐ Embankment failure near pipe encroaching on the roadway
☐ Multiple sections are out of alignment; pipe is not functioning

☐ Other ____

Overall Rating Number: ___________

Comments (include defect measurements): ____

____

____

Recommended Inspection Frequency:

☐ Regular inspection cycle (may exceed 10 years prior to reinspection)

☐ Annual

☐ After severe rainfall event

☐ Other: ____

Previous Inspection:

- Condition Rating: ____________
- Primary defect(s) identified: ____
- Date of previous inspection: ____________
- Inspector's Name/Organization: ____

Figure 4-27: Pipe Condition Rating Form (continued)

5
STORM DRAIN SYSTEMS

5.1 OVERVIEW

5.1.1 Introduction

This chapter provides design procedures and guidance on all elements of storm drainage design: system planning, hydrology, pavement drainage, gutter flow calculations, inlet and manhole spacing, pipe sizing and hydraulic grade line calculations, end treatments and outfall protection, maintenance, and control of runoff from future development.

5.1.2 Consequences of Inadequate Drainage

- Unsafe traffic conditions due to water and ice on the roadway.
- Delays to traffic and damage to vehicles caused by excessive ponding in sags or excessive spread along the roadway.
- Damage to adjacent property resulting from water flowing and splashing over the roadway curb and entering such property.
- Weakening of roadway base and subgrade due to saturation from frequent ponding.

5.1.3 General Design Guidelines

Storm drain systems are the inlets, manholes, pipes, and outlet structures that receive and convey runoff from storm events to where it is discharged into a channel or body of water or piped system. Storm drains should be designed with consideration for future development, if appropriate. Where feasible, the storm drains should be designed to avoid existing utilities. Attention should be given to the storm drain outfalls to ensure that the potential for erosion is minimized and that the outfall will be stable for the design storm event. Drainage system design should be coordinated with the proposed staging of large construction projects to maintain positive drainage throughout the construction project.

5.1.4 Definitions

The following definitions are important in storm drainage analysis and design. These definitions will be used throughout this chapter to address different aspects of storm drainage analysis:

Bypass. Carryover flow that bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade.

Combination Inlet. A drainage inlet usually composed of a curb opening and a grate opening.

Crown. The inside top of the pipe, sometimes known as the soffit.

Curb Opening. An opening in the curb to provide drainage relief.

Drop Inlet. A box that is sized to match the storm drainage pipe and provides a base for the grate frame.

Equivalent Cross Slope. A computed straight cross slope having conveyance capacity equal to that of the given compound cross slope.

Flanking Inlets. Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. These inlets intercept debris as the slope decreases and act in relief of the inlet at the low point.

Frontal Flow. The portion of the flow in the section of gutter occupied by the grate.

Side-Flow Interception. Flow that is intercepted along the side of a grate inlet, as opposed to frontal interception.

Grate Inlet. A drainage inlet composed of a grate in the roadway section or in the roadside swale or channel.

Grate Perimeter. The sum of the lengths of all sides of a grate, except that any side adjacent to a curb or barrier wall is not considered a part of the perimeter in weir-flow computations.

Gutter. That portion of the roadway section adjacent to the curb used to convey stormwater runoff. A composite gutter section consists of the section immediately adjacent to the curb, which has a cross slope steeper than the adjacent pavement, and the parking lane, shoulder or pavement at a cross slope of a lesser amount. A uniform gutter section has one constant cross slope.

Hydraulic Grade Line. The locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head). It is also equal to the energy grade line minus the velocity head.

Inlet Efficiency. The ratio of flow intercepted by an inlet to total flow in the gutter.

Invert. The inside bottom of the pipe.

Lateral Line. A lateral line, sometimes referred to as a lead, has inlets connected to it but has no other storm drains connected. It is usually 24 in. or less in diameter and is a tributary to the trunk line.

Positive Drainage. Continuous downhill grades in which water can flow without ponding.

Pressure Head. Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Sag Point/Major Sag Point. A low point in a vertical curve. A major sag point refers to a low point that can overflow only if water can pond to a depth of 2 ft or more.

Slotted Drain and Trench Drain Inlets. Slotted drain is composed of a continuous slot built into the top of a pipe that serves to intercept, collect and transport the flow. Trench drains are usually comprised of removable narrow grates built on a tray or preformed trench.

Storm Drain. A storm drain is a closed or open conduit that conveys stormwater that has been collected by inlets to an adequate outfall. It generally consists of laterals or leads and trunk lines or mains. Culverts connected to the storm drain system are considered part of the system.

Splash-Over. That portion of frontal flow at a grate that skips or splashes over the grate and is not intercepted.

Spread. The width of stormwater flow measured laterally from the roadway curb.

Sump Inlet. Any inlet placed at a sag point.

Trunk Line. A trunk line is the main storm drain line. Lateral lines may be connected at inlet structures, access Holes, or with pipe tees. A trunk line is sometimes referred to as a "main."

Velocity Head. A quantity proportional to the kinetic energy of flowing water expressed as a height or head of water ($V^2/2g$).

5.2 GENERAL CONSIDERATIONS

The placement and hydraulic design of storm drainage facilities should consider the potential for damage to adjacent property, the risk of traffic interruption due to flooding, the design speed and traffic service requirements, utility impacts, access, and the availability of funding.

5.2.1 Bridge Decks

Many short bridges may not require any drainage facilities at all. Longer and wider bridge decks may require drainage facilities to provide adequate traffic passage for the desired level of service. Bridge designs with sags, flat, or zero gradients should be avoided, otherwise an extensive drainage system will be required. Any drainage system interacting with a bridge deck should be coordinated with MDOT SHA Office of Structures.

Water flowing downgrade in closed approach roadway sections should be intercepted so as to not run onto the bridge. Longitudinal runs of piping should not be used, and no drainage system shall be placed in any substructure unit or attached to any substructure unit unless such a design is approved by the Office of Structures. The spread is to be limited to that shown in the Office of Structures' Manual for Hydrologic and Hydraulic Design *(MDOT SHA OOS, 2020)*.

5.2.2 Hydrology

Design storm drain systems using the Rational Method. Accurately model the time of concentration to each inlet, using a minimum of five minutes. Drainage systems involving detention storage, pumping stations, and large or complex storm systems require the development of a runoff hydrograph. The Rational Method and hydrograph methods are described in Chapter 2 “Hydrology”.

5.2.3 Storm Drain System Layout

Storm drain pipes should not decrease in size as they progress downstream regardless of the available pipe gradient. Connection to an existing smaller pipe may be made at the project extents with the approval of Highway Hydraulics Division.

Consistency in structures used in storm drain systems can improve constructability and reduce cost of construction for MDOT SHA projects. To this end, storm drain systems should be designed to use MDOT SHA precast round base structure details as much as possible. These structure types also can provide flexibility in design for accommodating pipe connections at non-90-degree angles. Designers must ensure that there is adequate room in the structure for all pipe connections, with a minimum wall “leg” of 6” of concrete between openings (accounting for 2” opening clearance from the outer diameter of the pipe). Designers should take care to minimize vertical depth payment for structures in excess of their minimum pay depth, except where necessary due to other constraints. Refer to the MDOT SHA Book of Standards for Highway and Incidental Structures for structure pay depths.

Locate the storm drain to avoid conflicts with foundations, subsurface and above-ground utilities, or other obstacles. Coordination with utility owners during the design phase is of high importance. Storm drain locations will affect construction activities and phasing, and complete or partially-constructed storm drains may be necessary for maintenance of flows during construction. Storm drain pipes can also conflict with existing and proposed traffic barrier installations and care should be taken to avoid damage to pipes due to placement of traffic barrier.

MDOT SHA storm drain systems primarily utilize a trunk line on one side of the roadway, rather than down the center of the roadway. Dual trunk lines along each side of the roadway may be used when it is difficult or more costly to install lateral crossings. Central trunk lines are discouraged, but may be used if it is determined to be the most appropriate alignment.

Many MDOT SHA construction projects are reconstructions of existing roads with existing storm drain systems. Designers should determine whether partial or whole reuse of the existing system is the best option. Factors to consider include the condition / remaining lifespan and capacity of the existing system, construction costs for replacement, maintenance and access of the system, and utility clearances.

5.2.4 Inlets

The term “inlets” refers to all types of inlets (e.g., grate inlets, curb inlets, combination inlets, slotted inlets, etc.). Drainage inlets are sized and located to limit the spread of water on traffic lanes to tolerable widths for the design storm in accordance with the design criteria specified in Section 5.3.2.

On closed-section roadways (i.e. those with curbing or barrier), use curb opening inlets unless there is a reason that curb opening inlets are not appropriate. Use of alternative inlet types must be approved by HHD. Inlet depression / gutter pan depression will be provided at all curb opening inlets unless specifically directed otherwise in the contract documents.

Grate inlets, when they are used, should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. Grate inlets should not be used on bicycle-forward facilities such as cycle tracks or bike lanes and all grate inlets shall be bicycle safe where used on roadways that allow bicycle travel. Curb inlets are greatly preferred to grate inlets at major sag locations because of their debris handling capabilities. When grate inlets must be used at sag locations, assume that they are half plugged with debris and size accordingly.

In locations where ponding may occur on the travel lanes due to clogging of a sump inlet, place at least one flanking inlet away from the sump at approximately 0.2 vertical feet above the low point in the sag. These inlets provide relief for the sump location in the event of clogging or excessive runoff.

Inlets should be located upgrade of all public road connections, commercial entrances, and bridges to avoid potential icing situations. These inlets should be integrated into the system design, rather than placed as additional inlets. Provide inlets in superelevation transitions approximately 50 to 100 feet upgrade of the section where the cross slope is 0% (level section) to prevent flow across the roadway at the transition. See Section 5.3.3.

5.2.5 Storm Drain Conduit

A. Pipe Materials

See Section 4.9 Pipe Materials.

B. Minimum Longitudinal Slope

Design storm drains such that full flow velocities are not less than 3 fps. For very flat grades where this velocity cannot be met, the suggested practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drain system should be checked to be sure there is sufficient velocity in all of the drains to deter sedimentation. Pipe velocities must be provided in storm drain computations. A slope of 0.5% is considered the minimum slope for constructability. Pipe slopes flatter than 0.5% must be approved by the Highway Hydraulics Division.

C. Maximum Longitudinal Slope

Slopes that incur uniform flow velocities in excess of 10 fps should be avoided because of the potential for abrasion. When using pipe with smooth exteriors such as RCP and CPP type D, slopes in excess of 20% are not preferred because of the need for anchor blocks. When anchor blocks are used, they should be installed at every other pipe joint, as a minimum.

Corrugated pipe such as CPP Type S, CPDP, PPWP or CMP is preferred on steep slopes. Corrugated metal pipe should not be used in areas where the flow is expected to carry an abrasive bed load or that have pH and resistivity factors beyond the ranges specified in Section 4.9.3 of this manual. In steeper terrain, large elevation differences can be accommodated using drop structures, otherwise known as “step down” manholes, to reduce the pipe gradient.

Where “step down” manholes are used, the designer should provide any needed protection to prevent deterioration of the bottom of the manhole. Provide drip stone landings in accordance with MDOT SHA standards. This is typically when the vertical difference between the inverts of the inlet pipe and outlet pipe is 6 ft or greater, and any one of the following factors are present or anticipated:

- The flow is expected to carry any abrasive material,
- Continuous live flow or live flow lasting several days may be expected, or
- The size of the main pipes are 48” or greater in diameter (for circular pipe) or the hydraulic opening is 12 sf or greater (for shapes other than circular)

D. Invert Drop Across Structures

A drop in pipe inverts across structures allows for robustness in construction, preventing ponding of water and sedimentation within structures, and can help to offset energy losses in the structure. The standard design drop from the pipe invert into a structure to the pipe invert out from a structure is 0.2 feet for equal sized pipes.

Where pipe sizes increase across a structure (pipe out is larger than pipe in), the inside crown of the pipes should be set to the same elevation, resulting in an invert drop of the difference in pipe size.

E. Alignment Changes

In most cases, changes to horizontal and vertical alignment are made using standard structures such as manholes and inlets with access points. In those locations where access is not practicable, buried junction boxes may be used with the approval of the Highway Hydraulics Division. Minimum half-benching is required for any structure with two or more pipe connections, see Section 5.7.

In situations where standard structures are not feasible, or where there is a significant bend loss, a pipe on radius solution may be implemented.

Pipe may be laid on a radius when necessary to conform to design features, alignment, or topography and to eliminate or minimize the need for manholes or other structures. Pipe laid on a radius is to be concrete only. Installation of concrete pipe on a radius may be done using beveled pipe with one side shorter than the other. Bevel pipe is expensive to manufacture and somewhat difficult to install. It is generally more economical to use prefabricated elbows, with a design exception, in cases where three or more joints of bevel pipe would be required. The minimum radius obtainable is dependent upon two factors that differ between manufacturers:

- Spigot or tongue length
- Pipe joint length

Radius pipe is only to be used where it is impossible or extremely impractical to place structures and requires a design exception with the approval of the Highway Hydraulics Division Chief.

Field connection elbows are not allowed.

F. Extension of Existing Pipes

Situations will be encountered, such as during roadway widening, where existing pipes must be extended. Designers should specify the type of connection to the existing pipe in the contract documents. The following methods can be used when an existing pipe must be extended:

- Remove a portion of existing pipe to provide a clean joint surface. Extension pipe to be the same type as existing, with a compatible joint type. This method is recommended where the extended pipe does not change alignment.
- Construct a manhole or junction box to connect existing pipe to extension.
- Design a collar to envelop the ends of the existing pipe and extension.
- Depending on condition of existing pipe, consider replacing entire length

5.2.6 Culverts

Any part of a storm drain system which crosses a roadway embankment or travel way and poses an overtopping risk shall be designed as a culvert.

5.2.7 Roadside Channels and Ditches

Large amounts of runoff should be intercepted before it reaches the highway in order to minimize the deposition of sediment or debris on the roadway and to reduce the amount of water which must be carried in the gutter section. Surface channels should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard. Where permitted by the design velocities, channels should have a vegetative lining. Appropriate linings may be necessary where vegetation will not control erosion, see Chapter 3 “Channels.” Right-of-way restrictions/costs in urban areas often render impracticable the provision of roadside ditches.

5.2.8 Median Drainage

Median runoff must be factored into the roadway drainage design. Runoff will be handled differently depending on the width of the median. Medians on wide open section roadways will typically be sloped to receive and convey runoff from adjacent lanes or shoulders, directing this flow to inlets within the median. Narrow raised medians will typically be sloped towards the roadway. Depending on the roadway geometry, runoff from these narrower raised medians will flow along the edge of the median or across the roadway as sheet flow. Often the former scenario will feature a “catch curb and gutter” (gutter sloping towards the curb) while the latter will feature a “spill curb and gutter” (gutter sloping away from the curb).

Raised medians wider than 6 feet should generally incorporate “catch curb and gutter” to retain longer duration runoff like snow melt and prevent it from flowing across the roadway travel lanes. This gutter flow should then be picked up by inlets at the low point / low end of the median. Gutter pans have limited capacity and should not be assumed to capture the entirety of median runoff during the design frequency storm.

5.2.9 Flood Hazard

In addition to ensuring proper roadway drainage, the designer must also evaluate the potential flood hazard to adjacent properties upstream and downstream of the storm drain system. Downstream impacts are typically assessed to meet stormwater management regulations for MDOT SHA projects. Storm drain systems should be also designed to consider downstream flood elevations when tailwater may affect the hydraulic design. The storm drain designer should verify there are no adverse impacts to upstream properties. Changes to roadway or driveway profiles or moving from open section to closed section will often require storm drain design considerations to ensure positive drainage from the higher property onto the lower property. Coordination is encouraged between MDOT SHA and adjacent property owners in these situations.

5.2.10 Hydroplaning

Hydroplaning conditions can develop for relatively low vehicular speeds and at low rainfall intensities for storms that frequently occur each year. Analysis methods have been developed through research that provide guidance in identifying potential hydroplaning conditions. Unfortunately, it is difficult to prevent water from exceeding a depth that would be identified through analysis procedures as a potential hydroplaning condition for wide pavements during high-intensity rainfall and under some relationship of the primary controlling factors of hydroplaning of:

- vehicular speed,
- tire conditions (pressure and tire tread),
- rainfall intensity,
- pavement micro and macrotexture,
- roadway geometrics (pavement width, cross slope, grade),
- pavement conditions (rutting, depressions, roughness), and
- effectiveness of storm drain system.

Designers do not have control of the first three factors. As such, assumptions need to be made, such as the concept that while speed is a significant factor in the occurrence of hydroplaning, most prudent drivers will reduce speed during high intensity rainfall. In many respects, hydroplaning conditions are analogous to ice or snow on the roadway.

Designers should strive to minimize hydroplaning potential, especially for lower intensity storms which do not affect speed. Remedial measures can be included in development of a project to reduce hydroplaning potential, see *Proposed Design Guidelines for Reducing Hydroplaning on New and Rehabilitated Pavements (NCHRP 243, 1999).*

If suitable measures cannot be implemented to address an area of high potential for hydroplaning, or an identified existing problem area, consideration should be given to installing advance warning signs.

5.2.11 Access

Access must be provided at regular intervals for inspection and cleanout. Access is typically provided through engineered access holes, also known as manholes. The terms access hole and manhole are synonymous and are used interchangeably. Access may also be provided using certain types of inlets in lieu of manholes.

Access holes located in traffic lanes should be avoided; however, where it is impossible to avoid locating an access hole in a traffic lane, care should be taken to ensure that it is not in the normal vehicular wheel path. Design Criteria for access holes are in Section 5.3.6.

5.2.12 Utilities

Utilities should be avoided when practical. Relocations are costly and add complexity to the project design process that may result in project delays. Coordinate all utility impacts with the District Utility Engineer and individual utility owners.

Refer to the MDOT-SHA Utility Manual (MDOT SHA OOC, 2021) for additional guidance.

5.2.13 Noise Barriers

For drainage strategies at noise barriers refer to FHWA Noise Barrier Design Handbook (FHWA, 2000). Note that the strategies which use a stone trench or gabions below the wall are discouraged due to the elevated risk of clogging over time.

5.3 DESIGN CRITERIA

Storm drain systems shall be analyzed using the Rational Method to generate inflow.

5.3.1 Design Frequency

Closed systems shall be sized initially so the full flow capacity is slightly greater than the computed 10-year frequency flow. The hydraulic gradient for the 25-year storm must remain below the finished grade and below the top of curb for the 100-year storm.

The following applies to storm drain systems:

- If a storm drain system provides conveyance for a sag inlet requiring a 50-year design storm for spread restrictions, as described in Table 5-1, then the hydraulic gradient for the 50-year storm must remain below finished grade for the portions of the storm drain system within the sag.
- Where a storm drain system includes a segment considered a culvert per Section 5.2.6, the storm drain system must be designed to ensure the appropriate design storm discharge in Table 4-1 from the culvert segment can be adequately conveyed within the entire downstream storm drain system.
- Where a dam or stormwater management pond discharges to a storm drain system, the storm drain system must be designed to ensure the design storm discharge from the dam or pond can be adequately conveyed within the entire downstream storm drain system.
- Sometimes an MDOT SHA project will connect to downstream drainage facilities and practices which do not provide adequate capacity to convey the design discharges of the MDOT SHA storm drain. In these cases, the designer must determine the practicability of connecting to these downstream systems. Factors to consider in these cases include impacts to the downstream system, the possibility of reducing the capacity of the MDOT SHA storm drain to match the downstream system, resiliency, and erosion potential of overland flood flows.

5.3.2 Design Spread

Design storm frequency values and spread restrictions are shown in Table 5-1. For storms of greater magnitude, the spread can be allowed to utilize "most" of the pavement as an open channel. For single-lane roadways and ramps, at least 8 ft of the travel lane should remain unflooded for design conditions. Shoulder areas used as part-time travel lanes should be considered as travel lanes.

Table 5-1: Criteria for Inlet Design

Condition	Design Speed	Design Storm	Additional Design Spread Restrictions
Interstate			
On-Grade	All	10-year	Shoulder Width
Sag Point	All	50-year	Shoulder Width
Principal Arterial			
On-Grade	≤ 45 mph	10-year	Maximum Encroachment to ½ Travel Lane
On-Grade	> 45 mph	10-year	Maximum Encroachment 3 feet into Travel Lane
Sag Point	All	50-year	Maximum Encroachment 3 feet into Travel Lane
Minor Arterial, Collector			
On-Grade	≤ 45 mph	10-year	Maximum Encroachment to ½ Travel Lane
On-Grade	> 45 mph	10-year	Maximum Encroachment 3 feet into Travel Lane
Sag Point	All	25-year	Maximum Encroachment to ½ Travel Lane
Local			
On-Grade	All	5-year	Maximum Encroachment to ½ Travel Lane
Sag Point	All	10-year	Maximum Encroachment to ½ Travel Lane

Notes:

1. Maximum design spread of 8 feet on all roadways.
2. Rainfall intensity and peak discharge are to be determined based on the design storm and methodologies outlined in Section 2.7. The minimum design rainfall intensity is 3 $^{in}/_{hr.}$
3. There shall be no encroachment into the travel lane on interstates.
4. Spread width includes the width of bike lanes. The designer may justify a smaller spread when the roadway cross section includes a bike lane and should consider the volume of bicyclists and the depth of flow within the bike lane.

5.3.3 Inlets

A. General

Inlets are drainage structures used to collect surface water through a grate, a curb opening, or a combination of both and convey it to storm drains or culverts.

Drainage inlets are sized and located to limit the spread of water on the roadway to allowable widths for the design storm as specified in Section 5.3.2. Grate inlets and the depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. Bicycle-safe inlet grates are required on bicycle-compatible roadways. In pedestrian areas, use ADA compliant grates.

B. Types of Inlets

Inlets used for the drainage of highway pavements can be divided into four major classes.

Curb Opening Inlets (COG and COS)
These inlets provide openings in the curb covered by a top slab. Curb-opening inlets are preferred at sag points because they can convey large quantities of water and debris. They may also be a viable alternative to grates in many locations where grates may be hazardous for pedestrians or bicyclists.

MDOT-SHA standard details have been developed for standard COG and COS inlets, Shallow COG and COS inlets, and COG and COS Opening inlets. MDOT-SHA standard details have been developed for modification of standard W-beam traffic barrier/guide rail where inlets are present.

Grate Inlets
These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Because they are susceptible to clogging with debris, the use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. Special-design (oversize) grate inlets can be used at major sag points if sufficient capacity is provided for clogging. Otherwise, flanking inlets are needed (see Section 5.2.4). Grate inlets within the roadway shall be safely traversable by bicycles. This requirement does not apply to facilities where bicycles are prohibited (i.e. expressways).

MDOT-SHA standard details have been developed for Reticuline Grates (WR, WRM, NR, MRM) and Curved-Vane Grates (CV-S and CV-E). Other grates may be used with approval from Highway Hydraulics Division.

Combination Inlets
Various types of combination inlets are in use. Curb-opening and grate combinations are common, some with the curb opening upstream of the grate and some with the curb opening adjacent to the grate. The gutter grade, cross slope, and proximity of the inlets to each other are significant factors when selecting this type of inlet. Combination inlets may be desirable in sags because they can provide additional capacity in the event of plugging.

Slotted Drain and Trench Drain Inlets
These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs because the flow usually enters perpendicular to the slot. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets.

Slotted drain or trench drain inlets located in the roadway may be allowed by design exception only. These inlets are prone to clogging and should be used only where larger inlets will not fit, or where it is necessary to intercept sheet flow or large volumes of water.

Curb Openings to Open Channels

MDOT SHA curb openings function in a similar manner to curb opening inlets, but they allow flow to open channels outside of the roadway rather than connecting to the closed drainage system. MDOT SHA prefers the use of open-back curb openings (COG / COS opening) which mimic the curb line presentation of curb opening inlets, including a top slab spanning the opening and inlet depression. These inlets are frequently used in areas where open channels are present for stormwater treatment, or to avoid utility conflicts. Appropriate stabilization must be provided for the design storm for flows exiting these inlets.

Alternatively, the use of a standard curb opening with no top slab or inlet depression should be limited to locations on interchange ramps where an inlet and pipe outfall are not feasible. Standard curb openings tend to collect debris and sediment, which may lead to premature pavement failure.

C. Inlet Locations

There are several locations where inlets may be necessary without regard to contributing drainage area. Examples of these necessary locations are:

- At all low points in the gutter grade.
- Upgrade of all public road connections and bridges where the 10-year storm peak flow equals or exceeds 1.0 cfs.
- On the upstream side of a median break, driveway entrance, curb-cut ramp, or pedestrian crosswalk where the 10-year storm peak flow equals or exceeds 1.0 cfs.
- Near the downstream terminus of curb. Place inlets to prevent the 10-year storm peak flow at the end of curb from exceeding 0.5 cfs.
- In superelevation transitions approximately fifty (50) to one hundred (100) feet upgrade of the level section (0% cross slope).

Additionally, inlets locations and types should adhere to the following conditions:

- Flanking inlets in sag vertical curves are standard practice. See Section 5.2.4.
- Inlets should not be placed in a depressed curb section, such as a driveway or sidewalk ramp, unless it is in a sag.
- Inlets grates should not be placed in the path where pedestrians walk. When an inlet must be placed within a crosswalk or pedestrian pathway, the inlet grate openings may not exceed ½” in width and be oriented perpendicular to the direction of foot traffic. See Standard ADA Compliant Inlets.
- Curb Opening Inlets are the preferred type of inlet within the roadway.
- Avoid placing Grate Inlets in travel lanes. If this is unavoidable, cast-iron grates should be used.
- Avoid placing Grate Inlets in turning or parking lanes where heavy truck traffic is anticipated.

- If Grate Inlets are used in travel or turning lanes, provide concrete aprons around the inlets.

D. Spacing Process

Locate inlets from the crest and work downgrade to the sag points. The design runoff can be computed as the maximum allowable flow in the curbed channel that will meet the design frequency and allowable water spread. The analysis for spacing shall be designed according to methodologies below or utilize FHWA's *Urban Drainage Design Manual (HEC-22) (FHWA, 2009)*, FHWA's Hydraulic Toolbox software or other software as approved by Highway Hydraulics Division.

To space successive downgrade inlets, it is necessary to compute the amount of flow that will be intercepted by the inlet (Qi) and subtract it from the total gutter flow to compute the bypass. The bypass from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the allowable water spread. Tabulate inlet spacing in a format matching Figure 5-15 or other formats approved by Highway Hydraulics Division.

Place and size inlets on grade to intercept a minimum 70% of flow for the design storm without exceeding the allowable spread.

Place and size inlets at sumps to intercept 100% of flow for the design storm without exceeding the allowable spread. Place flanking inlets as needed per Section 5.2.4.

E. Grate Inlets on Grade

The capacity of a grate inlet depends upon its geometry, cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb-opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate (frontal flow) is intercepted by grate inlets, and a small portion of the flow along the length of the grate (side flow) is intercepted. On steep longitudinal slopes, a portion of the frontal flow may tend to splash over the end of the grate for some grates.

The interception capacity of a combination inlet consisting of a curb opening and grate placed side-by-side is no greater than that of the grate alone. Capacity is computed by neglecting the curb opening.

The ratio of frontal flow to total gutter flow, Eo, for a straight cross slope is given by the following equation:

$$\boxed{E_o = Q_w/Q = 1 - (1 - W/T)^{2.67}}$$

***(Eq. 5.1)* Frontal Flow Ratio**

Where:

Q_w	=	flow in width W, ft^3/s
Q	=	total gutter flow, ft^3/s
W	=	width of depressed gutter or grate, ft
T	=	total spread of water in the gutter, ft

The ratio of side flow, Qs, to total gutter flow is:

$$\boxed{Q_s/Q = 1 - Q_w/Q = 1 - E_o}$$

(Eq. 5.2)
Side Flow Ratio

The ratio of frontal flow intercepted to total frontal flow, Rf, is expressed by the following equation:

$$\boxed{R_f = 1 - 0.09\,(V - V_o)}$$

(Eq. 5.3)
Frontal Flow Intercept.

Where:

V = velocity of flow in the gutter, ft/s

V_o = gutter velocity where splash-over first occurs, ft/s

This ratio is equivalent to frontal-flow interception efficiency. Figure 5-1 (from HEC-22, Chart 5) *(FHWA, 2009)* provides a solution of Equation 5-3 that incorporates grate length, bar configuration, and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 5-1 is total gutter flow divided by the area of flow. Figure 5-1 shows that parallel bar grates are the most efficient grates on steep slopes but they are not bicycle safe without adding transverse bars. Grate types such as reticuline and curved vane are considered bicycle safe.

The equations provided in Table 5-2 (from *AASHTO, 2014*) can be used to determine splash-over velocities (Vo) for various grate configurations. Equation 5-3 can then be used to compute the portion of frontal flow intercepted by the grate.

Table 5-2: Splash-Over Velocity Equations

GRATE CONFIGURATION	TYPICAL BAR SPACING (IN.)	SPLASH-OVER VELOCITY EQUATION
Parallel Bars (P-1$^7/_8$)	2.0	$V_o = 2.218 + 4.031L - 0.649L^2 + 0.056L^3$
Parallel Bars (P-1$^1/_8$)	1.2	$V_o = 1.762 + 3.117L - 0.451L^2 + 0.033L^3$
Curved Vane	4.5	$V_o = 1.381 + 2.78L - 0.300L^2 + 0.020L^3$
45° Tilt Bar	4.0	$V_o = 0.988 + 2.625L - 0.359L^2 + 0.029L^3$
Parallel Bars with Transverse Rods (P-1$^7/_8$-4)	2.0 Parallel/ 4.0 Transverse	$V_o = 0.735 + 2.437L - 0.265L^2 + 0.018L^3$
30° Tilt Bar	4.0	$V_o = 0.505 + 2.344L - 0.200L^2 + 0.014L^3$
Reticuline	N/A	$V_o = 0.030 + 2.278L - 0.179L^2 + 0.010L^3$

The ratio of side flow intercepted to total side flow, Rs, or side-flow interception efficiency, is expressed by:

$$\boxed{R_s = 1 \,/\, [1 + (0.15V^{1.8} \,/\, S_x L^{23})]}$$

(Eq. 5.4)
Side Flow Intercept.

Where:

V = velocity of flow in the gutter, ft/s

S_x = cross slope, ft/ft

L = length of the grate, ft

The efficiency, *E*, of a grate is expressed as:

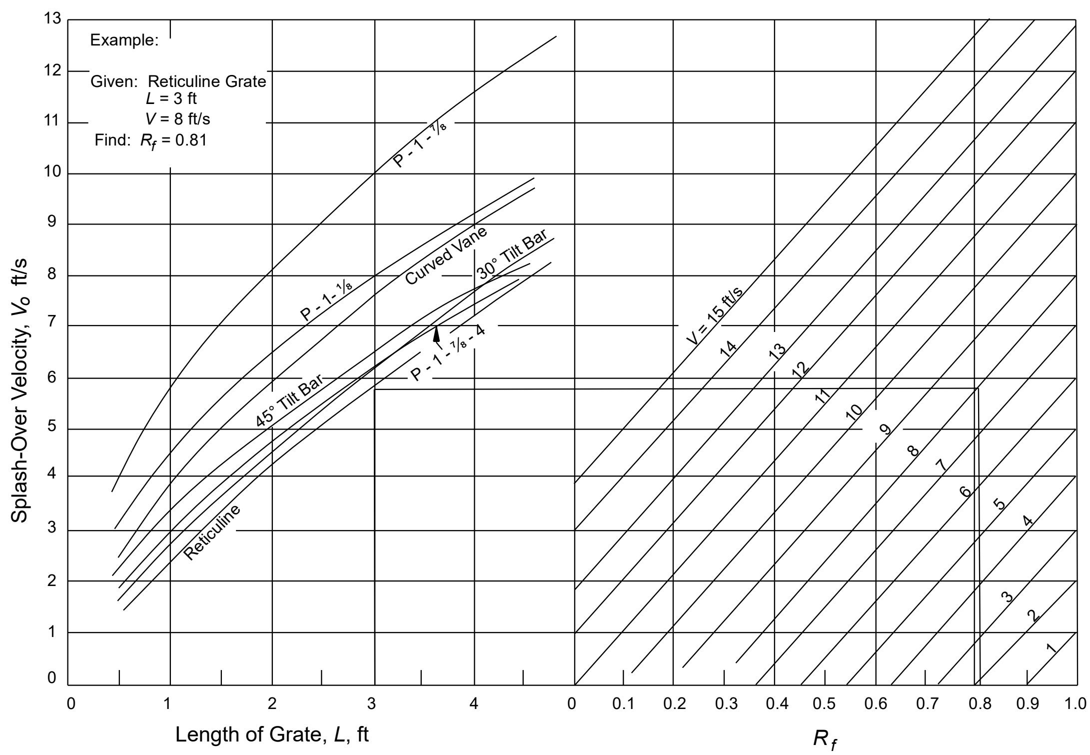
$$\boxed{E \;=\; R_f E_o + \; R_s(1 - E_o)}$$

(Eq. 5.5)
Efficiency of Grate

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$\boxed{Q_i = \; EQ \;\; = \; Q[R_f E_o + \; R_s(1 - E_o)]}$$

(Eq. 5.6)
Grate Intercept. Capacity



Source: HEC-22, 2009

Figure 5-1: Grate Inlet Frontal-Flow Interception Efficiency

F. Grate Inlets in Sumps

Although curb-opening inlets are generally preferred to grate inlets at a sag, grate inlets can be used successfully. For minor sag points where debris potential is limited, grate inlets without a curb-opening inlet can be utilized. An example of a minor sag point might be on a ramp as it joins a main line. Curb-opening inlets in addition to a grate are preferred at sag points where debris is likely. For major sag points, such as on divided high-speed highways, a curb-opening inlet is preferable to a grate inlet because of its hydraulic capacity and debris-handling capabilities. When grates are used, it is good practice to assume that half the grate is clogged with debris.

Flanking inlets may be necessary. See Section 5.2.4.

A grate inlet in a sag operates as a weir up to depths dependent on the size of the grate and as an orifice at greater depths. Grates of larger dimension will operate as weirs to greater depths than smaller grates.

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

$$\boxed{Q_i = C_w P d^{1.5}}$$

(Eq. 5.7)
Capacity of Grate

Where:

P = perimeter of grate excluding bar widths and side against curb, ft

C_w = 3.0, weir coefficient

d = average depth across the grate ($0.5(d_1 + d_2)$), ft (see Figure 5-2)

- Equation (5-9) is applicable only when *d* is less than 0.4 feet
- When no curb opening is present, compensate for clogging by using 75% of grate perimeter.
- Combination inlets will be designed using the full perimeter. The curb opening will be considered a factor of safety

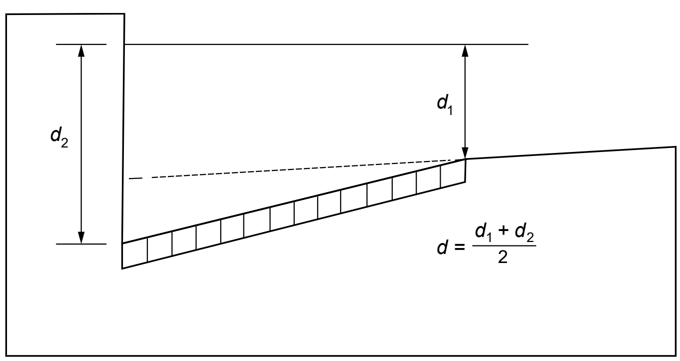


Figure 5-2: Grate Inlet in Sump Condition

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

$$Q_i = C_o A_g (2gd)^{0.5}$$

(Eq. 5.8)
Capacity of Grate - Orifice

Where:

C_o = 0.67, orifice coefficient

A_g = clear opening area of the grate, ft^2

g = 32.2 ft/s^2

d = average depth across the grate ($0.5(d_1 + d_2)$), ft

The use of Equation 5-8 requires the clear opening area of the grate, which is obtained by multiplying the total area by the opening ratios given in the following table (from *HEC-22, 2009*):

Table 5-3: Opening Ratios for Various Grate Types

GRATE	OPENING RATIO
P-1$^7/_8$-4	0.8
P-1$^7/_8$	0.9
P-1$^1/_8$	0.6
Reticuline	0.8
Curved vane	0.35
Tilt-bar	0.34

G. Curb Opening Inlets on Grade

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

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

$$L_T = KQ^{0.42}(S_L)^{0.3}[1/(nS_x)]^{0.6}$$

(Eq. 5.9)
Curb-opening Inlet Length

Where:

L_T = curb-opening length required to intercept 100% of the gutter flow, ft

K = 0.6

FHWA research has indicated that Equation 5.9 generally overestimates the flow accepted by the curb-opening inlet. The research recommends a different equation and procedure for estimating curb-opening inlet length on grade for 100 percent capture. For locations where 100 percent capture on-grade is needed, designers should consider following the process outlined in FHWA publication number FHWA-HRT-22-061 to determine the minimum length of a curb-opening.

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

$$\boxed{E = 1 - (1 - L / L_T)^{18}}$$

(Eq. 5.10)
Curb-opening Inlet Effic.

Where:

L = actual curb-opening length, ft

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections or for a continuously depressed gutter (composite gutter) can be found by the use of an equivalent cross slope, S_e, in Equation 5-11:

$$\boxed{S_e = S_x + S'_w E_o}$$

(Eq. 5.11)
Equiv. Cross Slope

Where:

S'_w = (a/12W) = $S_w - S_x$ = cross slope of the gutter measured from the cross slope of the pavement, ft/ft, in which

a = gutter depression, in. (see Figure 5-3)

E_o = ratio of flow in the depressed section to total gutter flow. It is determined by the gutter configuration upstream of the inlet (see Equation 5-1).

Note: S_e can be used to calculate the length of curb opening by substituting S_e for S_x in Equation 5-9.

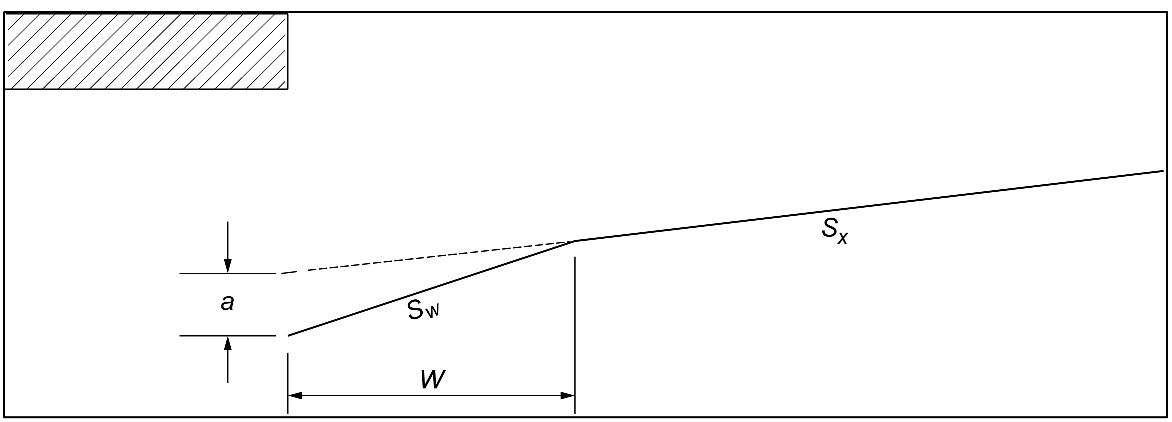


Figure 5-3: Curb Opening Inlet Geometry

The design and evaluation of MDOT-SHA standard curb opening inlets will be perform using a value for the local gutter depression (*a*) of 1.5 inches. This value is subject to change.

H. Curb Opening Inlets in Sumps

The capacity of a curb-opening inlet in a sag depends on the water depth at the curb, the curb-opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage. See Figure 5-4 for a definition sketch.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$\boxed{Q_i = C_W(L + 1.8W)d^{1.5}}$$

(Eq. 5.12)
Depressed Curb-opening

Where:

CC_W = 2.3 (with depression)

L = length of curb opening, ft

W = width of depression, ft

d = depth of water at curb measured from the normal cross slope, ft (i.e., d = TSx for a uniform gutter and d = a/12 + TSx for a composite section

The weir equation is applicable to depths at the curb less than or equal to the height of the opening plus the depth of the depression ($D \leq h + a$).

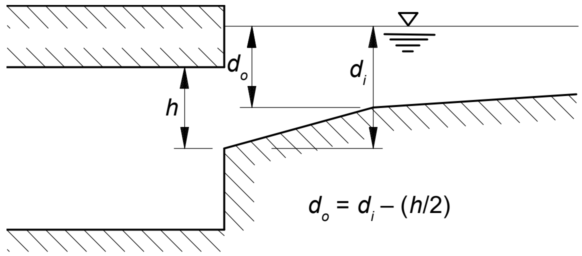
The weir equation for curb-opening inlets without a depression becomes:

$$\boxed{Q_i = C_W L d^{1.5}}$$

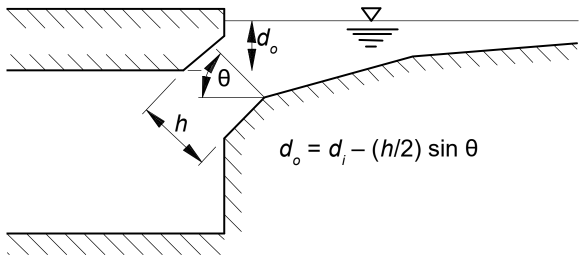
(Eq. 5.13)
Weir Equation

C_W = 3.0 (without depression)

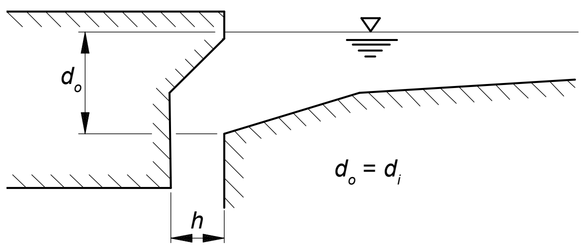
The depth limitation for operation as a weir becomes $d \leq h$.



a. Horizontal Throat



b. Inclined Throat



c. Vertical Throat

Figure 5-4: Throat Configuration of Curb Opening Inlets

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the height of curb opening (1.4h). The interception capacity can be computed by Equation 5-14. The depth at the inlet includes any gutter depression:

$$\boxed{Q_i = C_o h L[2g(d_o)]^{0.5}}$$

(Eq. 5.14)
Interception Capacity

Where:

C_o = orifice coefficient (0.67)

h = height of curb-opening orifice, ft

L = length of orifice opening, ft

d_o = effective head on the center of the orifice throat, ft (see Figure 5-5)

d_i = depth at lip of curb opening, ft = d + a/12 = TS_x + a/12 (a = local depression)

The following applies to the local depression.

Weir Flow: *a* = local gutter depression, in. This depression is used to determine if the inlet is in weir flow only. Local depression is only used to check if the orifice is submerged. The design and evaluation of MDOT-SHA standard curb opening inlets will be perform using a value for the local gutter depression (a) of 1.5 inches. This value is subject to change.

Orifice Flow: *a* = local depression at curb opening, in. This depression is used in orifice flow inlet capacity computations. The design and evaluation of MDOT-SHA standard curb opening inlets will be perform using a value for the local gutter depression (a) of 1.5 inches. This value is subject to change.

5.3.4 Pipes

A. General

Storm drain systems shall comply with Chapter 4, Section 4.10 on pipe material. Corrugated metal pipe arch and elliptical reinforced concrete pipe may be used only with lack of available cover makes use of round pipe impractical or when their use is dictated by hydraulic considerations. Minimum cover requirements are provided in Section 4.9.4. A 50-year minimum pipe material service life is required, see Section 4.10 for additional service life guidance.

In closed storm drain systems, longitudinal pipe shall not be less than 15 inches in diameter between the first two structures and not less than 18 inches in diameter thereafter. In closed storm drain systems, transverse pipe shall not be less than 18 inches in diameter for pipe lengths 60 feet or less. A minimum pipe size of 24 inches in diameter is required for transverse pipes for lengths greater than 60 feet. See Figure 5-5 below.

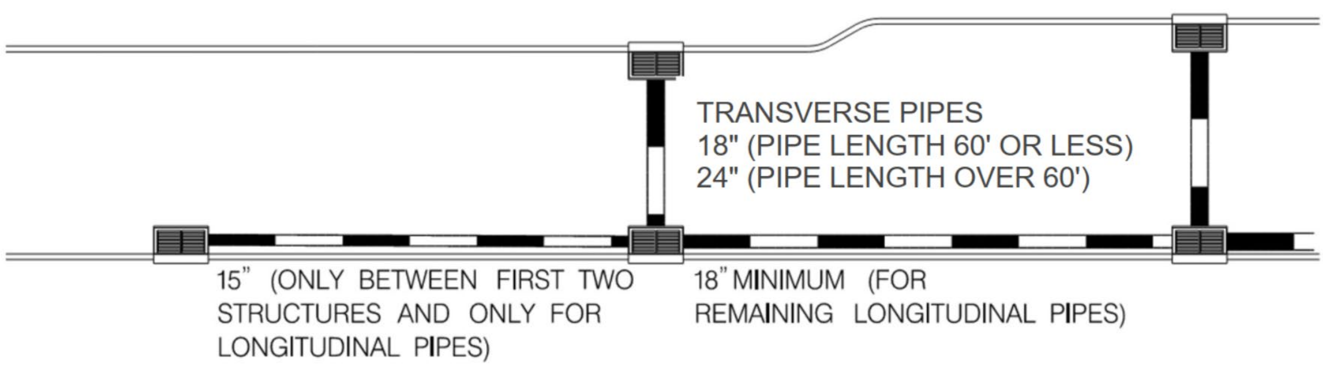


Figure 5-5: Minimum Pipe Sizes

B. Design

All closed system shall be initially sized so that full flow capacity is slightly greater than the flow computed for the design storm as required by Table 5-1. Full flow capacity shall be determined using Manning's Formula, with the proper roughness coefficient 'n', as selected from Table 4-13 (see section 4.15). The desirable minimum velocity in pipes flowing full shall be three (3) feet per second. The minimum pipe slope for closed storm drain system shall be 0.50%.

Although on grade inlets are spaced to pick up only a portion of the gutter flow, storm drain pipes shall be designed to carry the total flow from the design storm at any location. Tabulate storm drain flow computations using Figure 5-16 or other formats approved by Highway Hydraulics Division.

5.3.5 Storm Drain Outfall Stability

Stability should be evaluated at the storm drain system outfall and the designer should provide the type of dissipation appropriate for velocity, pipe size, discharge and site constraints. For storm drain systems, the outfall must be stable for the 10-year storm. If the outlet discharges flow downstream of a headwall or flow entrance that could be considered a culvert, then culvert outlet protection must be provided as described in Section 4.8. Storm drain systems that function as spillways for Stormwater Management Facilities should be designed to ensure outfall stability for the design storm or the 10-year storm, whichever is greater.

5.3.6 Access Holes

Access Holes are used to provide entry to continuous underground storm drains for inspection and cleanout. Grate inlets can be used in lieu of manholes when entry to the system can be provided at the grate inlet. Locations where manholes should be located are:

- where two or more storm drains converge,
- where cleanouts or inspections, or both, may be required, or
- where storm drain alignment or grade changes.

Access Holes must be provided when the length of pipe exceeds 400 feet. For low-slope pipes (<1%), small diameter pipes (<18 inches), and for pipes especially subject to sediment and debris, closer spacing of Access Holes is encouraged. When a state-maintained storm drain system is connected to a local, federal, or private system, ensure there is an access hole near the state right-of-way boundary.

The outside diameter of all pipes entering the access hole or inlet structure shall fit between the face of the walls. A minimum structural leg width between adjacent pipe openings of 6 inches shall be provided.

Where two or more pipes are connected to an access hole or inlet structure, channeled half benching inverts shall be provided. See Figures 5-12 through 5-14 for typical benching and channelization configurations.

5.3.7 Bend Structures

Bend Structures reduce energy losses and should be provided on all storm drains which are 30 inches in diameter and larger where the length of bend, as computed with Eq. 5-15, is greater than 5 feet. The radius of bend should be a minimum of 2 ½ times the pipe diameter.

$$\boxed{L = 2\pi R\,(\Delta/360)}$$

(Eq. 5.15)
Bend Length

$$\boxed{T = R\,\mathrm{Tan}\,\tfrac{1}{2}\,\Delta}$$

(Eq. 5.16)
Tangent Length

Where:

L = length of bend, ft

Δ = deflection angle, degrees

R = radius, centerline of bend, ft

T = tangent length, ft

Bend Structures shall include an access hole where, in the designer's judgement, a means of physical access may be necessary for maintenance purposes.

5.4 ANALYSIS

5.4.1 Hydraulic Gradient Calculations

A. Beginning Elevation

All hydraulic gradient calculations must comply with Section 5.4 and be recorded on Figure 5-17 or approved substitute. When a free outfall is expected, new and existing systems should be designed by beginning the hydraulic gradient at the outfall with an elevation equal to the pipe crown.

For systems without free outfalls, the hydraulic gradient should be started with an estimated or (when possible) the computed tail water elevation.

When the proposed storm drain discharges into an existing storm drain system, the beginning elevation for the hydraulic gradient can be determined as follows:

a. If sufficient data is available, calculate the hydraulic gradient through the existing system and extend it through the proposed system (for each design storm).

b. If the gradient in the existing system cannot be computed, select the highest structure in the existing system which will flood away from the SHA roadway and assume a flooding condition at this structure, i.e., begin the gradient at this structure using the grate or manhole cover elevation as the hydraulic gradient elevation.

B. Pipe and Structure Losses

After determining the beginning elevation, E_1, calculate the head loss H_f due to friction in the pipe from point 1 to point 2. (See Figure 5-6)

$$\boxed{H_f = S_f L}$$

(Eq. 5.17)
Frictional Head Loss

Where:

H_f = Frictional head loss, ft

S_f = Frictional slope of the pipe, ft/ft

L = Length of pipe between structures, ft

The frictional pipe slope may be determined by rewriting Manning's equation as follows:

$$S_f = [\,Qn/(1.486\ AR^{\frac{2}{3}})]^2$$

(Eq. 5.18)
Frictional Pipe Slope

Where:

Q = Rate of flow, ft^3/s

n = Manning's roughness coefficient

R = Hydraulic radius, ft = area of flow (A) divided by the wetted perimeter (WP)

S_f = The slope of the hydraulic grade line, ft/ft

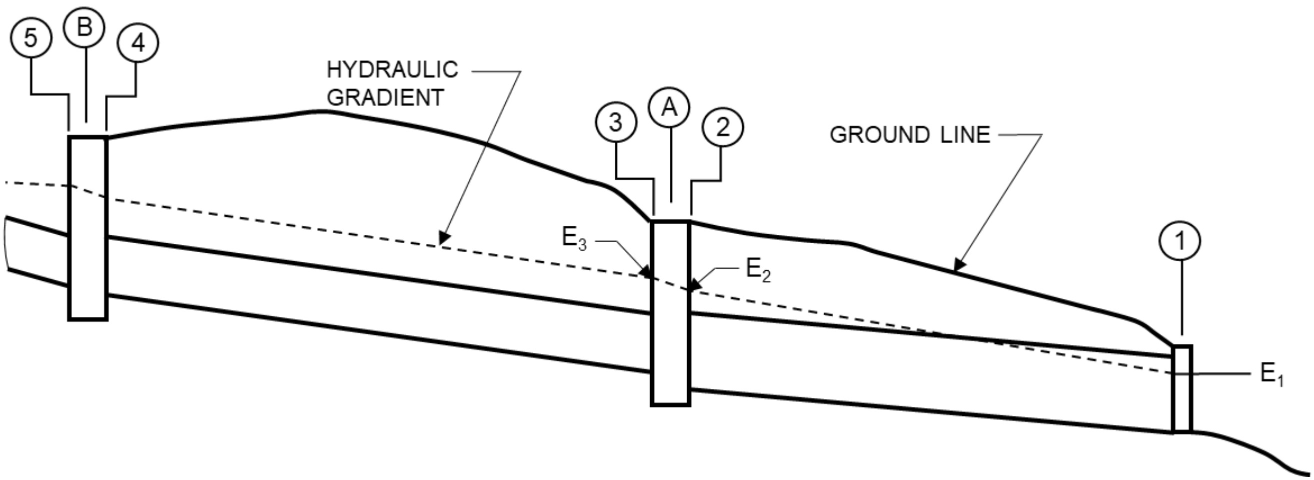


Figure 5-6: Pipe and Structure Losses

This frictional head loss is now added to the beginning elevation, E_1. This new elevation, E_2, is the hydraulic gradient at point 2. Now calculate the head loss due to structure A. The magnitude of the structure loss is dependent on the type of structure (i.e. inlet, manhole, or bend) and the angle between the incoming and outgoing pipes. It is computed by the following formula.

$$H_b = K_b\left(\frac{{V_f}^2}{2g}\right)$$

(Eq. 5.19)
Structure Head Loss

Where:

H_b = Head loss, ft

K_b = Head loss coefficient (See Table 5-4 in Section 5.7)

V_f = Frictional Velocity in the outlet pipe, ft/s (The velocity for the given q and d = ho)

g = Acceleration due to gravity (32.2 ft/s^2)

This loss may also be determined by the appropriate chart. The structure loss at a field connection is the same as that for a manhole.

The structure loss, H_b, is now added to the hydraulic gradient. This elevation, E_3, is the new beginning elevation to compute the hydraulic gradient up to structure B. Repeat this procedure for the entire system.

Note: The last structure at the top of a closed system (i.e. Inlets) shall be treated as a headwall and the head water computed as outlined for open culverts with the tailwater equal the hydraulic gradient in the previous structure.

<u>Manhole Losses (HEC-22 Method)</u>

HEC-22 contains a detailed loss-calculation method, which is included as an option in some storm drain design software.

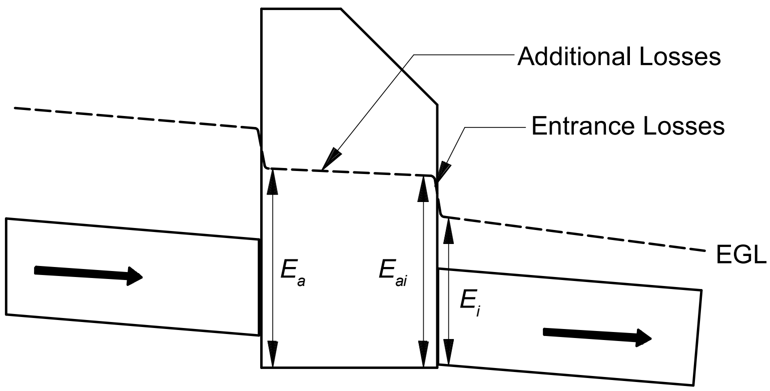


Figure 5-7: Definition Sketch for HEC-22 (2009) Manhole Loss Method

$$\boxed{K_b = K_o C_D C_d C_Q C_p C_B}$$

(Eq. 5.20)
Manhole Loss Coefficient

Where:

K_b = adjusted loss coefficient

K_o = initial head loss coefficient based on relative manhole size

C_D = correction factor for pipe diameter (pressure flow only)

C_d = correction factor for flow depth (non-pressure flow only)

i = correction factor for relative flow

C_p = correction factor for plunging flow

C_B = correction factor for benching

The equations for calculating the above correction factors were initially found in the 2001 edition of HEC-22. FHWA has improved the above method and published the following method in the 2009 update to *HEC-22*. The method involves three fundamental steps (with terms as defined in Figure 5-7).

Step 1 Determine an initial manhole energy level (E_{ai}) based on inlet control (weir and orifice) or outlet control (partial and full flow) equations.

Step 2 Adjust the initial manhole energy level based on benching, inflow angle(s), and plunging flows to compute the final calculated energy level (E_a).

Step 3 Calculate the exit loss from each inflow pipe and estimate the energy grade line (EGL), which will then be used to continue calculations upstream

C. Junctions

When two pipes feed into one structure, as illustrated in Figure 5-8, the controlling angle is determined by the following method.

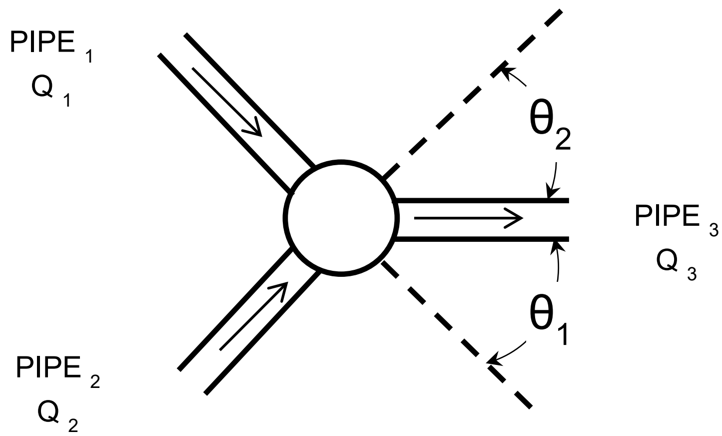


Figure 5-8: Pipe Junctions

Using the 10-year storm flow data from Figure 5-16:

- Determine $V_{1/3}$ the friction velocity of Q_1 in pipe 3
- Determine $V_{2/3}$ the friction velocity of Q_2 in pipe 3
- With $V_{1/3}$ and θ_1 determine the structure loss $L\theta_1$
- With $V_{2/3}$ and θ_2 determine the structure loss $L\theta_2$
- If $L\theta_2$ is greater, θ_1 is the controlling angle θ_c
- If $L\theta_1$ is greater, θ_2 is the controlling angle θ_c

The controlling angle is used to calculate the loss in that structure. Determine the friction velocity of Q_3 in pipe 3 and use θ_c for H_b, the head loss at that structure.

D. General Limitations

When the hydraulic gradient is being computed, the designer must have available either a profile of the system or a list of invert elevations and structure flooding elevations against which each computed gradient elevation may be checked.

After computing the friction loss in a section of pipe and determining the hydraulic gradient elevation at the upstream end of that section, the designer should also compute the normal depth elevation at this point. When a pipe is flowing less than full (not under pressure) the elevation and the water surface elevation at any point are the same. This depth of flow is called the normal depth.

The normal depth elevation is the normal depth plus the invert elevation at that point. If the computed hydraulic gradient elevation is lower than the normal depth elevation, the gradient must be adjusted to the normal depth elevation at that point.

Each time a structure loss is computed and added to the hydraulic gradient; the resulting gradient elevation should be compared to the flooding elevation for the structure. Structures without surface access cannot flood and need not be checked. When the gradient exceeds the prescribed limits, two practical measures are available to reduce the gradient elevation without changing alignment and/or structure type:

1. Lower the entire system or if one section of pipe flows at normal depth, lower the section of the system above that pipe.

2. Increase the capacity of the pipe below the flooding structure by increasing size and/or decreasing roughness coefficient or, when this is not sufficient, by use of multiple pipes

5.5 ROADWAY

5.5.1 Introduction

This Section discusses the role of roadway geometrics on pavement drainage applicable to the hydraulic design of storm drain systems. Where applicable, the discussion extracts information from or references the AASHTO's *A Policy on Geometric Design of Highways and Streets (AASHTO, 2018)*, also known as the "AASHTO Green Book". This Section does not discuss the following pavement drainage considerations:

- Bridge decks,
- Roadside channels, (see Chapter 3 "Channels"), and
- Fill slopes, see AASHTO Green Book.

Roadway geometric features that impact gutter, inlet and pavement drainage for storm drain systems include:

- Roadway width and cross slope,
- Vertical alignment,
- Pavement texture,
- Curb and gutter sections), and
- Presence of median barriers.

The pavement width, cross slope, profile and pavement texture control the time it takes for stormwater to drain to the gutter section. The gutter cross section and longitudinal slope control the quantity of flow that can be carried in the gutter section. Each of these is discussed in the following sections.

5.5.2 Roadway Cross Section

A. Width

In general, the wider the roadway width (i.e., traveled way plus shoulder/curb offset width), the greater the quantity of water that must be accommodated by the curb and gutter.

B. Cross Slope

The design of pavement cross slope is a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. The AASHTO Green Book reports that cross slopes of 2 percent have little effect on driver effort in steering, especially with power steering or on friction demand for vehicular stability. Use of a cross slope steeper than 2 percent on pavements with a central crown line is not desirable. In areas of intense rainfall, a somewhat steeper cross slope may be necessary to facilitate drainage. In such areas, the cross slope may be increased to 2.5 percent.

When three or more lanes are inclined in the same direction on multi-lane pavements, it is desirable that each successive pair of lanes, or the portion thereof outward from the first two lanes from the crown line, have an increased slope. The two lanes adjacent to the crown line should be pitched at the normal slope and successive lane pairs, or portions thereof outward, should be increased by approximately 0.5 percent to 1 percent. Where three or more lanes are provided in each direction, the maximum pavement cross slope should be limited to 4 percent.

It is desirable to provide a break in cross slope at two lanes, with three lanes being the upper limit. Although not widely encouraged, inside lanes can be sloped toward the median. This should not be used unless four continuous lanes or some physical constraint on the roadway elevation occurs, because inside lanes are used for high-speed traffic and the allowable water depth is lower. Median areas should not be drained across traveled lanes. A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas, and consideration should be given to increasing cross slopes in sag vertical curves and crest vertical curves, and in sections of flat longitudinal grades. Where curbs are used, depressed gutter sections can be effective at increasing gutter capacity and reducing spread on the pavement.

5.5.3 Vertical Alignment

A. Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement because of the impact on the spread of stormwater against the curb. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Desirable longitudinal gutter grades should be not less than 0.5 percent for curbed pavements with an absolute minimum of 0.3 percent. Minimum grades can be maintained in very flat terrain by use of a rolling profile.

B. Sag Vertical Curves

On curbed facilities, sag vertical curves should be sufficiently “sharp” to prevent inadequate drainage near the bottom of the vertical curve. This can be achieved by designing the sag vertical curve to provide a minimum longitudinal slope of 0.3 percent at the two points 50 ft from the bottom. This yields a maximum K value of 167 for the vertical curve, which is typically called the drainage maximum.

The bottom of the vertical curve should not be located in the crosswalk of pedestrian paths.

C. Crest Vertical Curves

Drainage considerations are not as critical on crest vertical curves as sag vertical curves. However, good design practice is to design crest vertical curves based on a maximum K value of 167 for curbed roadways or where concrete barrier is used.

D. Pavement Texture

The pavement texture is an important consideration for roadway surface drainage. Although the hydraulics engineer will have little control over the selection of the pavement type or texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. Macrotexture provides a channel for water to escape from the tire/pavement interface and, thus, reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tining new concrete pavements while it is still in the plastic state. Re-texturing of an existing concrete surface can be accomplished through pavement grooving and cold milling. Both longitudinal and transverse grooving are very effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. Combinations of longitudinal and transverse grooving provide the most adequate drainage for high-speed conditions.

E. Curb and Gutter

Curbing at the outside edge of pavements is normal practice for low-speed (≤ 45 mph), urban highway facilities. Curbs serve several purposes:

- Containing the surface runoff within the roadway and away from adjacent properties,
- Preventing erosion,
- Providing pavement delineation, and
- Enabling access control and the orderly development of property adjacent to the roadway.

Curbs may be either barrier or mountable type, and they are typically concrete, although bituminous curb is used occasionally. Barrier curbs range in height from 6 to 10 inches. Mountable curbs are less than 6 inches in height and have rounded or plane-sloping faces. Gutters are available in widths ranging from 1 to 3 feet.

A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility that can convey runoff of a lesser magnitude than the design flow without interruption of the traffic. When a design storm flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders and portions of the traveled surface. This is the width the hydraulics engineer is most concerned with in curb and gutter flow, and limiting this width becomes a very important design criterion.

Where practicable, it is desirable to intercept runoff from cut slopes and other areas draining toward the roadway, before it reaches the highway, to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water that must be carried in the gutter section. Section 5.3.2 discusses the allowable water spread.

F. Medians

Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled pavement. The following applies to surface drainage considerations on facilities with medians that are not depressed:

Flush Medians
Flush medians consist of a relatively flat paved area separating the traffic lanes with only painted stripes on the pavement. Flush medians should be either slightly crowned to avoid ponding of water in the median area or slightly depressed (with median drains) to avoid carrying all surface drainage across the travel lanes.

Curbed Medians
Curbed, raised medians are most commonly used on lower-speed (≤ 45 mph) urban arterials. The roadway is typically crowned to transport a portion of the pavement drainage to the outside and a portion to the median, which then requires a collection and conveyance system for the median drainage.

Median Barriers
With narrow medians on high-speed facilities (e.g., Interstates), a median barrier may be used to prevent out-of-control vehicles from crossing into opposing traffic lanes. When median barriers are used, it is necessary to provide inlets, especially on horizontal curves with superelevation, and connecting storm drains to collect the water that accumulates against the barrier

5.6 References

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National Cooperative Highway Research Program *Research Results Digest, Issue Number 243: Proposed Design Guidelines for Reducing Hydroplaning on New and Rehabilitated Pavements*. NCHRP, Transportation Research Board, Washington, DC, 1999.

5.7 DESIGN AIDS

This section presents several tables, figures, and forms required for the hydraulic design of culverts. These include:

Tables and Charts

- Table 5-4 K_b Values — Formerly SHA-61.1-408.0
- Figure 5-9 Head Losses in Inlets — Formerly SHA-61.1-408.1
- Figure 5-10 Head Losses in Manholes — Formerly SHA-61.1-408.2
- Figure 5-11 Head Losses in Bend Structures — Formerly SHA-61.1-408.3
- Figure 5-12 Access Hole Benching Methods
- Figure 5-13 Round Structure Channelization
- Figure 5-14 Square/Rectangular Structure Channelization

Forms

- Figure 5-15 Inlet Spacing — Formerly SHA-61.1-491
- Figure 5-16 Storm Sewer Design — Formerly SHA-61.1-492
- Figure 5-17 Hydraulic Gradient for Storm Sewers — Formerly SHA-61.1-493

Table 5-4: K_b Values

ANGLE	INLET	MANHOLE	BEND STRUCTURE	ANGLE	INLET	MANHOLE	BEND STRUCTURE
0	0.50	0.15	0.00	46	1.11	0.76	0.18
1	0.51	0.16	0.00	47	1.12	0.76	0.18
2	0.51	0.18	0.01	48	1.13	0.77	0.18
3	0.53	0.19	0.01	49	1.14	0.78	0.19
4	0.54	0.20	0.02	50	1.15	0.78	0.19
5	0.54	0.22	0.02				
6	0.55	0.23	0.03	51	1.16	0.79	0.19
7	0.56	0.24	0.03	52	1.17	0.80	0.19
8	0.57	0.26	0.04	53	1.18	0.80	0.20
9	0.58	0.27	0.04	54	1.19	0.81	0.20
10	0.59	0.28	0.04	55	1.20	0.82	0.20
				56	1.21	0.82	0.20
11	0.60	0.30	0.05	57	1.22	0.83	0.21
12	0.61	0.31	0.05	58	1.23	0.84	0.21
13	0.62	0.32	0.06	59	1.24	0.84	0.21
14	0.62	0.34	0.06	60	1.25	0.85	0.21
15	0.63	0.35	0.07				
16	0.64	0.36	0.07	61	1.26	0.86	0.21
17	0.65	0.38	0.08	62	1.27	0.86	0.22
18	0.66	0.39	0.08	63	1.28	0.87	0.22
19	0.67	0.40	0.08	64	1.28	0.87	0.22
20	0.68	0.42	0.09	65	1.29	0.88	0.22
				66	1.30	0.88	0.22
21	0.69	0.43	0.09	67	1.31	0.89	0.22
22	0.70	0.44	0.10	68	1.32	0.89	0.22
23	0.71	0.46	0.10	69	1.33	0.90	0.22
24	0.73	0.47	0.11	70	1.33	0.90	0.23
25	0.74	0.48	0.11				
26	0.76	0.50	0.11	71	1.34	0.91	0.23

Table 5-4: K_b Values (continued)

ANGLE	INLET	MANHOLE	BEND STRUCTURE	ANGLE	INLET	MANHOLE	BEND STRUCTURE
27	0.78	0.51	0.12	72	1.35	0.91	0.23
28	0.80	0.52	0.12	73	1.36	0.92	0.23
29	0.82	0.54	0.12	74	1.37	0.92	0.23
30	0.83	0.55	0.13	75	1.38	0.93	0.23
				76	1.38	0.93	0.23
31	0.85	0.56	0.13	77	1.39	0.94	0.23
32	0.87	0.58	0.13	78	1.40	0.94	0.24
33	0.89	0.59	0.14	79	1.41	0.95	0.24
34	0.90	0.60	0.14	80	1.42	0.95	0.24
35	0.92	0.62	0.14				
36	0.94	0.63	0.15	81	1.43	0.96	0.24
37	0.96	0.64	0.15	82	1.43	0.96	0.24
38	0.98	0.66	0.15	83	1.44	0.97	0.24
39	0.99	0.67	0.16	84	1.45	0.97	0.24
40	1.01	0.68	0.16	85	1.46	0.98	0.24
				86	1.47	0.98	0.25
41	1.03	0.70	0.16	87	1.48	0.99	0.25
42	1.05	0.71	0.17	88	1.48	0.99	0.25
43	1.06	0.72	0.17	89	1.49	1.00	0.25
44	1.08	0.74	0.17	90	1.50	1.00	0.25
45	1.10	0.75	0.18				

Inlets	Kb derived from AASHTO Volume 2, Chapter 13, Table 13-5
Manhole	Kb derived from AASHTO Volume 2, Chapter 13, Table 13-5
Bends	Kb derived from APWA Special Report No. 49, 1981

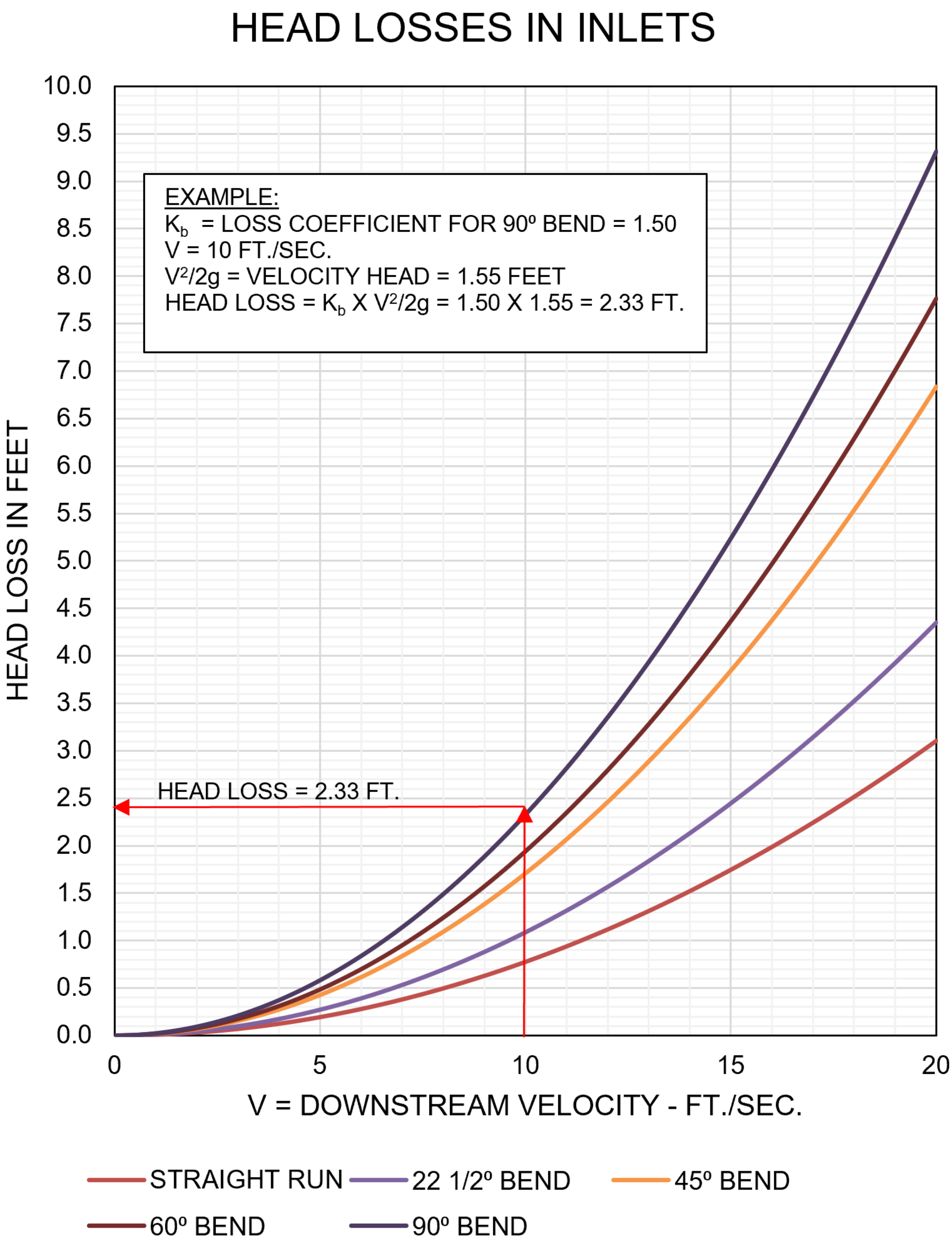


Figure 5-9: Head Losses in Inlets

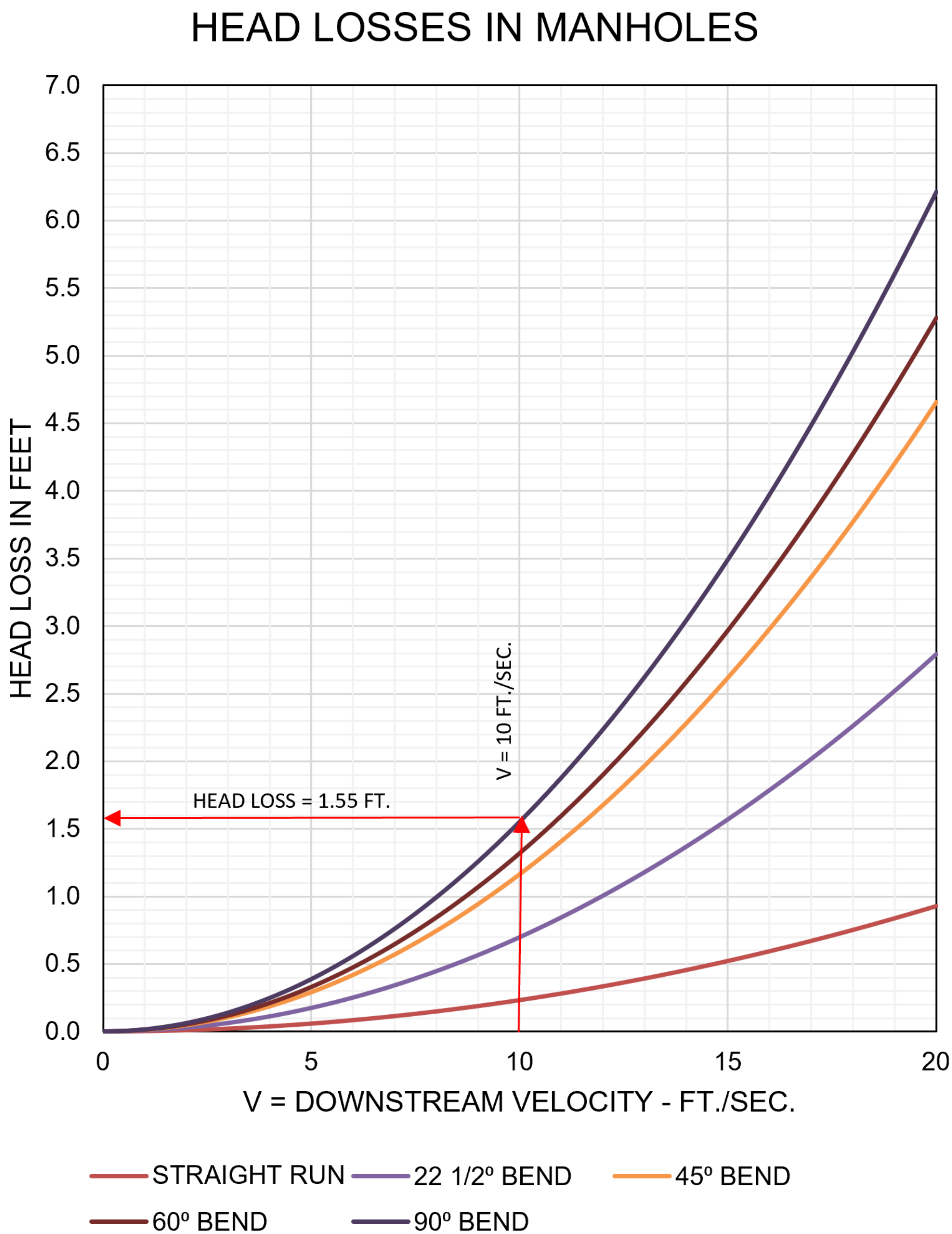


Figure 5-10: Head Losses in Manholes

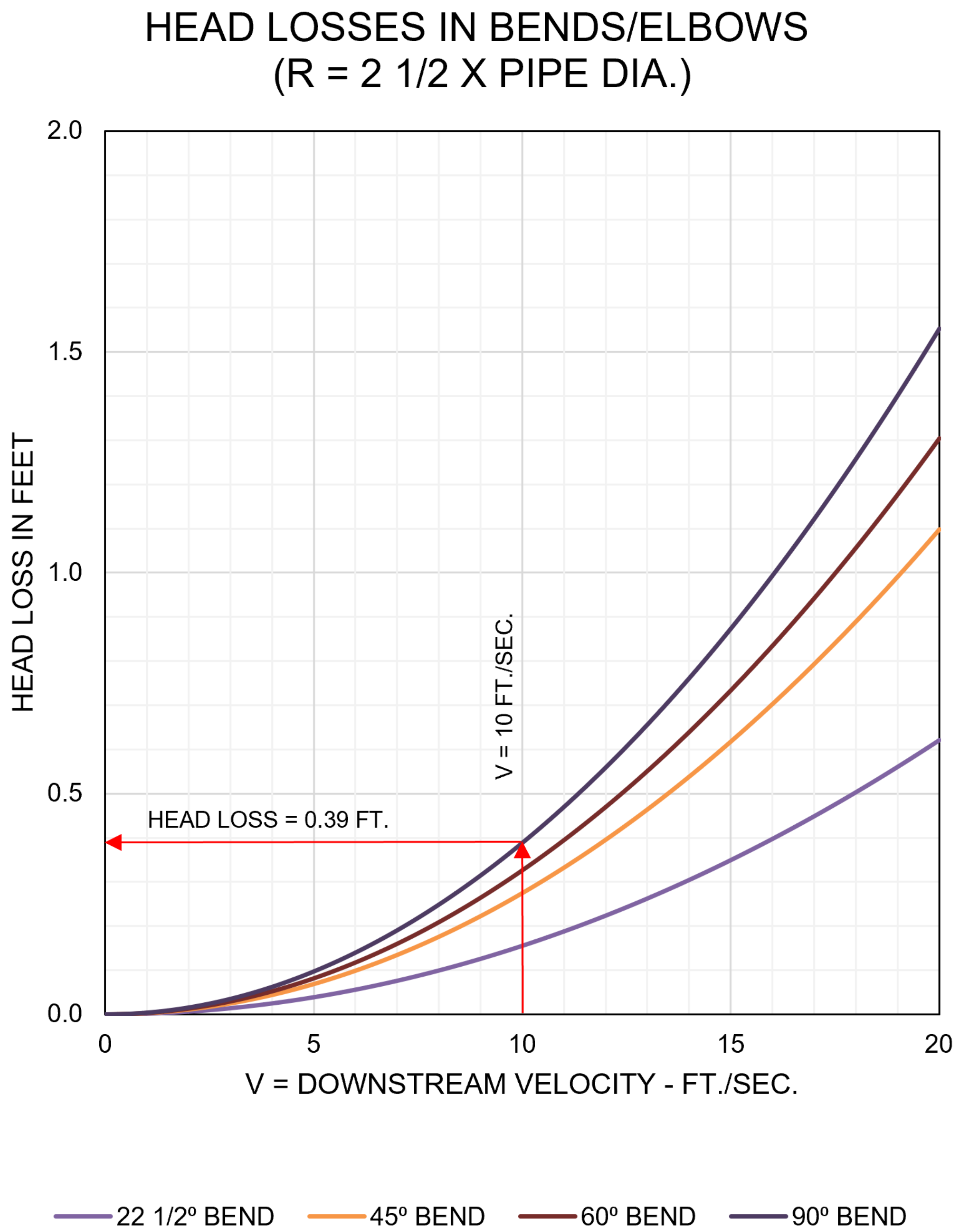


Figure 5-11: Head Losses in Bends/Elbows

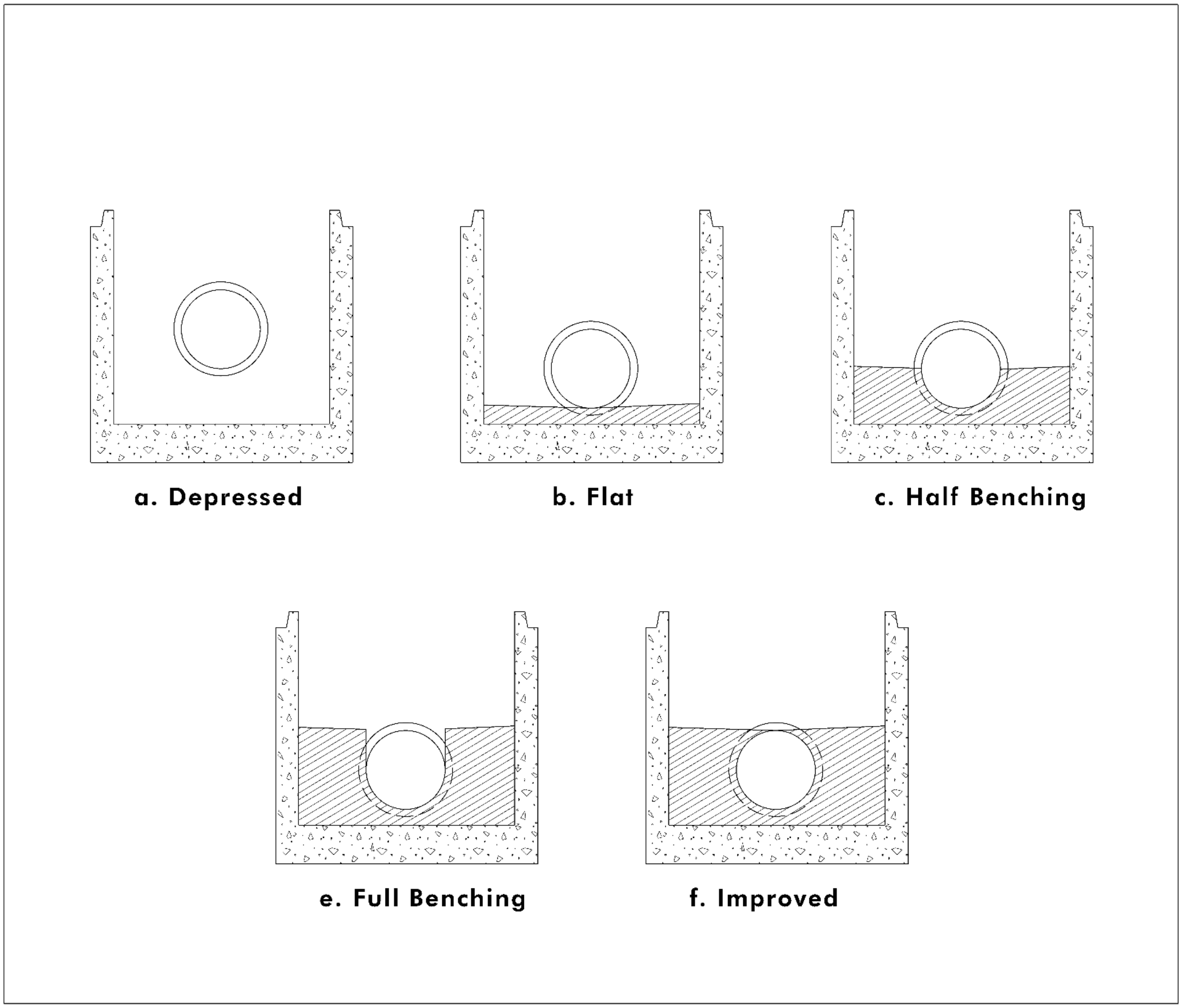
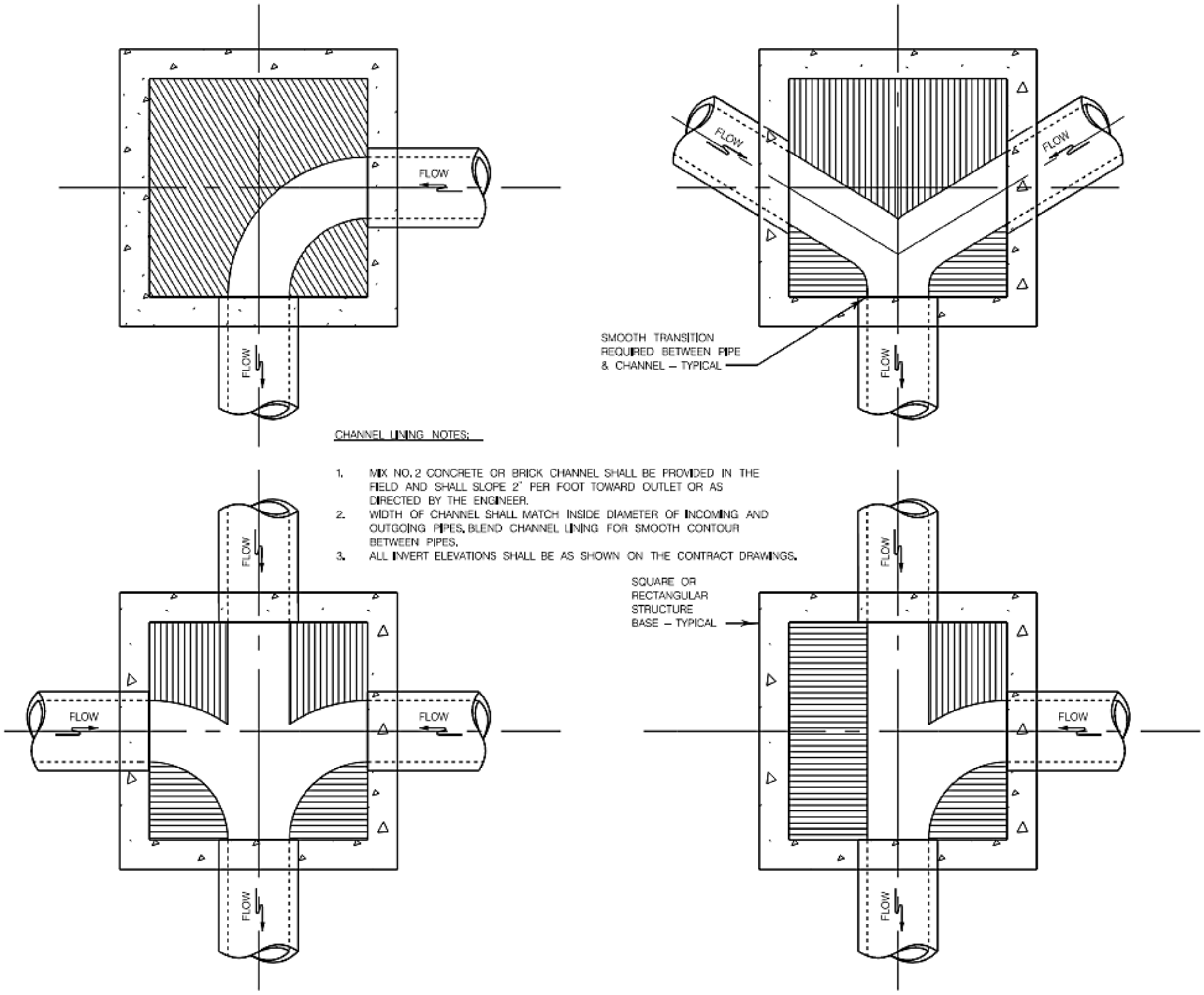


Figure 5-12: Access Hole Benching Methods



PLAN OF TYPICAL CHANNELIZATION OF SQUARE AND RECTANGULAR STORM DRAIN STRUCTURES

Figure 5-13: Square Structure Channelization Sketches

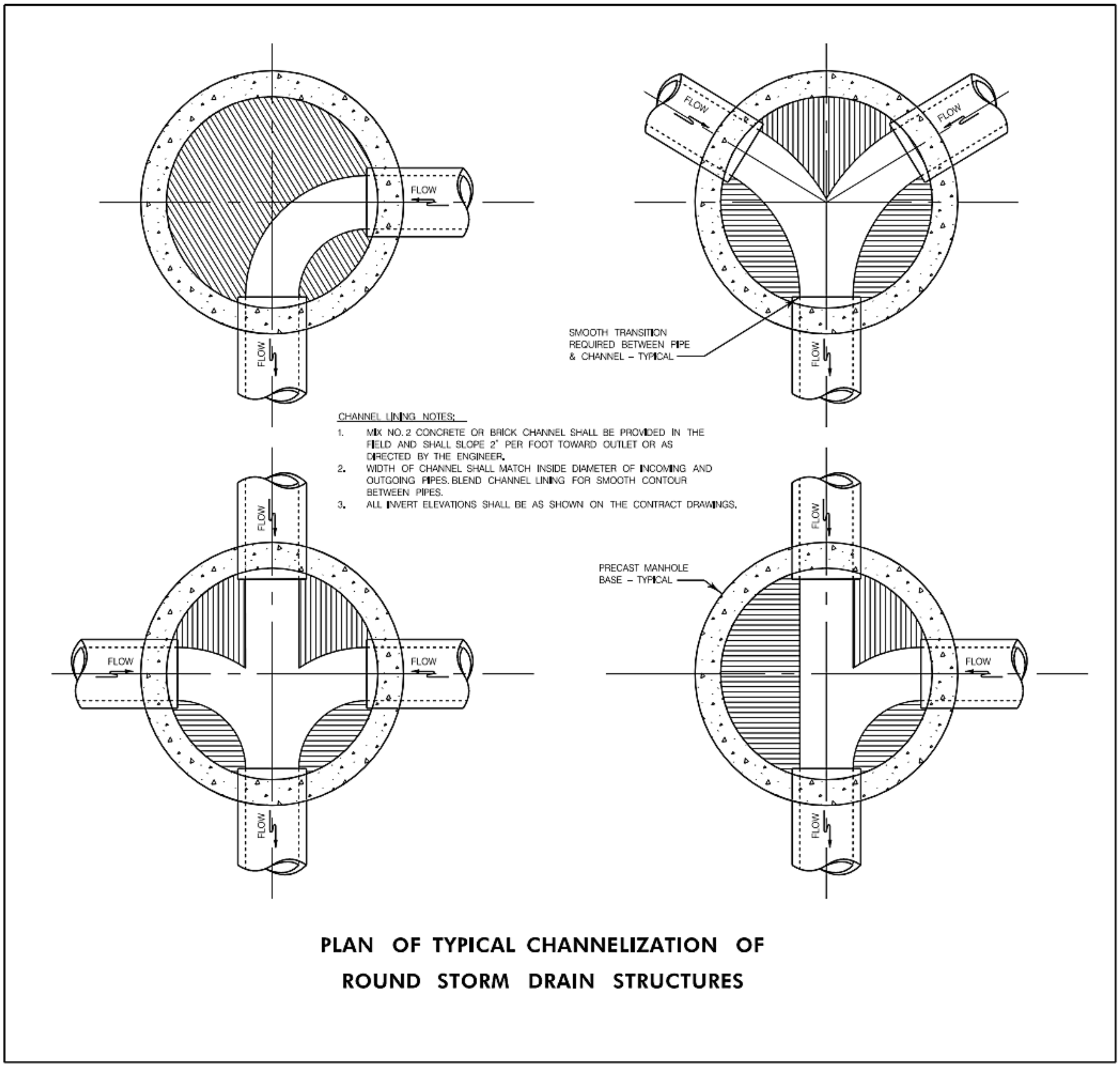


Figure 5-14: Round Structure Channelization Sketches

MARYLAND DEPARTMENT OF TRANSPORTATION

DATE: STATE HIGHWAY ADMINISTRATION PAGE OF

INLET SPACING

CONTRACT ______________________ **DESIGNED BY** ______________________

PROJECT ______________________ **CHECKED BY** ______________________

Inlet #	Station and Offset	A Area (ac.)	CA	Bypass CA	Total CA	Tc (min.)	I_2 (in./hr.)	Q_2 CFS	Inlet Type	S_x (ft./ft.)	Street Grade (ft./ft.)	T Spread (ft.)	L (ft.)	Pickup %	Bypass CA	To Inlet #	Remarks

Figure 5-15: Inlet Spacing Form

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STORM DRAIN DESIGN

CONTRACT ____________________ **RAINFALL FACTOR** ____________________ **DESIGNED BY** ____________________

PROJECT ____________________ **RETURN PERIOD** ____________________ **CHECKED BY** ____________________

Structure		Contributing Area				____ Year Runoff				Pipe								Remarks	Structure Type
From	To	C Area (#)	A Area (ac.)	Δ CA	Σ A	ΣCA	tc Time Conc. (min.)	i Rainfall Intens. (in./hr.)	Q cfs	Size (in)	Type	n Mann-ing's Coef.	So Slope (%)	Vo Vel. (fps)	L Length (ft.)	tt Travel Time (min.)	Capac. Full (cfs)		

Figure 5-16: Storm Sewer Design Form

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HYDRAULIC GRADIENT

CONTRACT ______________________ **RAINFALL FACTOR** ______________________ **DESIGNED BY** ____________

PROJECT ______________________ **RETURN PERIOD** ______________________ **CHECKED BY** ____________

Structure		Design Storm Runoff				Pipe									Hydraulic Gradient	
From	To	ΣCA	tc Time Conc. (min.)	i Rainfall Intens. (in./hr.)	Q cfs	Size (in)	Type	n Mann-ing's Coef.	So Slope (%)	Sf Slope (%)	Vf Vel. (fps)	L Length (ft.)	dn Normal Depth (ft)	Kb Bend Coeff.	Description of Loss	Elevation

Figure 5-17: Hydraulic Gradient